



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
NAMEWAMINIKAN RIVER BRIDGE REPLACEMENT
HIGHWAY 801, JELLCOE
DISTRICT OF THUNDER BAY, ONTARIO
LATITUDE: 49.7643°, LONGITUDE -87.7670°**

SITE No. 48C-0125

GEOCRES Number: 42E-33

Report

to

HATCH

Date: October 23, 2020
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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual data obtained from a foundation investigation carried out by Thurber Engineering Ltd. (Thurber) for the proposed Namewaminikan River Bridge replacement located on Highway 801, near Jellicoe, District of Thunder Bay, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the bridge location and, based on the data obtained, to provide a borehole location plan, stratigraphic profile, records of boreholes, laboratory test results, and a written description of the subsurface conditions.

Thurber was retained by the HATCH to carry out this foundation investigation under the MTO Agreement Number 6019-E-0009, Work Item #3.

2. SITE DESCRIPTION

The site is located on Highway 801, approximately 14 km north of the Highway 11/801 junction, near Jellicoe, District of Thunder Bay, Ontario. The existing bridge crosses Namewaminikan River, which flows in a east to west direction beneath the bridge. Highway 801 generally runs in a north-south direction at the bridge site. The river valley is approximately 33 m wide at the bridge location and up to 72 m wide just east and west of the bridge. The valley is up to 9 m deep from the crest of the slope to the bottom of the channel at the bridge location.

The Ontario Structure Inspection Manual (Inspection Form) prepared by MTO on March 1, 2013



indicates that the existing structure is a single span steel bailey panel bridge. It is understood that the bridge was constructed by MTO in approximately 2011 to replace a previous modular bridge. The inspection report indicates that the bridge deck is 33.5 m long and 6 m wide. The road leading to the bridge is a gravel surfaced road. Based on the Namewaminikan River Bridge Pre-Construction Drawings prepared by MTO in July 2009 the bridge deck is supported on bearing timbers and a timber abutment wall supported on 500 mm of crushed granular material. The ground surface elevation, based on an assumed benchmark elevation, at the existing bridge deck is approximately at Elevation 99.1 m at the south abutment and Elevation 98.7 m at the north abutment. The bridge is typically used by logging trucks.

The water level at Namewaminikan River Bridge was at an assumed elevation of 95.3 m as shown on survey drawings provided by HATCH.

A letter memo produced by Adamson Consulting on July 9, 2011, subject Namewaminikan River Bridge, 801 Road (Auden Road), indicated that differential settlement had been observed at the north abutment between the bridge deck and butt block (approach). Significant erosion of old timber cribs at both the north and south banks was also observed at this time. The old timber cribs at the banks are supporting the fill on which the new bearing timbers are supported.

A Bridge inspection completed by JML Engineering produced in February of 2020 also observed 25 to 40 mm of differential settlement between the top of the wearing surface and top of the ballast wall which was noted as a possible result of settlement of the bearing timbers. The inspection report also observed severe scouring at the waters edge at both the north and south embankments. A load restriction currently applies for the existing bridge.

A limited foundation investigation at the site was recently conducted by CSL Environmental and Geotechnical Ltd. consisting of 2 boreholes (1 per abutment), however the borehole information is not sufficient for detailed foundation design purposes as the depth to bedrock or competent founding soil was not confirmed by rock coring.

The lands surrounding the bridge site predominantly consist of heavily forested areas with lakes, swamps, rivers, and creeks. Local topography consists of undulating plains of low to moderate relief. Photographs of the bridge and surrounding area are presented in Appendix C.

Based on published geological information, the bridge lies within an area consisting of organic deposits of peat and muck and sandy soils overlying shallow bedrock with a knobby profile. Based on local geological maps, the bedrock in the area is identified as foliated tonalite to granite.



3. INVESTIGATION PROCEDURES

The current investigation and field testing program was carried out between July 9 to July 11, 2020 and consisted of drilling and sampling five (5) boreholes, labeled 20-01 to 20-05 to depths of between 3.4 m to 22.0 m (assumed Elevation 95.1 m to 77.2 m). One borehole was drilled at the south abutment and four (4) boreholes were drilled at the north abutment.

The approximate locations of the boreholes from the current investigation are shown on the Borehole Locations and Soil Strata Drawing included in Appendix D.

Utility clearances were obtained prior to the start of drilling. The ground surface elevations for the boreholes were estimated from the cross sections and topographic drawings provided to Thurber by HATCH. The elevations are relative to assumed local benchmarks provided on the survey plan by HATCH. The coordinate system MTM NAD 83, Zone 14 was used for the boreholes.

A truck mounted CME 750 drill rig was used to advance all the boreholes using hollow and solid stem augers. Soil samples were obtained in the boreholes at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). NQ coring methods were used to advance Boreholes 20-01, 20-02 and 20-05 3.4 to 3.7 m into bedrock.

The drilling and sampling operations were supervised on a full-time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

The rock core was logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Groundwater conditions were observed in the open boreholes upon completion of drilling. The boreholes were backfilled in general accordance with Ontario Regulation 903.

Completion details of the boreholes from the current investigation are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Borehole Number	Borehole Depth / Base Elevation (m)	Completion Details
20-01	22.0 / 77.2	Backfilled with bentonite holeplug to 3 m then sand and gravel to surface
20-02	7.1 / 91.5	Backfilled with bentonite holeplug to 1.8 m then sand and gravel to surface
20-03	3.9 / 94.6	Backfilled caved to 2.6 m then backfilled with sand and gravel cuttings to surface
20-04	3.4 / 95.1	Backfilled with sand and gravel cuttings to surface
20-05	7.3 / 91.2	Backfilled with bentonite holeplug to 3 m then sand and gravel to surface

4. LABORATORY TESTING

All recovered soil samples were subjected to visual identification (VI) and to natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (sieve and/or hydrometer). The results of this laboratory testing program are shown on the Record of Borehole sheets included in Appendix A and on the figures included in Appendix B.

In order to assess the potential for sulphate attack on concrete foundations, as well as the potential for corrosion associated with the structure, samples of the embankment fill from Borehole 20-01 at the south abutment and 20-03 from the north abutment were collected. A surface water sample was also collected from the Namewaminikan River upstream of the bridge location. The samples were then submitted to Bureau Veritas Laboratories a CALA accredited analytical laboratory in Mississauga, Ontario, for analytical testing of corrosivity parameters and sulphate content. The results of the analytical testing are summarized in Section 6 and are presented in Appendix B.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets and on the Borehole Locations and Soil Strata Drawing included in Appendix D. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following



paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and must be used for interpretation of the site conditions. It must be recognized and expected that soil conditions may vary between and beyond the borehole locations.

In general, the subsurface conditions encountered consisted of embankment fill, consisting of sandy gravel to sand overlying native silty sand at the north abutment and native silt at the south abutment. The native soils were further underlain by granodiorite bedrock. Descriptions of the individual strata are presented below.

5.1 Sandy Gravel to Gravelly Sand Fill

All boreholes were drilled within the gravel roadway of Highway 801 and the road surface and base consisted of sandy gravel to gravelly sand fill with some silt and occasional to some cobbles in all boreholes. The sandy gravel to gravelly sand fill was approximately 0.3 m to 1.5 m thick.

An SPT 'N' value measured in the gravelly sand fill was 10 blows for 0.3 m penetration, indicating a compact condition. The measured moisture content in the granular fill ranged from 4 to 7 percent.

The results of a grain size analysis conducted on a sample of the gravelly sand fill is provided on the Record of Borehole sheets in Appendix A, and illustrated in Figure B1 Appendix B. The results are summarized as follows:

Soil Particle	Percentage
Gravel	20
Sand	60
Silt	17
Clay	3

5.2 Sand Fill

Sand fill, with some silt and trace gravel and clay was encountered below the granular fill in Borehole 20-01 at the south abutment. The sand fill was approximately 3.2 thick and extended to a depth of 3.5 m (assumed Elevation 95.7).

SPT 'N' values measured in the sand fill ranged from 8 to 15 blows for 0.3 m penetration, indicating a loose to compact condition. The measured moisture content in the sand fill ranged from 7 to 9 percent.



The results of a grain size analysis conducted on a sample of the sand fill is provided on the Record of Borehole sheets in Appendix A, and illustrated in Figure B2 Appendix B. The results are summarized as follows:

Soil Particle	Percentage
Gravel	1
Sand	87
Silt	10
Clay	2

5.3 Silt

Native silt, containing trace to some sand and trace clay was encountered in Borehole 20-01 at the south abutment at a depth of 3.5 m (assumed Elevation 95.7 m) beneath the sand fill. The silt layer was approximately 15.1 m thick extending to bedrock at a depth of 18.6 m (assumed Elevation 80.6 m).

SPT 'N' values measured in the silt generally ranged from 0 (weight of hammer) to 24 blows for 0.3 m penetration, indicating a very loose to compact condition. The measured moisture content in the silt ranged from 15 to 23 percent.

The results of grain size analyses tests conducted on samples of the silt are provided on the Record of Borehole sheets in Appendix A, and illustrated in Figure B3 of Appendix B. The results are summarized as follows:

Soil Particle	Percentage
Gravel	0
Sand	1 to 12
Silt	83 to 94
Clay	4 to 5

5.4 Silty Sand

Native silty sand, containing trace clay, was encountered below the sandy gravel to gravelly sand fill in Boreholes 20-02 to 20-05 at the north abutment. The silty sand was approximately 1.9 to 3.1 m thick and extended to depths of between 3.4 m to 3.9 m (assumed Elevation 94.6 and 95.2 m).



SPT 'N' values measured in the silty sand generally ranged from 2 to 16 blows for 0.3 m penetration, indicating a very loose to compact condition. The measured moisture content in the silty sand ranged from 4 to 22 percent.

The results of grain size analyses tests conducted on samples of the silty sand are provided on the Record of Borehole sheets in Appendix A, and illustrated in Figure B4 of Appendix B. The results are summarized as follows:

Soil Particle	Percentage
Gravel	0 to 5
Sand	68 to 70
Silt	26 to 29
Clay	1 to 3

5.5 Bedrock

The overburden soils described above are underlain by granodiorite bedrock. The bedrock was grey to dark grey and of intermediate mafic composition. The bedrock is generally described as slightly weathered and contains quartz seems. Bedrock was proved by coring in Boreholes 20-01, 20-02, and 20-05. The bedrock dips steeply from the north abutment to the south abutment. Table 5.1 summarizes the depths and elevations to the top of bedrock.

Table 5.1 - Depths and Elevations of Top of Bedrock

Location	Borehole	Top of Bedrock		Details
		Depth (m)	Assumed Elevation (m)	
South Abutment	20-01	18.6	80.6	Proved by coring
North Abutment	20-02	3.4	95.2	Proved by coring
	20-03	3.9	94.6	Auger refusal on assumed bedrock surface
	20-04	3.4	95.1	Auger refusal on assumed bedrock surface
	20-05	3.9	94.6	Proved by coring



Total Core Recovery (TCR) in the bedrock ranged between 69% to 100% and Solid Core Recovery (SCR) ranged between 27% and 96%. The Rock Quality Designation (RQD) determined from the recovered cores ranged between 0% and 84%, which indicates a very poor to very good rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 10.

Average unconfined compressive strengths (UCS) of the rock typically ranged between 25 and 101 MPa at the north abutment and 168 to 307 MPa at the south abutment, indicating the rock is medium strong to extremely strong. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. A summary of the point load tests results presented in Appendix B.

5.6 Groundwater Conditions

Groundwater conditions were observed during drilling operations and groundwater levels were measured in the open boreholes upon completion of drilling. A summary of the water level measurements is provided in Table 5.2 below:

Table 5.2 - Groundwater Measurements

Borehole	Date	Water Level (m)		Remark
		Depth	Elevation*	
20-01	July 10, 2020	3.1	96.1	Water level likely represents water added to borehole while coring bedrock
20-02	July 10, 2020	1.1	97.5	Water level likely represents water added to borehole while coring bedrock
20-03	July 10, 2020	-	-	Borehole caved to 2.6 m, no water level measurement possible
20-04	July 11, 2020	3.3	95.2	Open borehole
20-05	July 11, 2020	1.1	97.4	Water level likely represents water added to borehole while coring bedrock

* Assumed Elevation



Water was added to Boreholes 20-01, 20-02, and 20-05 during the coring operations, and water levels recorded in those boreholes may not necessarily represent the groundwater level. The groundwater level should be assumed to reflect the river water level. The water level of Namaewaminiken River was measured at an assumed Elevation 95.3 m as shown on survey drawings provided by HATCH.

Groundwater levels are short-term observations and seasonal fluctuations of the groundwater levels are to be expected. In particular, the groundwater levels may be at a higher elevation during spring and after periods of significant or prolonged precipitation.

6. CORROSIVITY AND SULPHATE TEST RESULTS

In total, two soil samples and a sample of the creek water from Namewaminikan River were submitted for analytical testing of corrosivity parameters and sulphate. The results of the analytical tests are shown in Table 6.1. The laboratory certificates of analysis are presented in Appendix B.

Table 6.1 - Analytical Test Results

Parameter	Units (Soil)	Units (Water)	Test Results		
			20-01 SS 2 Depth 1.1 m	20-03 SS 2 Depth 1.1 m	Namewaminikan River
			(Sand Fill)	(Silty Sand Fill)	(River Water)
Sulphide	%	mg/L	-	-	<0.02
Chloride	µg/g	mg/L	<20	<20	2.3
Sulphate	µg/g	mg/L	<20	<20	<1.0
pH	No unit	No unit	8.06	7.63	7.82
Electrical Conductivity	µmho/cm	µmho/cm	64	85	130
Resistivity	Ohms.cm	Ohms.cm	16000	12000	7800

7. MISCELLANEOUS

Thurber obtained subsurface utility clearances prior to drilling. The northing and easting coordinates and ground surface elevations were estimated based on field measurements relative



to the topographic plans provided by HATCH.

RPM Drilling of Thunder Bay, Ontario supplied and operated the drilling, sampling and in-situ testing equipment for the field investigation. The field investigation was supervised on a full-time basis by Mr. Greg Stanhope of Thurber. The overall supervision of the field program was conducted by Mr. Cory Zanatta, P.Eng., of Thurber.

Geotechnical laboratory testing was carried out in Thurber's geotechnical laboratory. Analytical laboratory testing was carried out by Bureau Veritas Laboratories.

Interpretation of the field data and preparation of this report was carried out by Mr. Cory Zanatta, P. Eng. and Mark Farrant, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

8. GENERAL

This report provides an interpretation of the geotechnical data in the factual report, and presents foundation recommendations for design of the proposed replacement of the Namewaminikan River Bridge on Highway 801, located near Jellicoe, District of Thunder Bay, Ontario.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

A letter memo produced by Adamson Consulting on July 9, 2011, subjected Namewaminikan River Bridge, 801 Road (Auden Road), indicated that 25 mm of differential settlement had been observed at the north abutment between the bridge deck and butt block (approach). Significant erosion of old timber cribs at both the north and south banks was also observed at the time. The old timber cribs at the banks are supporting the fill on which the new bearing timbers are supported.

A Bridge inspection completed by JML Engineering produced in February of 2020 also observed 25 to 40 mm of differential settlement between the top of the wearing surface and top of the ballast wall at both approaches which was noted as a possible result of settlement of the bearing timbers. The inspection report also observed severe scouring at the water's edge at both the north and south embankments. The JML Engineering report also included a foundation investigation by CSL



Environmental and Geotechnical Ltd. This included a slope stability analysis that indicated that both abutment foreslopes exhibited a factor of safety against failure of less than the minimum required value of 1.5.

Information on the existing bridge site was obtained from the MTO Terms of Reference, the Ontario Structure Inspection Manuals (OSIMs) prepared by MTO on March 3, 2013, and the MTO Pre-Construction Drawings of the existing bridge. The existing structure is a single span steel bailey bridge built in 2011 replacing the original bailey bridge constructed in 1969. The inspection reports indicate that the bridge deck is approximately 33.5 m long and 6 m wide. Based on the Namewaminikan River Bridge Pre-Construction Drawings prepared by MTO, the bridge deck is supported on bearing timbers and a timber abutment wall supported on 500 mm of crushed granular material. The existing bridge is typically used by logging trucks.

It is understood that due to the settlement, erosion and forward slope stability concerns, the bridge currently has a 5 Tonne Load restriction, which is not sufficient for upcoming hauling loads needed on Highway 801. The current investigation was conducted to address foundation replacement options as part of a detailed design of the bridge replacement, and to address stabilization of the foreslopes.

9. STRUCTURE FOUNDATIONS

The subsurface stratigraphy at the site typically consists of embankment fill, consisting of sandy gravel to gravelly sand, overlying native silt on the south bank and silty sand on the north bank. The native soils are underlain by granodiorite bedrock at approximately 18.6 m below the top of embankment at the south bank, and 3.4 m to 3.9 m at the north bank.

Groundwater levels were measured in open boreholes upon completion of drilling were between 1.1 m and 3.3 m below ground surface elevation (assumed elevations 95.2 m to 97.5 m). The water level of Namewaminikan River was measured at approximately assumed Elevation 95.3 as shown on survey drawings provided by MTO.

Due to the observed settlements of the bearing timbers at both abutments, the new abutments of the replacement bridge are proposed to be set back 3 m behind the existing abutments. Based on the subsurface conditions, consideration was given to supporting the replacement bridge on the following foundation types:

- Spread footings placed on engineered fill pads, and
- Steel H-piles driven to bedrock or socketed into bedrock



A comparison of the technical advantages and disadvantages of the alternative foundation options is presented in Appendix F.

Footings directly founded on native soil or bedrock would require deep excavation to reach competent stratum and extend below the groundwater level. Additionally, recent observations at the site suggest that differential settlement may be an issue here and footings founded on native soils would increase that risk. Hence recommendations were not developed further for this option.

Recommendations for design of feasible foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation option from a geotechnical perspective is recommended.

9.1 Spread Footings on Engineered Fill Pads

9.1.1 Founding Level

Based on the subsurface conditions encountered at this site. The use of spread footings placed on minimum 2 m thick engineered granular fill pads is considered feasible from a geotechnical perspective.

At the north abutment, the base of the engineered fill pad may be placed on the native silty sand encountered at 0.7 to 1.5 m depth in the boreholes or below assumed Elevation 97.1 m to 97.8 m. At the south abutment the base of engineered fill pad may be placed on the existing sand embankment fill at or below a depth of 1.5 m (assumed Elevation 97.7 m).

9.1.2 Engineered Fill Construction

The engineered fill pads should consist of OPSS Granular A or Granular B Type II placed in 150 mm lifts and compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD) at $\pm 2\%$ of Optimum Moisture Content (OMC).

For construction of the engineered fill pad, the following construction sequence may be considered:

1. The minimum depth of excavation should accommodate the concrete foundation slab and the thickness of engineered fill pad below the slab;
2. The subgrade for the engineered fill pad should be inspected and all organics, soft/loose soils, and any deleterious materials should be removed from the footprint of the excavation and replaced with compacted Granular A or Granular B Type II;

3. Dewatering measures should be provided, as required, to place the engineered fill in the dry;
4. The dimensions of the base of the excavation should be determined by assuming a granular pad 1.0 m wider than the spread footing at the level of the footing base and projecting outward and downward at 1H:1V.

If the engineered fill pads are located close to the river valley, the forward slope of the foundation pads should be embedded at least 1.0 m below a 2H:1V face of the forward slope, with the front edge of footing at a minimum of 3 m behind the crest of the forward slope. Provision of properly designed erosion protection works of the forward and side slopes will be critical to ensure adequate performance of the foundations/engineered fill pads.

9.1.3 Axial Geotechnical Resistance and Geotechnical Reaction

The following values of factored Geotechnical Resistance at ULS and factored Geotechnical Resistance at SLS may be used for preliminary design of spread footings (assuming footing widths of 1, 1.5 or 2 m) placed on 2 m thick engineered fill pads prepared as outlined below:

Table 9.1 – Geotechnical Resistances for Footings on Engineered Fill Pads

Footing Width (m)	Geotechnical Resistance (kPa)	
	Factored ULS	Factored SLS
1	275	225
1.5	250	165
2	225	135

The values of the factored Geotechnical Resistance at SLS correspond to up to 25 mm of settlement.

The value of Factored Geotechnical Resistance at ULS was assessed assuming a Consequence Factor equal to 1 (Typical), and a Resistance Factor equal to 0.5 (Typical degree of understanding of the subsurface conditions), as per Canadian Highway Bridge Design Code (CHBDC). The factored Geotechnical Resistance at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

The geotechnical resistance quoted above is for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance should be calculated as indicated in the CHBDC 2014 Clause 6.10.3 and Clause 6.10.4.



Results of the slope stability analyses of the forward river valley slopes (presented in Section 15), show a low Factor of Safety. As a result, a strip footing on engineered pad foundation is not recommended at this site without significant slope stability enhancement measures.

9.1.4 Sleeper Slab

For a 0.9 m wide concrete sleeper slab supported on a minimum 1 m thick engineered fill pad below the base of the slab, the recommended geotechnical resistances are 135 kPa at the factored ULS and 90 kPa at the factored SLS. The sleeper slab should be embedded at least 0.5 m in the engineered fill pad and set back a minimum distance of 0.5 m behind the crest of the forward slope of the pad with inclination not steeper than 1.5H:1V. The forward slope and side slopes of the engineered fill pad should be protected against erosion as per OPSS 511.

9.1.5 Lateral Resistance

The lateral resistance of the concrete footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.5. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance. A resistance factor of 0.8 should be used for concrete footing sliding on the engineered granular fill pad in accordance with the CHBDC.

9.2 Steel H-Pile Foundations

Steel H-piles, driven to refusal on bedrock may be used to support both abutments. The depth to bedrock at the north abutment is shallow (3.4 to 3.9 m) and it should be checked whether 3 to 4 m pile length is adequate to satisfy lateral pile stability. If the 3 to 4 m pile length is not sufficient, then the H-Piles at the north abutment may need to be socketed into the bedrock to satisfy lateral pile stability. At the south abutment in Borehole 20-01, bedrock was encountered at 18.6 m depth. At this abutment piles may be driven to bedrock and socketing into bedrock is likely not required at the south abutment. The factored Geotechnical Resistances and the estimated tip elevations recommended for HP 310x110 piles driven to the bedrock surface are presented below in Table 9.2.

Table 9.2 – Recommended Axial Geotechnical Resistances for Steel HP 310x110 Piles

Foundation Element	Approximate Pile Tip Depth Below Existing Ground (m)/Assumed Elevation	Factored ULS Geotechnical Resistance Per Pile (kN)	SLS Resistance (kN)
North Abutment (20-02 to 20-05)	3.4 to 3.9 / 94.6 to 95.2	2,500	Does not govern
South Abutment (20-01)	18.6 / 80.6	2,500	Does not govern

The actual pile tip elevations may vary during installation due to the potential sloping bedrock.

The axial resistances based on the bedrock strength are expected to exceed the factored structural capacity of the pile. Accordingly, the structural capacity of the HP 310x110 (2000 kN per pile) will govern the design.

The SLS condition will not govern the design of piles founded on bedrock.

The structural resistance of the pile must be checked by the structural designer.

Since there is no grade raise or embankment widening proposed, downdrag on the piles is not considered to be an issue at this site.

9.2.1 H-Pile Installation

Piles installation must be in accordance with OPSS.PROV 903.

Due to the likely presence of steeply dipping bedrock at the abutment locations, the pile tips of any driven piles should be equipped with rock injector points from an approved manufacturer such as Titus Steel, or approved equivalent. This is to prevent the pile tip from sliding or skipping down the bedrock slope and to ensure the pile is fixed in a firm seat on the bedrock surface. A NSSP describing a procedure for setting a driven pile on sloping bedrock is included in Appendix G.

9.2.2 Pile Lateral Resistance in Soil

The geotechnical lateral resistance acting on an H-pile in cohesionless soil may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \gamma' z K_p \quad (\text{kPa})$$

Where

z = depth of embedment of pile (m)

D = pile width or diameter (m)

n_h = coefficient related to soil relative density (kN/m^3)

γ' = effective unit weight (kN/m^3)

K_p = passive earth pressure coefficient

For analysis of the interaction between a pile and the surrounding soil, the above equations and parameters recommended in Table 9.3 below, may be used. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

Table 9.3 – Soil Parameters for Lateral Pile Resistance

Abutment	Assumed Elevation	S_u (kPa)	n_h (kN/m^3)	K_p	Unit Weight* (kN/m^3)	Soil Conditions
North	98.6 to 94.6	-	2,500	3.0	20	Very loose to compact silty sand
South	99.2 to 95.7	-	2,500	3.0	20	Loose to compact sand fill
	95.7 to 80.6	-	3,000	3.0	10*	Very loose to compact silt

*Bouyant unit weight below groundwater level

The lateral resistance in the upper 2.4 m of each pile should be neglected to account for frost action. The spring constant, K_s , for analysis may be obtained by the expression, $K_s = k_s L D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} L D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

The modulus of subgrade reaction and ultimate lateral resistance may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Section C.6.11.3.4 of CHBDC.



Horizontal loads may be resisted by means of battered piles (i.e. for H-pile case) if load requirements exceed the available lateral pile resistances.

9.2.3 Pile Lateral Resistance in Rock Socket

The ultimate passive resistance that can be mobilized by the embedded portion of a pile socketed in the bedrock may be assumed to be a constant with depth and is given by:

$$P_p = 6 C D L$$

Where $C = 3,000 \text{ kPa}$ (strength of rock mass)

$D =$ Socket diameter (m)

$L =$ Depth of socket in rock (m)

The depth of the pile socket should be calculated by the design engineer but should be socketed into bedrock a minimum of 2.0 m. Deeper sockets may be required to meet lateral resistance requirement. The socket should have a minimum diameter of 610 mm to accommodate a HP 310x110. The annular space between the H-Piles and socket wall should be filled with 30 Mpa tremied concrete. A temporary liner sealed into bedrock will be required through the cohesionless overburden in order to excavate the socket and to prevent collapsing or washing of cohesionless soils in the rock socket. Contractor must be alerted that they will be excavating the socket in very hard granodiorite rock. A NSSP to this effect is included in Appendix G.

9.3 Frost Cover

The depth of frost penetration at this site is approximately 2.4 m, as per OPSD 3090.100. Typically, the base of all footings and pile caps, if employed, must be provided with a minimum of 2.4 m of earth cover as protection against frost action.

Concrete spread footings founded on granular engineered fill pads, provided they consist of non-frost susceptible, free draining engineered fill, above the river water level should be provided with a minimum embedment of 0.5 m. These footings do not need to be placed below the depth of frost. If the design changes to a conventional bridge instead of a modular bridge, then at least 2.4 m of frost cover will be required for all footings.

9.4 Recommended Foundation

From cost effectiveness and constructability perspectives and in light of the observed settlement, instability and scouring of the foreslopes, steel H-Piles driven to bedrock or socketed into bedrock are the preferred foundation option at this site.

10. TEMPORARY DETOUR

No temporary detour is required during construction of the project since a full road closure is proposed for construction of the replacement bridge.

11. LATERAL EARTH PRESSURES

Backfill behind the abutments should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150. All granular material should meet the specifications of OPSS.PROV 1010 as amended by Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS.PROV 501.

Earth pressures acting on the abutment walls may be assumed to be triangular and to be governed by the characteristics of the abutment backfill and the underlying native soils. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K^*(\gamma h + q)$$

Where:

p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 10.1)

γ = unit weight of retained soil (see Table 10.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 11.1.

Table 11.1 – Earth Pressure Coefficient (K)

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Existing Sand and Gravel Fill $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface	Sloping Surface in front of Wall (2H:1V)	Horizontal Surface	Sloping Surface in front of Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is preferred as it results in lower earth pressures acting on the wall.

The factors in Table 11.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.12.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 1.7 m for Granular B Type I or 2.0 m for Granular A or Granular B Type II.

12. EXCAVATION AND GROUNDWATER CONTROL

All excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the sand and gravel fill, silty sand to gravelly sand fill, and native sands and silts at this site are classified as Type 3 soils above the water level and Type 4 soils below the water level.



Excavation and backfilling for the bridge construction should be carried out in accordance with OPSS 902. Excavations for the bridge replacement will be carried out through the existing sand and gravel fill and native silty sand. It must be noted that obstructions may be encountered within the fill such as cobbles and boulders, or timbers from old bridge foundations and cribs. To avoid working in the water, only obstructions above the river water level should be removed. Any existing timbers or obstructions below the river water level should be left in place.

Installation of the foundations should be carried out in the dry. It is understood that all constructions in intended to be conducted above the river water level and that minimal dewatering will be required for control of the river water.

If the excavations for abutments are extend below the groundwater level, then seepage should be anticipated from the embankment fill and native soils. In this case, the water level must be depressed below the base of the excavation to permit construction in the dry and to facilitate compaction of the backfill materials.

Roadway protection will not be required for this site as it is understood that the road will be closed for the duration of the project.

13. SEISMIC CONSIDERATIONS

In accordance with the CHBDC 2014, the selection of the seismic site class is based on the soil conditions encountered in the upper 30 m of the stratigraphy. In view of the presence of the silt deposit at the north abutment, which contains very loose conditions and has an average SPT 'N' value of less than 15 for its 15 m thickness, the site should be classified as Site Class E in accordance with Table 4.1, Clause 4.4.3.2 of the CHBDC. The peak ground acceleration, PGA, for a 2,475-year return period seismic event at this site is 0.062 g as per the National Building Code of Canada (NBCC).

In accordance with Clause 4.6.5 of the CHBDC 2014, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 13.1 may be used:

Table 13.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)		
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	Existing Fill $\phi = 28^\circ; \gamma = 20 \text{ kN/m}^3$
Active (K_{AE})*	0.29	0.33	0.39
At Rest (K_{OE})**	0.51	0.55	0.61
Passive (K_{PE})	3.6	3.2	2.7

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

The site is underlain by shallow bedrock at the north abutment, and a very loose to compact silt deposit at the south abutment. The majority of the silt deposits may be considered as compact and considering the shallow bedrock at the north abutment and low potential for seismic activity in the area, liquefaction is not considered to be a concern at this site.

14. SCOUR AND EROSION PROTECTION

Erosion protection should be provided along any soil surfaces that may be in contact with the river flow. In particular, erosion protection must be provided to prevent loss of soils in front of the abutment walls. Rock protection of the forward valley slopes is critical at this site.

The existing old timbers, and timber crib embedded in the foreslopes at both the north and south abutments, are susceptible to rot and decomposition. The decomposing timbers may initiate soil movement within the embankments. Efforts should be made to remove exposed timbers above the river water level prior to implementing rock protection. Timbers below the water level should be left in place to avoid work in the water.

Design of the erosion protection measures should consider hydrologic and hydraulic factors and should be carried out by specialists experienced in this field and in accordance with OPSS 511 and OPSS PROV 1004. A vegetation cover should be established on all other exposed earth surface to protect against surficial erosion, in general accordance with OPSS 804.

15. SLOPE STABILITY ANALYSIS

A stability analysis of the existing south forward valley slope, as well as a global stability assessment of an assumed condition for a scenario which involves replacement of the foundations with footings founded on H-Piles at both the north and south slopes was carried out



with a commercially available program SLOPE-W. A cross-section along the profile of the bridge was estimated based on data from the JML report and Adamson memos and was used for all the stability analyses. The existing forward slopes above the water line based on the survey data, is at approximately 2H:1V slope angle.

The existing slopes on both the north and south sides are undergoing heavy erosion. Old timber cribs from previous iterations of the bridge are located within the slopes on both sides and are also undergoing erosion and are falling apart. In the analysis of the existing condition, the influence of the timber cribs was not included as it is considered to be failing and will eventually no longer provide any support to the slope.

The assumed H-Pile foundations geometry is based on drawings provided by HATCH which includes moving the front edge of the foundation elements 3 m back away from the existing foundations at both the north and south abutments, extending the span of the bridge a total of 6 m. The design also includes removal of the timber cribs, unloading of soil at the crests of the slopes, and placement of 1 m of rock protection along the crest of the slope from the bank of the river up to the abutments. No loading was applied on the footings to the underlying soils is the analysis as the bridge load would be transferred through the piles to the underlying bedrock.

The results of the stability analysis are provided in Appendix E. The Table 15.1 below summarizes the soil parameters used for evaluating the existing slope and proposed design conditions.

Table 15.1 – Slope Stability Soil Parameters

Soil	$\gamma(\text{kN/m}^2)$	$c'(\text{kPa})$	$\phi'(^{\circ})$
Sand Fill	19	0	30
Sandy Gravel Fill	21	0	35
Silt	19	0	30
Silty Sand	20	0	30
Silt- Compact	19	0	32
Rock Protection	18	0	42

The ground water level used in the analysis was based on the most recent river water level of 95.3 m. Table 15.2 below summarizes the attained Factor of Safety (F.S.) for the analyses described above.

Table 15.2 – Results of Stability Analyses

Analyses	Factor of Safety (Drained Analysis)	Appendix E Figure Number
Existing South Slope River Valley	1.0	Figure E1
Proposed South Abutment H-Pile Foundation	1.4	Figure E2
Proposed North Abutment H-Pile Foundation	1.6	Figure E3

The stability analysis of the existing river valley shows a F.S. of 1.0 against sliding failure through the existing footing. This would suggest that the existing timber crib located within the slope is helping to prevent slope instability. However visual evidence suggests that the soils around the timber crib are eroding away.

The analyses shows that the new bridge design with an H-Pile foundation set back 3 m from the existing crest of slope with piles driven to or socketed in bedrock has a F.S. of 1.4 at the south abutment and a F.S. of 1.6 at the north abutment against sliding failure through the foundation. The analysis includes removal of the existing timber cribs and removal of soil from the crest of the slope down to the river water level, and placement of 1 m of rock protection along the crest of the slope from the bank up to the new abutments. The new slope geometry should consists of a 2H:V slope that starts at the top 0.6 m inwards from the new abutments and extends down to 1 m above the river water level, then carries horizontally across the existing river bank.

While the south slope F.S. is less than the typically required 1.5 for long term stability, based on Materials Engineering and Research Office (MERO) memo #2020-01 dated March 23, 2020, a factor of safety of 1.3 to 1.5 would be considered acceptable based on a typical to high degree of understanding at the site. At this site, a number of boreholes have been conducted for this investigation and in past investigations, at both the north and south abutments, to assess the subsurface conditions, with boreholes extending down and into bedrock at both abutments. Thus it can be considered that the degree of understanding at this site ranges between typical to high and that a F.S. of 1.4 is considered acceptable.

Improvements to the north and south slopes as described here, including removal of the timber cribs, removal of soil at the crest of slopes, and placement 1 m of rock protection, are essential in increasing the stability of the slope. The geometry of the foreslopes of any new bridge design



should follow very closely to the geometry of the slopes as presented in Figures E2 and E3 in Appendix E. If there is any deviation from this arrangement additional slope stability analyses should be conducted to confirm the required stability can be achieved.

16. APPROACH EMBANKMENTS

The existing Highway 801 embankment fill is approximately 4 m to 5 m in height at the existing bridge location and no grade raise is anticipated for the proposed bridge replacement. The existing embankment side slopes appear to be stable. If grade raise is required, settlement due to grade raise should be assessed.

At the south and north abutments, provided that the embankment is reconstructed at the same slope inclination as the existing embankment, but not steeper than 2H:1V, the restored embankment slope should remain stable.

Embankment restoration after completion of the bridge replacement should be carried out in accordance with OPSS.PROV 206. The embankment material may consist of imported Granular A, Granular B Type II, or Granular B Type III material.

In general, surface vegetation, peat, topsoil, organic deposits, disturbed material or otherwise loose/soft soils should be stripped from the areas within the reinstated embankment footprint. Inspection and approval of the foundation subgrade by qualified geotechnical personnel should be conducted.

17. CORROSION AND SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on the samples of the sand fill, silty sand soils and river water indicate the following conditions at the locations tested:

- The potential for sulphate attack or corrosion on concrete foundations from the surrounding sands or river water is considered to be negligible due to low concentrations of sulphate and chloride in the samples tested. The effect of road deicing salt should also be considered while selecting the class of concrete.
- The potential for soil or water corrosion on metal is considered to be very mild to mild.
- Appropriate protection measures are recommended if metal structural elements are used. The effect of road deicing salt should be considered while selecting the corrosion protection measures.

18. CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- The exposed timbers embedded within the foreslopes at both abutments should be removed and the slopes re-instated to a 2H:1V and proper erosion protection, such as rock protection, should be implemented at the banks.
- Due to the steeply sloping bedrock at the site, the top of bedrock elevations may vary significantly. In addition, driven steel H-Piles should be equipped with rock injector points.
- To reduce the risk of slope stability issues and settlement of the new foundations, the front edge of the new bridge foundation elements should be placed a minimum of 3 m back from the crest of the river valley slopes at the north and south abutment, respectively.
- The water level in the river may fluctuate and be at higher elevation at the time of construction than indicated in the report.
- The Contractor's selection of construction equipment and methodology should include assessment of the capability of the existing embankment to support the proposed construction equipment and any temporary structures or fill (i.e., as a pad for crane support). Site conditions may limit the type of equipment suitable for use during construction. The design and safety of any temporary works is the responsibility of the Contractor.
- Use of a crane or heavy construction equipment may be needed for construction of this bridge. An NSSP for use of heavy construction equipment and crane support has been provided in Appendix G

19. CLOSURE

Engineering analysis and preparation of this report was carried out by Mr. Cory Zanatta, P.Eng. and Mr. Keli Shi, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer


4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$


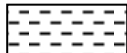



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS W _L < 50%	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. (W _L < 30%).
		CI	Inorganic clays of medium plasticity, silty clays. (30% < W _L < 50%).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS W _L > 50%	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Very thinly bedded	20 to 60mm				
Laminated	6 to 20mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Thinly Laminated	Less than 6mm				
<u>TERMS</u>		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.				
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No 20-01

1 OF 3

METRIC

W.P. _____ LOCATION MTM Zone 14 N 5 514 368.8 E 249 567.7 ORIGINATED BY GS
 DIST _____ HWY 801 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2020.07.09 - 2020.07.10 LATITUDE _____ LONGITUDE _____ CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
99.2	GROUND SURFACE													
0.0	Sandy GRAVEL , some silt, some cobbles Brown Moist (FILL) SAND, some silt, trace gravel Loose to Compact Grey to Brown Moist (FILL)		1	GS			99							1 87 10 2
98.9			2	SS	8		98							
0.3			3	SS	15		97							
			4	SS	10		96							
95.7	SILT, trace to some sand, trace clay Compact to Very Loose Brown to Grey Wet		5	SS	12		95							0 12 83 5
3.5			6	SS	19		94							
			7	SS	15		93							
			8	SS	0		92							
			9	SS	1		91							
							90							

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 20-01

2 OF 3

METRIC

W.P. _____ LOCATION MTM Zone 14 N 5 514 368.8 E 249 567.7 ORIGINATED BY GS
 DIST _____ HWY 801 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2020.07.09 - 2020.07.10 LATITUDE _____ LONGITUDE _____ CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
	Continued From Previous Page													
			10	SS	7		89							0 8 88 4
							88							
			11	SS	10		87							
							86							
			12	SS	22		85							0 1 94 5
							84							
			13	SS	24		83							
							82							
			14	SS	15		81							
							80							
80.6			15	SS	100/									
18.6	Weathered, phaneritic, massive, dark grey: (GRANODIORITE BEDROCK) Sub-vertical fracture from 18.6 to 18.9m Rubble zone from 18.6 to 18.7m Horizontal fractures at 18.8m and 19.0m Sub-horizontal fractures at 18.9m, 19.3m and 19.4m Vertical fractures from 18.9 to 19.1m		1	RUN	0.100									RUN #1 TCR=100% SCR=58% RQD=45%
														RUN #2 TCR=93% SCR=83% RQD=68%

Continued Next Page

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 20-01

3 OF 3

METRIC

W.P. _____ LOCATION MTM Zone 14 N 5 514 368.8 E 249 567.7 ORIGINATED BY GS
 DIST _____ HWY 801 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2020.07.09 - 2020.07.10 LATITUDE _____ LONGITUDE _____ CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)					
								20 40 60 80 100				w _p w w _L					
						○ UNCONFINED + FIELD VANE											
						● QUICK TRIAXIAL × LAB VANE											
						20 40 60 80 100				20 40 60							
	Continued From Previous Page																
	and 19.3 to 19.4m Horizontal fractures at 19.5m, 19.6m, 19.7m, 19.8m, 20.3m, 20.5m and 20.7m Rubble zone from 19.5 to 19.6m Horizontal fracture at 20.1m		2	RUN			79								1	RUN #3 TCR=91% SCR=86% RQD=68%	
	Sub-horizontal fracture at 21.0m														1		
	Horizontal fractrues at 21.0m, 21.1m, 21.2m, 21.5m and 21.8m		3	RUN			78								2		
															2		
															5		
77.2															1		
22.0	END OF BOREHOLE AT 22.0m. WATER LEVEL MEASURED AT 3.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 3.0m THEN SAND AND GRAVEL TO SURFACE.																

RECORD OF BOREHOLE No 20-02

1 OF 1

METRIC

W.P. _____ LOCATION MTM Zone 14 N 5 514 391.2 E 249 524.2 ORIGINATED BY GS
 DIST _____ HWY 801 BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2020.07.11 - 2020.07.11 LATITUDE _____ LONGITUDE _____ CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
98.6	GROUND SURFACE													
0.0														
98.3	Sandy GRAVEL , some silt, some cobbles		1	GS										
0.3	Brown Moist (FILL)													
	Gravelly, SAND , some silt, trace clay		2	SS	10									20 60 17 3
	Loose Brown Moist (FILL)													
97.1			3	SS	8									
1.5	Silty SAND , trace gravel Loose to Very Loose Brown Moist to Wet													
			4	SS	3									0 69 29 2
95.2	Very Dense		5	SS	100/									
					0.125									
3.4	Weathered, massive, slightly foliated, quartz seams, dark grey: (GRANODIORITE BEDROCK) Horizontal fractures at 3.4m, 3.5m and 3.6m		1	RUN										RUN #1 TCR=69% SCR=61% RQD=22%
	Sub-horizontal fractures at 3.5m, 3.7m, 37.8m and 3.9m													
	Sub-horizontal fractures at 4.3m, 4.4m, 4.6m, 4.8m, 4.9m, 5.1m and 5.3m Vertical fractures from 4.6 to 4.9m and 5.1 to 5.2m		2	RUN										RUN #2 TCR=79% SCR=28% RQD=9%
	Rubble zone from 5.3 to 5.4m													
	Rubble zone from 5.6 to 5.7m													
	Sub-horizontal fractures at 5.7m, 5.9m, 6.1m, 6.3m, 6.4m and 6.5m													
	Horizontal fractures at 5.8m and 6.9m		3	RUN										RUN #3 TCR=92% SCR=27% RQD=0%
	Vertical fractures from 5.7 to 5.8m and 6.1m to 6.3m													
91.5	WATER LEVEL AT 1.1m.													
7.1	END OF BOREHOLE AT 7.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 1.8m, THEN SAND AND GRAVEL TO SURFACE.													

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

ONTMT4S2 MTO-28671.GPJ 2017TEMPLATE(MTO).GDT 8/19/20

RECORD OF BOREHOLE No 20-03

1 OF 1

METRIC

W.P. _____ LOCATION MTM Zone 14 N 5 514 394.2 E 249 526.3 ORIGINATED BY GS
 DIST _____ HWY 801 BOREHOLE TYPE Soild Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2020.07.11 - 2020.07.11 LATITUDE _____ LONGITUDE _____ CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
98.5	GROUND SURFACE													
0.0	Sandy GRAVEL , some cobbles													
98.2	Grey		1	GS										
0.3	(FILL)													
97.7	Gravelly SAND , some silt													
0.8	Brown		2	SS	8									
	Moist													
	(FILL)													
	Silty SAND , trace clay		3	SS	16									
	Loose to Compact													
	Brown		4	SS	4									
	Moist													
			5	SS	3									
94.6	Very Dense		6	SS	100/									
3.9	END OF BOREHOLE AT 3.9m UPON REFUSAL AT ASSUMED BEDROCK. BOREHOLE CAVED TO 2.6m WITH NO WATER LEVEL. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.				0.050									

RECORD OF BOREHOLE No 20-04

1 OF 1

METRIC

W.P. _____ LOCATION MTM Zone 14 N 5 514 392.8 E 249 521.6 ORIGINATED BY GS
 DIST _____ HWY 801 BOREHOLE TYPE Soild Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2020.07.11 - 2020.07.11 LATITUDE _____ LONGITUDE _____ CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
98.5	GROUND SURFACE													
0.0	Sandy GRAVEL , some silt, some cobbles Grey Moist (FILL) Gravelly SAND Brown Moist (FILL) Silty SAND , trace clay Compact to Loose Brown Moist Very Dense		1	GS										0 69 29 2
98.2														
0.3														
97.8														
0.7														
			2	SS	12									
			3	SS	6									
			4	SS	7									
95.1			5	SS	100/ 0.050									
3.4	END OF BOREHOLE AT 3.4m ON ASSUMED BEDROCK WATER LEVEL AT 3.3m IN BOREHOLE. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.													

RECORD OF BOREHOLE No 20-05

1 OF 1

METRIC

W.P. _____ LOCATION MTM Zone 14 N 5 514 396.0 E 249 523.8 ORIGINATED BY GS
 DIST _____ HWY 801 BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2020.07.11 - 2020.07.11 LATITUDE _____ LONGITUDE _____ CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE			WATER CONTENT (%) w _p w w _L											
98.5	GROUND SURFACE						20	40	60	80	100		20	40	60	GR	SA	SI	CL			
0.0	Sandy GRAVEL , some silt, some cobbles		1	GS		▽							○									
98.2	Grey Moist (FILL)																					
0.3																						
97.7																						
0.8	Gravelly SAND , some silt		2	SS	7									○					0	70	28	2
	Brown Moist (FILL)																					
	Silty SAND , trace clay, trace gravel Loose to Very Loose		3	SS	7								○									
	Brown Moist																					
			4	SS	2								○									
			5	SS	4								○					5	68	26	1	
94.6			6	SS	100/								○									
3.9	Weathered, unbedded, slightly foliated, dark grey: (BEDROCK) Rubble zone from 3.9 to 4.0m Sub-horizontal fracture at 4.3m		1	RUN	0.100																	
	Horizontal fractures at 4.0m, 4.2m and 4.4m Sub-horizontal fractures at 4.5m, 4.8m, 4.9m, 5.4m, 5.6m, 5.9m and 6.0m Horizontal fractrues at 4.6m and 4.7m		2	RUN																		
	Highly fractured zones from 4.7 to 4.8m and 5.2 to 5.3m Sub-vertical fractures from 5.0 to 5.2m and 5.3 to 5.6 m																					
	Horizontal fractrues at 6.5m and 7.3m Sub-vertical fractrue from 7.0 to 7.2m		3	RUN																		
91.2																						
7.3	END OF BOREHOLE AT 7.3m. BOREHOLE OPEN AND WATER LEVEL AT 1.1m BELOW GROUND SURFACE. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 3.0m THEN SAND AND GRAVEL TO SURFACE.																					

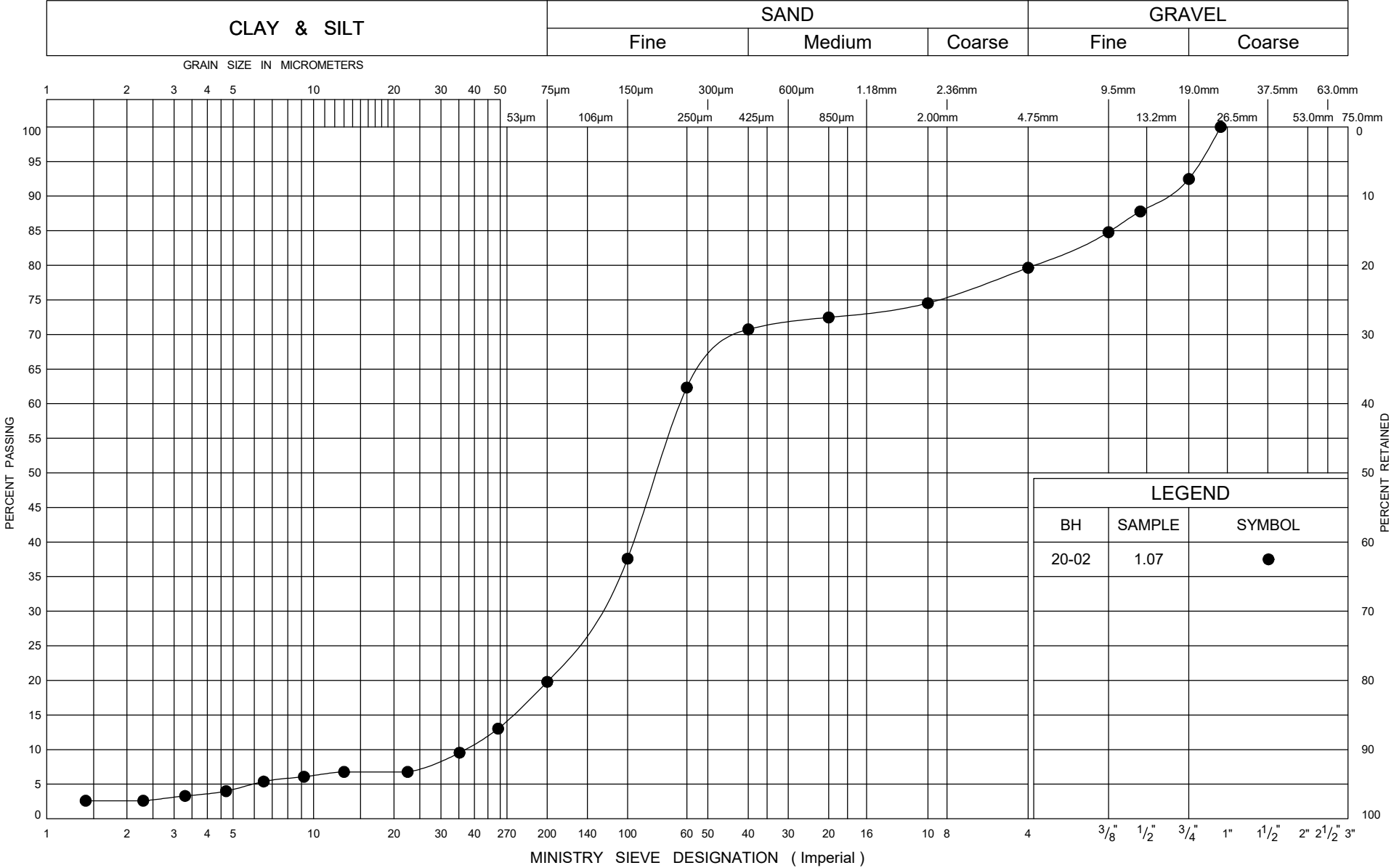
ONTMT4S2 MTO-28671.GPJ 2017TEMPLATE(MTO).GDT 8/19/20



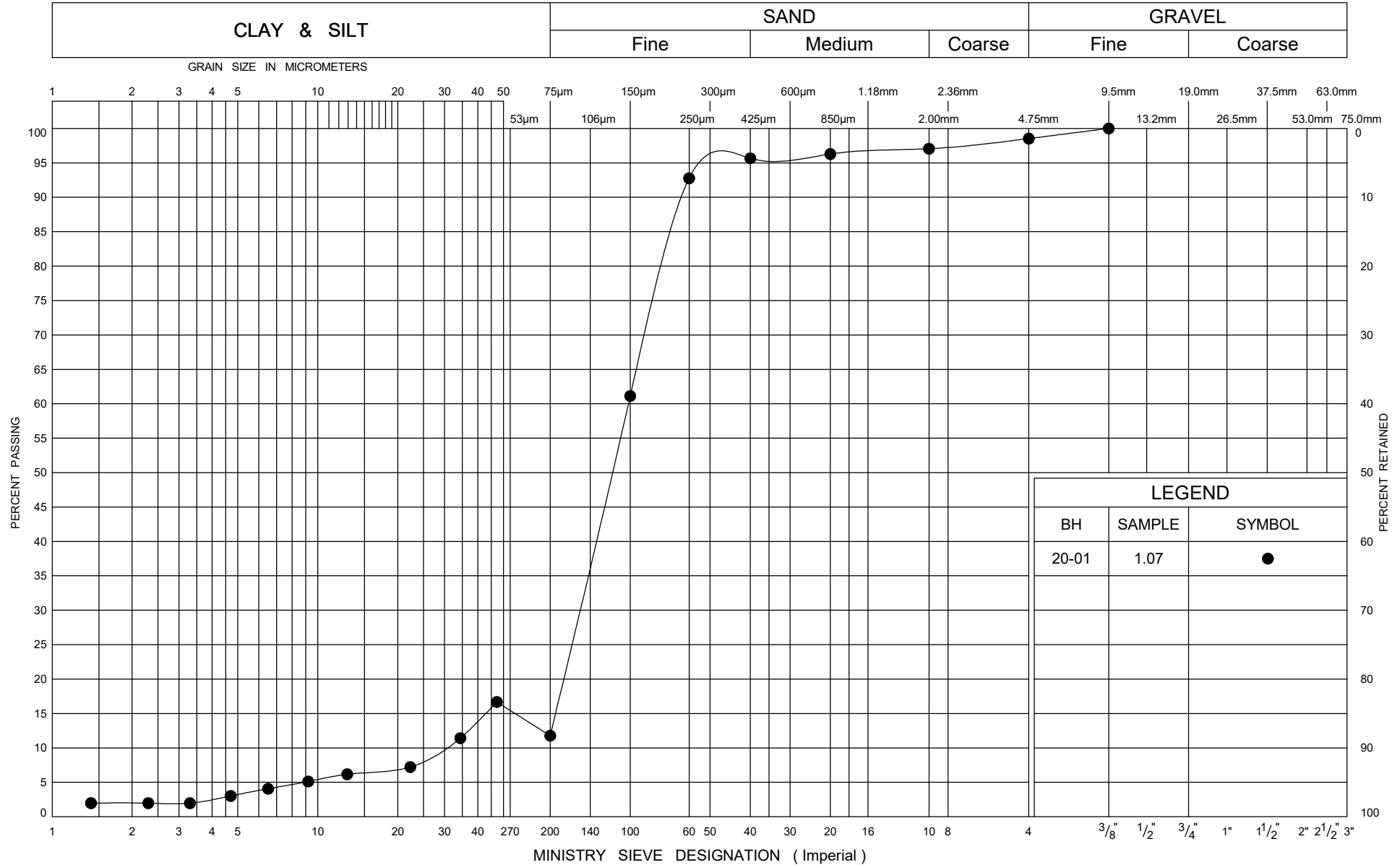
Appendix B

Geotechnical and Analytical Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

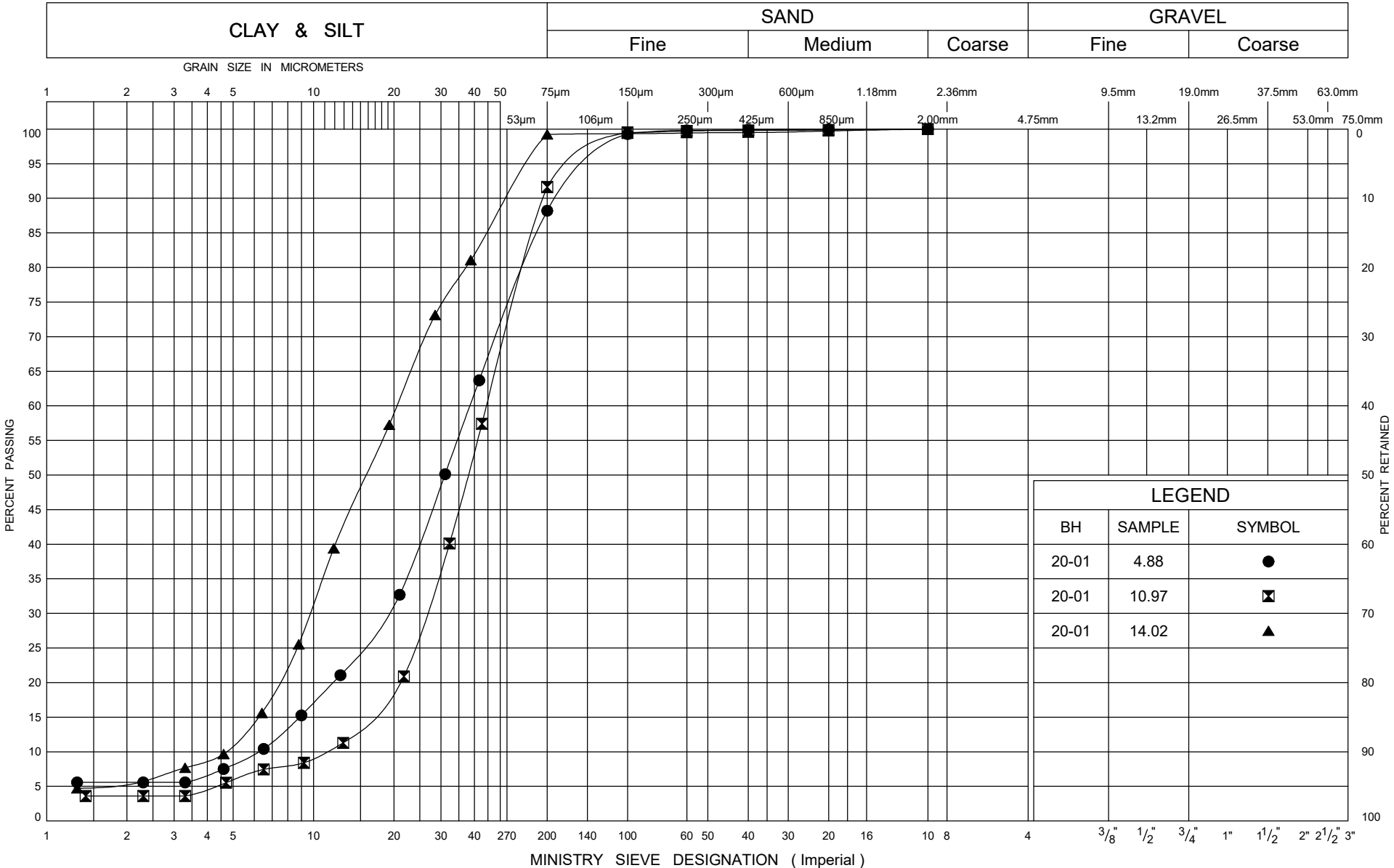
Sand Fill

FIG No B2

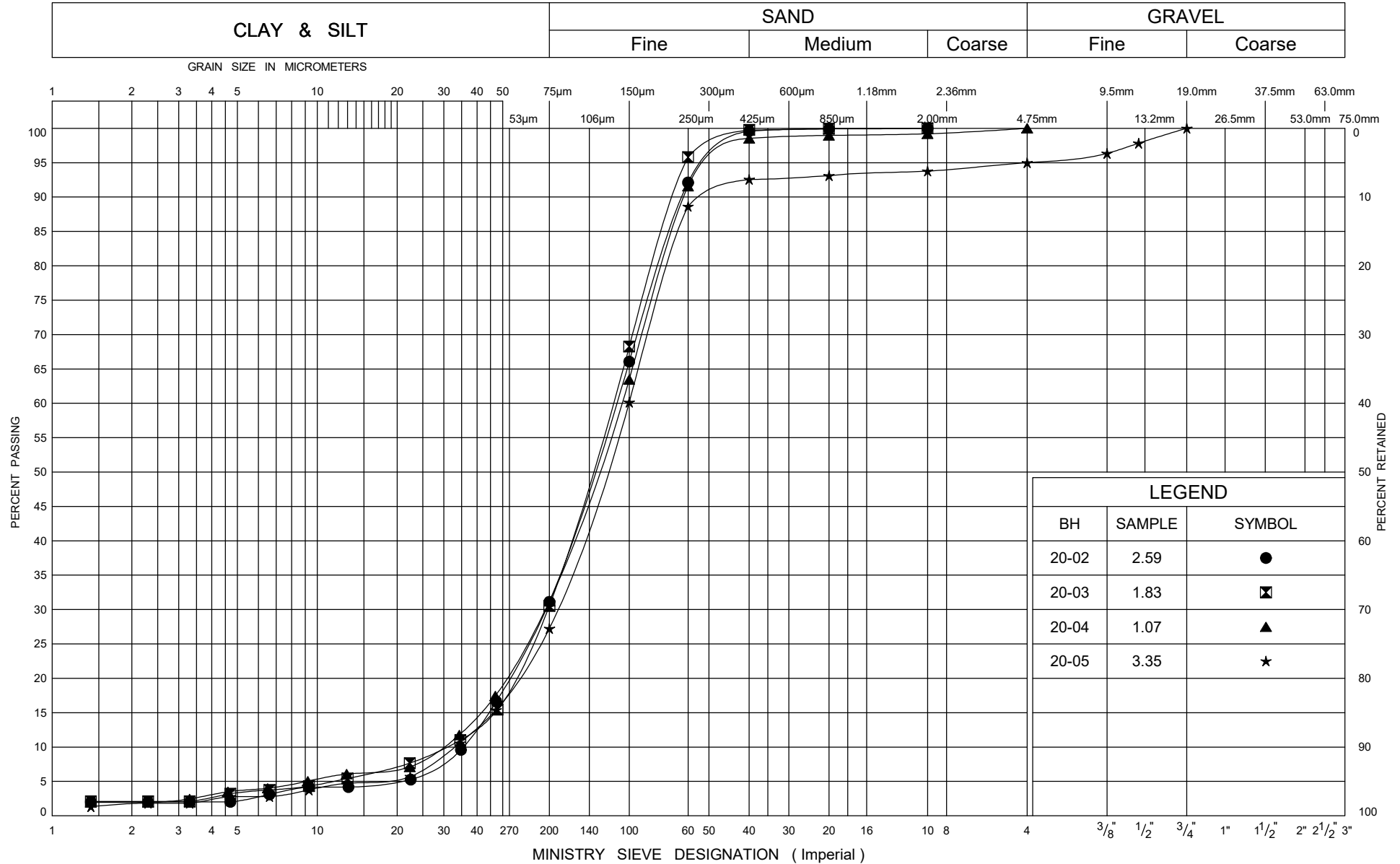
W P

NAMAWAMINKEN RIVER BRIDGE

UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

Silty Sand

FIG No B4

W P



Ministry of
Transportation



Your Project #: 28671
Site Location: NAMEWAMINIKAN RIVER
Your C.O.C. #: na

Attention: Cory Zanatta

Thurber Engineering Ltd
2010 Winston Park Dr
Suite 103
Oakville, ON
CANADA L6H 5R7

Report Date: 2020/08/19
Report #: R6297815
Version: 2 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C0K4424

Received: 2020/08/12, 12:35

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	2	2020/08/17	2020/08/18	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	2	2020/08/14	2020/08/14	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2020/08/17	2020/08/17	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2020/08/12	2020/08/14	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	2	2020/08/17	2020/08/17	CAM SOP-00464	EPA 375.4 m
Redox Potential (1, 2)	2	N/A	N/A		

Remarks:

Bureau Veritas Laboratories are accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by BV Labs are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in BV Labs profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and BV Labs in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

BV Labs liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. BV Labs has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by BV Labs, unless otherwise agreed in writing. BV Labs is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by BV Labs, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

(1) This test was performed by Sub from Campo to Env. Testing Canada (Eurofins)

(2) Oxidation-Reduction Potential (ORP) values are determined using a Ag/AgCl reference electrode.



Your Project #: 28671
Site Location: NAMEWAMINIKAN RIVER
Your C.O.C. #: na

Attention: Cory Zanatta

Thurber Engineering Ltd
2010 Winston Park Dr
Suite 103
Oakville, ON
CANADA L6H 5R7

Report Date: 2020/08/19
Report #: R6297815
Version: 2 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C0K4424

Received: 2020/08/12, 12:35

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Antonella Brasil, Senior Project Manager
Email: Antonella.Brasil@bvlabs.com
Phone# (905)817-5817

=====

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BUREAU
VERITAS

BV Labs Job #: COK4424
Report Date: 2020/08/19

Thurber Engineering Ltd
Client Project #: 28671
Site Location: NAMEWAMINIKAN RIVER
Sampler Initials: GS

SOIL CORROSIVITY PACKAGE (SOIL)

BV Labs ID		NIS724		NIS725		
Sampling Date		2020/07/09		2020/07/11		
COC Number		na		na		
	UNITS	20-01 SS2	QC Batch	20-03 SS2	RDL	QC Batch
Calculated Parameters						
Resistivity	ohm-cm	16000	6885392	12000	N/A	6885392
Inorganics						
Soluble (20:1) Chloride (Cl-)	ug/g	<20	6893103	<20	20	6893103
Conductivity	umho/cm	64	6889826	85	2	6889826
Available (CaCl2) pH	pH	8.06	6892777	7.63	N/A	6892784
Soluble (20:1) Sulphate (SO4)	ug/g	<20	6893108	<20	20	6893108
RDL = Reportable Detection Limit QC Batch = Quality Control Batch N/A = Not Applicable						



BUREAU
VERITAS

BV Labs Job #: COK4424
Report Date: 2020/08/19

Thurber Engineering Ltd
Client Project #: 28671
Site Location: NAMEWAMINIKAN RIVER
Sampler Initials: GS

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	21.3°C
-----------	--------

Results relate only to the items tested.



QUALITY ASSURANCE REPORT

QA/QC Batch	Init	QC Type	Parameter	Date Analyzed	Value	Recovery	UNITS	QC Limits
6889826	NYS	Spiked Blank	Conductivity	2020/08/14		104	%	90 - 110
6889826	NYS	Method Blank	Conductivity	2020/08/14	<2		umho/cm	
6889826	NYS	RPD	Conductivity	2020/08/14	0.48		%	10
6892777	SAU	Spiked Blank	Available (CaCl ₂) pH	2020/08/17		100	%	97 - 103
6892777	SAU	RPD	Available (CaCl ₂) pH	2020/08/17	0.90		%	N/A
6892784	NYS	Spiked Blank	Available (CaCl ₂) pH	2020/08/17		100	%	97 - 103
6892784	NYS	RPD	Available (CaCl ₂) pH	2020/08/17	0.079		%	N/A
6893103	KAD	Matrix Spike	Soluble (20:1) Chloride (Cl ⁻)	2020/08/18		NC	%	70 - 130
6893103	KAD	Spiked Blank	Soluble (20:1) Chloride (Cl ⁻)	2020/08/18		100	%	70 - 130
6893103	KAD	Method Blank	Soluble (20:1) Chloride (Cl ⁻)	2020/08/18	<20		ug/g	
6893103	KAD	RPD	Soluble (20:1) Chloride (Cl ⁻)	2020/08/18	1.9		%	35
6893108	KAD	Matrix Spike	Soluble (20:1) Sulphate (SO ₄)	2020/08/17		NC	%	70 - 130
6893108	KAD	Spiked Blank	Soluble (20:1) Sulphate (SO ₄)	2020/08/17		103	%	70 - 130
6893108	KAD	Method Blank	Soluble (20:1) Sulphate (SO ₄)	2020/08/17	<20		ug/g	
6893108	KAD	RPD	Soluble (20:1) Sulphate (SO ₄)	2020/08/17	14		%	35

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)



BUREAU
VERITAS

BV Labs Job #: COK4424
Report Date: 2020/08/19

Thurber Engineering Ltd
Client Project #: 28671
Site Location: NAMEWAMINIKAN RIVER
Sampler Initials: GS

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Antonella Brasil, Senior Project Manager

Anastassia Hamanov, Scientific Specialist

Brad Newman, Scientific Service Specialist

BV Labs has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation please refer to the Validation Signature Page.



6740 Campobello Road, Mississauga, Ontario L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266
CAM FCD-01191/6

CHAIN OF CUSTODY RECORD

Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name:	Thurber Engineering	Company Name:		Quotation #:		<input type="checkbox"/> Regular TAT (5-7 days) Most analyses	
Contact Name:	Cory Zanatta	Contact Name:		P.O. #/ AFE#:		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS	
Address:	2010 Winston Park Drive, Suite 103	Address:		Project #:	28671	Rush TAT (Surcharges will be applied)	
	Oakville, Ontario, L6H 5R7			Site Location:	Nomeawaminiken River	<input type="checkbox"/> 1 Day	<input type="checkbox"/> 2 Days <input checked="" type="checkbox"/> 3-4 Days
Phone:	905-829-8666 Fax:	Phone:		Site #:		Date Required: 17-Aug-20	
Email:	czanatta@thurber.ca	Email:		Site Location Province:	Ontario	Rush Confirmation #: ABR081102	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE BUREAU VERITAS LABORATORIES' DRINKING WATER CHAIN OF CUSTODY				Sampled By: Greg Stanhope			
Regulation 153		Other Regulations		Analysis Requested		LABORATORY USE ONLY	
<input checked="" type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/ Fine		<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw		# OF CONTAINERS SUBMITTED FIELD FILTERED (CIRCLE) Metals / Hg / Cu / Pb BTEX / PHC F1 PHCs F2 - F4 VOCs REG 153 METALS & INORGANICS REG 153 ICPMS METALS REG 153 METALS (Hg, Cr-VI, ICPMS Metals, HWS - B) Corrosivity Analysis		CUSTODY SEAL Y / N	
<input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Lomm <input type="checkbox"/> Coarse		<input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw				Present Intact	
<input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other		<input type="checkbox"/> PWWQ <input type="checkbox"/> Region				22/21/21	
<input type="checkbox"/> Table _____		<input type="checkbox"/> Other (Specify) _____					
FOR RSC (PLEASE CIRCLE) Y / N		<input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)					
<input type="checkbox"/> REG 406 Table _____		<input type="checkbox"/> REG 406 Table _____				COOLING MEDIA PRESENT: Y / <input checked="" type="checkbox"/> N	
Include Criteria on Certificate of Analysis: Y / N						COMMENTS	
SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS							
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX			
1	20-01 SS 2	2020-07-09		Soil		X	
2	20-03 SS 2	2020-07-11		Soil		X	
3							
4							
5							
6							
7							
8							
9							
10							
RELINQUISHED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	BV JOB #
				DIPKA SINGH	2020/08/12	12:35	

12-Aug-20 12:35
Antonella Brasil
C0K4424
3K1 ENV-1100



Your Project #: 28671
Your C.O.C. #: 777879-01-01

Attention: Mark Farrant

Thurber Engineering Ltd
2010 Winston Park Dr
Suite 103
Oakville, ON
CANADA L6H 5R7

Report Date: 2020/07/23
Report #: R6258600
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C0H5443

Received: 2020/07/14, 09:26

Sample Matrix: Water
Samples Received: 1

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride by Automated Colourimetry	1	N/A	2020/07/16	CAM SOP-00463	SM 23 4500-Cl E m
Conductivity	1	N/A	2020/07/17	CAM SOP-00414	SM 23 2510 m
pH	1	2020/07/16	2020/07/17	CAM SOP-00413	SM 4500H+ B m
Resistivity of Water	1	2020/07/15	2020/07/17	CAM SOP-00414	SM 23 2510 m
Sulphate by Automated Colourimetry	1	N/A	2020/07/17	CAM SOP-00464	EPA 375.4 m
Sulphide	1	N/A	2020/07/16	CAM SOP-00455	SM 23 4500-S G m

Remarks:

Bureau Veritas Laboratories are accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by BV Labs are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in BV Labs profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and BV Labs in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

BV Labs liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. BV Labs has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by BV Labs, unless otherwise agreed in writing. BV Labs is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by BV Labs, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.



Your Project #: 28671
Your C.O.C. #: 777879-01-01

Attention: Mark Farrant

Thurber Engineering Ltd
2010 Winston Park Dr
Suite 103
Oakville, ON
CANADA L6H 5R7

Report Date: 2020/07/23
Report #: R6258600
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C0H5443

Received: 2020/07/14, 09:26

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Antonella Brasil, Senior Project Manager

Email: Antonella.Brasil@bvlabs.com

Phone# (905)817-5817

=====

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RESULTS OF ANALYSES OF WATER

BV Labs ID		NCL545		
Sampling Date		2020/07/12 11:30		
COC Number		777879-01-01		
	UNITS	NAMA RIVER	RDL	QC Batch
Calculated Parameters				
Resistivity	ohm-cm	7800	N/A	6837530
Inorganics				
Conductivity	umho/cm	130	1.0	6839339
pH	pH	7.82	N/A	6839343
Dissolved Sulphate (SO ₄)	mg/L	<1.0	1.0	6839776
Sulphide	mg/L	<0.020	0.020	6839253
Dissolved Chloride (Cl ⁻)	mg/L	2.3	1.0	6839770
RDL = Reportable Detection Limit QC Batch = Quality Control Batch N/A = Not Applicable				



GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	6.0°C
-----------	-------

Results relate only to the items tested.



QUALITY ASSURANCE REPORT

QA/QC Batch	Init	QC Type	Parameter	Date Analyzed	Value	Recovery	UNITS	QC Limits
6839253	NYS	Matrix Spike	Sulphide	2020/07/16		98	%	80 - 120
6839253	NYS	Spiked Blank	Sulphide	2020/07/16		106	%	80 - 120
6839253	NYS	Method Blank	Sulphide	2020/07/16	<0.020		mg/L	
6839253	NYS	RPD	Sulphide	2020/07/16	NC		%	20
6839339	SAU	Spiked Blank	Conductivity	2020/07/17		99	%	85 - 115
6839339	SAU	Method Blank	Conductivity	2020/07/17	<1.0		umho/cm	
6839339	SAU	RPD	Conductivity	2020/07/17	0.39		%	25
6839343	SAU	Spiked Blank	pH	2020/07/17		102	%	98 - 103
6839343	SAU	RPD	pH	2020/07/17	0.24		%	N/A
6839770	KAD	Matrix Spike	Dissolved Chloride (Cl-)	2020/07/16		NC	%	80 - 120
6839770	KAD	Spiked Blank	Dissolved Chloride (Cl-)	2020/07/16		103	%	80 - 120
6839770	KAD	Method Blank	Dissolved Chloride (Cl-)	2020/07/16	<1.0		mg/L	
6839770	KAD	RPD	Dissolved Chloride (Cl-)	2020/07/16	1.3		%	20
6839776	KAD	Matrix Spike	Dissolved Sulphate (SO ₄)	2020/07/17		98	%	75 - 125
6839776	KAD	Spiked Blank	Dissolved Sulphate (SO ₄)	2020/07/17		105	%	80 - 120
6839776	KAD	Method Blank	Dissolved Sulphate (SO ₄)	2020/07/17	<1.0		mg/L	
6839776	KAD	RPD	Dissolved Sulphate (SO ₄)	2020/07/17	0.96		%	20

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).



BUREAU
VERITAS

BV Labs Job #: COH5443
Report Date: 2020/07/23

Thurber Engineering Ltd
Client Project #: 28671

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

A handwritten signature in black ink, appearing to read "BRAD NEWMAN", positioned above a horizontal line.

Brad Newman, Scientific Service Specialist

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Appendix C

Site Photographs



Photo 1: Namewaminikan River Bridge Looking North East



Photo 2: Namewaminikan River Bridge Looking South



Photo 3: Namewaminikan River Bridge Looking North



Photo 4: Namewaminikan River Bridge South Abutment Looking West



Photo 4: South Abutment, existing timber cribs



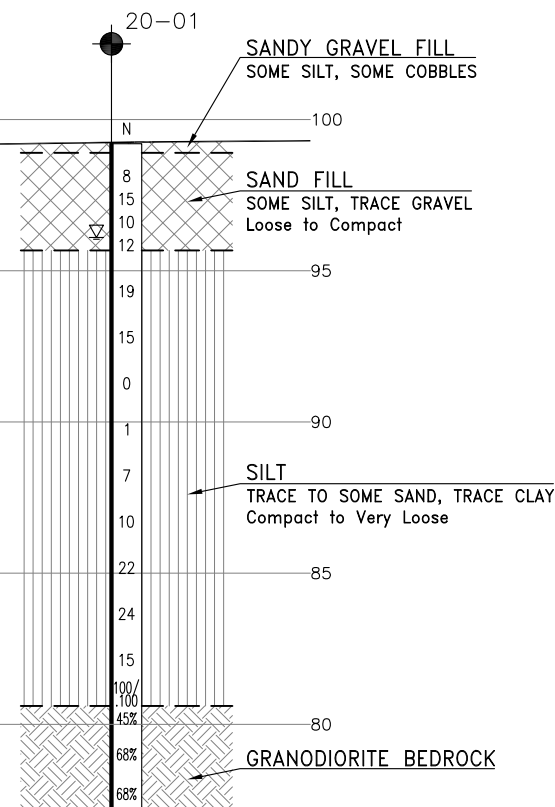
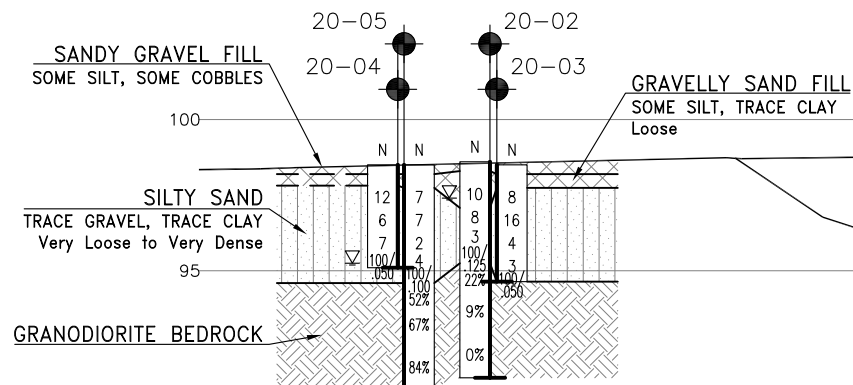
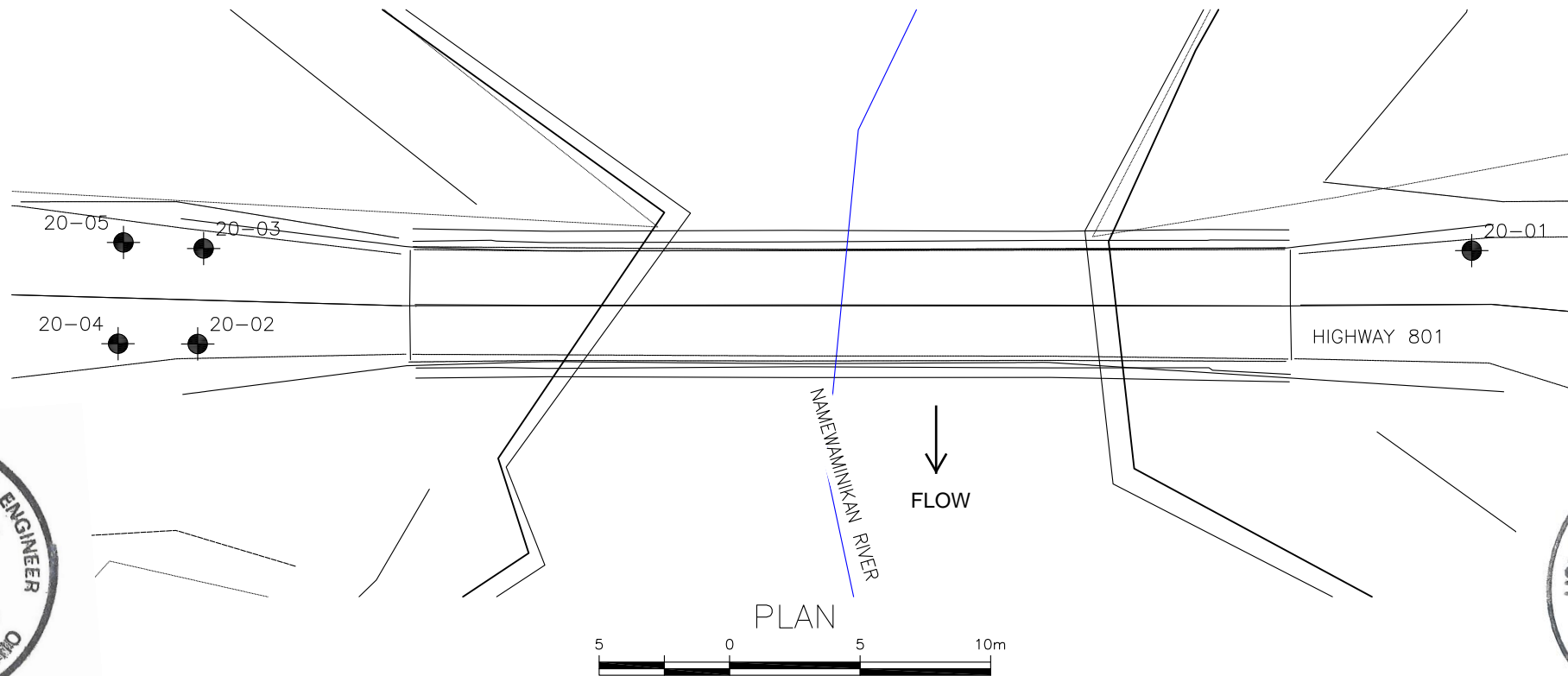
Photo 4: North abutment, forward slope and timber cribs.



Appendix D

Borehole Locations and Soil Strata Drawing

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No 2020-6051
WP No

HIGHWAY 801
NAMEWAMINIKAN RIVER
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

HATCH



KEYPLAN

LEGEND

●	Borehole
⊙	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
⊥	Head Artesian Water
⊥	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
20-01	99.2	5 514 368.8	249 567.7
20-02	98.6	5 514 391.2	249 524.2
20-03	98.5	5 514 394.2	249 526.3
20-04	98.5	5 514 392.8	249 521.6
20-05	98.5	5 514 396.0	249 523.8

-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD 83 Zone 14.

GEOCRES No. 42E-33

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	CZ	CHK PKC	CODE
DRAWN	MFA	CHK CZ	SITE 48C-0125/80
			STRUCT
			DWG 2
			DATE OCT 2020



Appendix E

Slope Stability Figures

FIGURE E1
EXISTING SOUTH SLOPE

Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
Bedrock			
Engineered Pad	22	0	35
Footing	20	3,000	32
Sand Fill	20	0	30
Sandy Gravel Fill	21	0	32
Silt	19	0	30
Silty Sand	20	0	30

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1 m
PWP Conditions from: Piezometric Line
Center: (34.023264, 116.83608) m
Radius: 25.675472 m

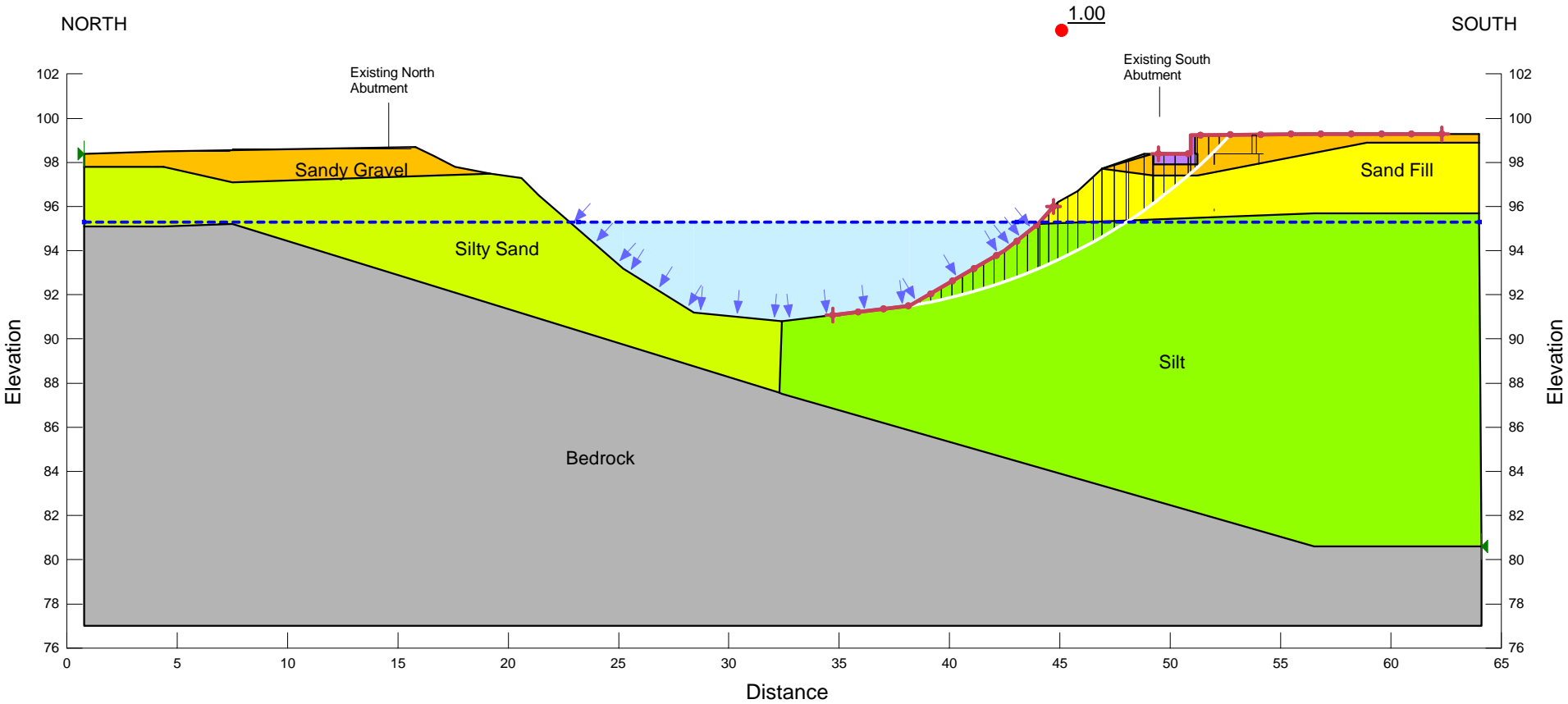
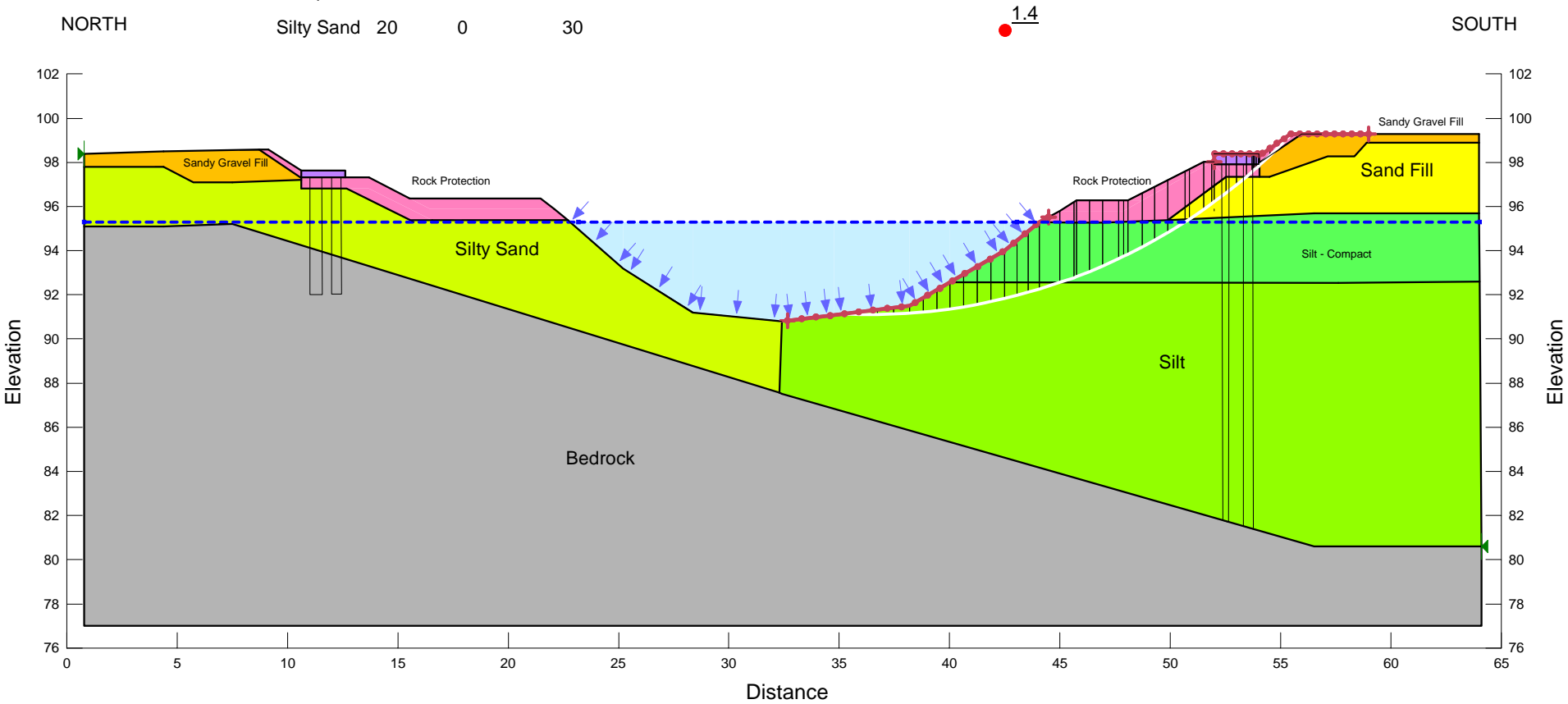


FIGURE E2 PROPOSED SOUTH SLOPE H-PILE FOUNDATION

Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
Bedrock			
Footing	1	3,000	32
Rock Protection	18	0	42
Sand Fill	19	0	31
Sandy Gravel Fill	21	0	35
Silt	19	0	30
Silt - Compact	19	0	32
Silty Sand	20	0	30

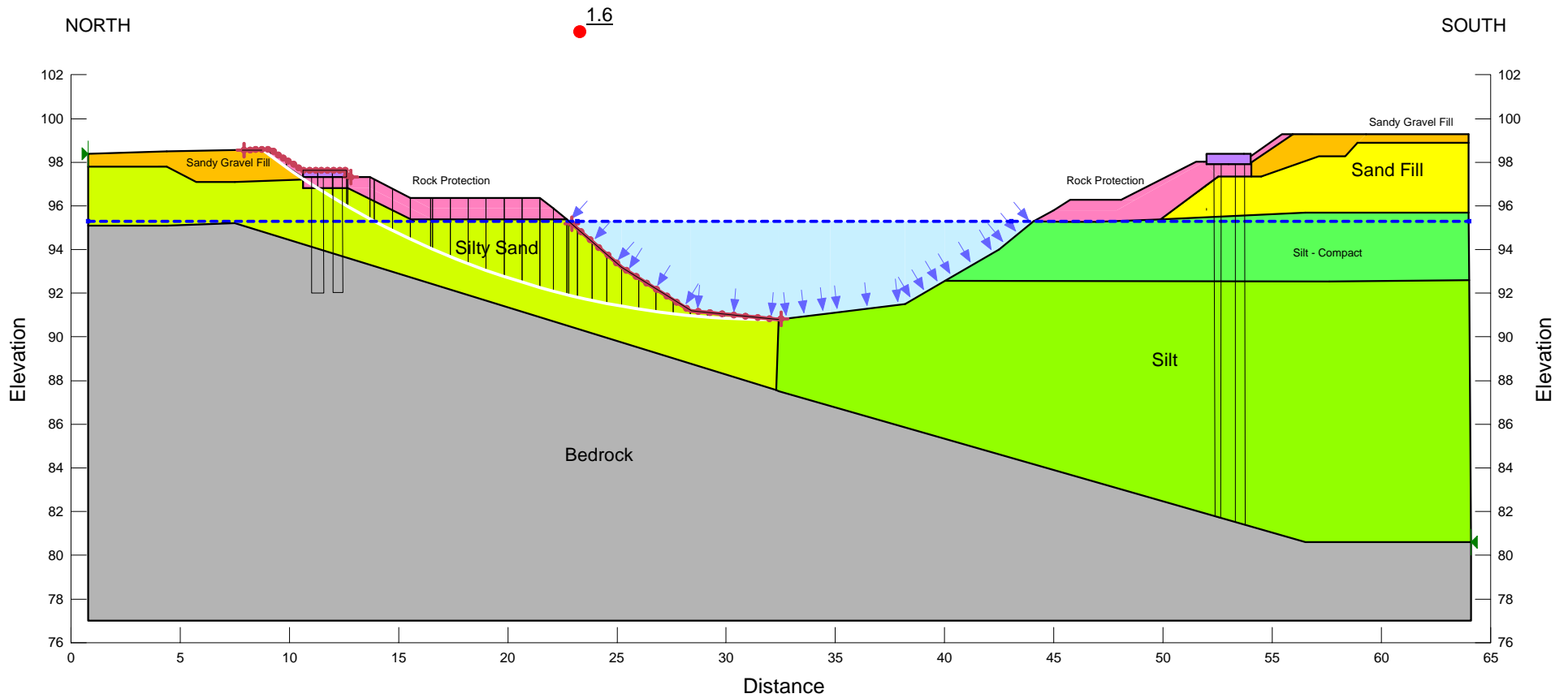
Method: Morgenstern-Price
Minimum Slip Surface Depth: 1 m
PWP Conditions from: Piezometric Line
Center: (36.405143, 117.50961) m
Radius: 26.392022 m



Name	Unit Weight (kN/m ³)	Cohesion' (kPa)	Phi' (°)
Bedrock			
Footing	1	3,000	32
Rock Protection	18	0	42
Sand Fill	19	0	31
Sandy Gravel Fill	21	0	35
Silt	19	0	30
Silt - Compact	19	0	32
Silty Sand	20	0	30

FIGURE E2
PROPOSED NORTH SLOPE H-PILE
FOUNDATION

Method: Morgenstern-Price
Minimum Slip Surface Depth: 1 m
PWP Conditions from: Piezometric Line
Center: (31.939756, 129.3275) m
Radius: 38.516858 m





Appendix F

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Spread Footings on Engineered Fill Pads	Spread Footings on Native Soils or Bedrock	Driven Steel H-Piles
<p><u>Advantages:</u></p> <ul style="list-style-type: none"> i. Conventional construction ii. Engineered fill pad can be placed on loose to compact sand fill or native silty sand depending on geotechnical resistance requirements 	<p><u>Advantages:</u></p> <ul style="list-style-type: none"> i. Conventional construction ii. Higher geotechnical capacities can be achieved over spread footings on engineered fill pads if placed on bedrock. 	<p><u>Advantages:</u></p> <ul style="list-style-type: none"> i. Conventional construction. ii. Generally less costly than other deep foundation elements. iii. Higher geotechnical resistances can be achieved versus shallow foundation options
<p><u>Disadvantages:</u></p> <ul style="list-style-type: none"> i. Engineered fill preparation must be carried out in the dry. ii. Excavation may extend below groundwater level which would require dewatering iii. Large cobbles and boulders observed on the front slopes of abutments may require excavation and special handling iv. Low slope stability F.S. against sliding through footings at 3 m setback without significant slope reinforcement. v. May have to set back footings greater than 3 m. 	<p><u>Disadvantages:</u></p> <ul style="list-style-type: none"> i. Deeper excavations required ii. Lower geotechnical capacities if footings placed sandy fill or loose silts iii. Excavation would extend below ground water level iv. Pre-existing slope stability issues at both abutments 	<p><u>Disadvantages:</u></p> <ul style="list-style-type: none"> i. Presence of cobbles and boulders in the highway embankment. ii. Presence of steeply dipping bedrock
FEASIBLE	NOT RECOMMENDED	RECOMMENDED



Appendix G

List of OPSSs and OPSDs and Suggested Wording for NSSP



1. List of OPSS and OPSD Documents Relevant to this Project

- OPSS PROV 206 (Construction Specification for Grading)
- OPSS PROV 501 (Construction Specification for Compacting)
- OPSS 511 (Construction Specification for Rip-Rap, Rock Protection, And Granular Sheeting)
- OPSS 517 (Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation)
- OPSS PROV 539 (Construction Specification for Temporary Protection Systems)
- OPSS PROV 804 (Construction Specification for Seed and Cover)
- OPSS 902 (Construction Specification for Excavating and Backfilling – Structures)
- OPSS PROV 1004 (Material Specification for Aggregates – Miscellaneous)
- OPSS PROV 1010 (Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material)
- OPSD 3090.100 (Foundation Frost Depths for Northern Ontario)
- OPSS PROV 903 (Deep Foundations)

2. Suggested Wording for NSSP

• Suggested Text for NSSP on Obstructions

Excavations at this site will encounter obstructions such as cobbles and boulders, or timbers embedded in the fill and native soils. Such obstructions may impede excavation progress. The Contractor shall be prepared to remove, these obstructions to achieve the design depths.

• Suggested Text for NSSP on Setting of Driven Piles on Sloping Bedrock

For setting of driven piles on sloping bedrock the following procedure shall be used. Upon initial contact with the bedrock:

- A. Apply 10 blows at 10% of hammer energy, record the penetration,
- B. Apply further 10 blows at 50% of hammer energy. If the pile penetration under 10 blows is less than 12.5 mm, the pile is set,



C. If the penetration under 10 blows is greater than 12.5 mm, the issue should be referred to the design team for resolution.

- **Suggested Text for NSSP on Formation of Rock Socket**

Contractor is advised that at the north abutment the H-Piles will be installed in a rock socket. The bedrock at the site is granodiorite which is strong to extremely strong. Contractors drilling equipment must be able to drill the socket in this very strong rock. Rock strengths are provided in the foundation investigation report. The drilling method for the rock sockets must not fracture or shatter the base or sidewall of the socket.

The overburden soils above the bedrock is cohesionless soils below the groundwater table. Measures must be employed to maintain sidewall stability during pile installation and prevent collapse or washing of the cohesionless soils into the rock socket. The use of temporary casing sealed into the rock is highly recommended. Selection of the methods and equipment employed to achieve this is the responsibility of the contractor.

- **Suggested Text for NSSP on Use of Heavy Construction Equipment**

The use of heavy construction equipment, in particular heavy lift cranes, may be required during removal of the existing bridge and construction / relocation of the existing bridge. The impact of heavy equipment loads on the embankments and bridge foundations must be considered during selection of the methodology and equipment employed for construction.

Prior to commencement of construction activities which include, but are not limited to, bridge lifting and existing structure removal, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and construction methodology, as well as determine requirements and/or restrictions necessary to safely support these loads without a foundation or slope failure. All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO's RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – Medium Complexity.

The assessment shall include, but not be limited to, the following:

- Determining appropriate setback distances for heavy equipment from the existing and/or new bridge abutments and their foundations. And from the crests of the river valley slopes and embankment slopes;



- Determining the permissible ground pressure that may be applied to the foundation soils by the equipment; and
- Providing recommendations for crane pad design to distribute the crane loads without causing foundation and creek bank failure.

The findings and recommendations provided by the Geotechnical Consultant shall bear the seal and signature of the Engineer licensed to practice in Ontario.

The Contractor shall submit the findings of the geotechnical assessment and details of the proposed equipment and construction methodology to the Contractor Administrator for information purposes a minimum of 7 days prior to the start of lifting operations.