



FOUNDATION INVESTIGATION AND DESIGN REPORT

**Replacement of Nagagamisis Narrows Bridge -
Site No. 38N-0001/BO**

Highway 631, Township of Frost, Ontario

MTO Contract DB 2021-5168

Submitted to:

Facca Incorporated

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PART A

**FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF NAGAGAMISIS NARROWS
BRIDGE – SITE NO. 38N-0001/BO
HIGHWAY 631, TOWNSHIP OF FROST, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
DB CONTRACT# 2021-5168**

1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd.), now a member of WSP Canada Inc., hereafter referenced as WSP Golder) has been retained by Facca Incorporated (Facca) on behalf of the Ministry of Transportation, Ontario (MTO) to provide Foundation Engineering services to support detail design for the Design Build (DB) contract for the replacement of the Nagagamisis Narrows Bridge (Site No. 38N-0001/BO). The bridge is located on Highway 631 about 50 km southwest of Hearst and about 30 km north of Hornepayne, in the Township of Frost, Ontario. The key plan showing the general location of this section of Highway 631 and the location of the bridge are shown on Drawing 1.

The scope of this report addresses the new permanent Nagagamisis Narrows bridge and temporary modular bridge to be constructed as part of the overall bridge replacement operations. WSP Golder (formerly Golder Associates Ltd.) was previously retained by LEA Consulting Ltd. (LEA) in 2017 on behalf of the MTO to carry out site-specific foundation investigation services for the previous detail design of the permanent replacement structure and temporary modular bridge. Based on the subsurface information collected at the site (including the borehole drilling, rock coring and laboratory testing program), this report provides sufficient geotechnical data and geotechnical design parameters and foundation design recommendations for the proposed bridge replacement strategy.

2.0 SITE DESCRIPTION

The surrounding land at the site is generally flat, with dense tree-covered terrain. The bridge is located within the Nagagamisis Provincial Park and specifically situated at a narrows which separates the Nagagamisis Lake into two sections. The Narrows is about 16 m wide at the existing bridge and the water flows in a westerly direction.

The existing bridge is a three-span bridge about 27 m long by 10 m wide, consisting of wood deck on steel girders that was originally constructed in 1959. The existing approach embankments are about 2 m to 3 m high relative to the lake. The existing highway grade is between approximately Elevations 289.6 m and 289.8 m. The water level in Nagagamisis Narrows was measured at the bridge site at Elevation 286.9 m in November 2016, Elevation 287.4 m in May 2017, Elevation 287.2 m in June 2017 and Elevation 286.8 m in August 2017.

During our 2017 foundation investigation discussed further in Section 3, surficial sloughing was observed at the east side of the north approach embankment. The remaining portions of the approach embankments were observed to be performing satisfactorily from a geotechnical perspective, with no visual evidence of global instability or excessive settlement, during our investigation and during a subsequent site visit by WSP Highway Engineering representatives in the Fall 2022.

Photographs at the bridge site are shown on Photographs 1 to 4, following the text of this report.

It should be noted that the orientation (i.e., north, south, east, west) stated in the text of the report is referenced to project north and therefore may differ from magnetic north shown on the drawing. Highway 631 is generally oriented in a north-south direction.

3.0 INVESTIGATION PROCEDURES

As previously mentioned, site specific investigation for detail design was carried out at the site by WSP Golder in 2017 and the results are summarized in the existing Foundation Investigation and Design Report (dated 27 June 2018, Golder Report No. 1661607 R03, Geocres No. 42F-054).

In summary, the field work was carried out between 29 May and 18 August 2017, during which time a total of eight boreholes (Boreholes NG-1 to NG-8) were advanced at the locations shown on Drawing 1. The borehole and drillhole records are presented in Appendix A. The field investigation was carried out using the following drilling equipment:

- Boreholes NG-1, NG-2, NG-4, and NG-7 were advanced using a CME-55 truck-mounted drill rig supplied and operated by Landcore Drilling Inc. (Landcore) of Sudbury, Ontario.
- Boreholes NG-3 and NG-8 were advanced using a CME-55 track-mounted drill rig supplied and operated by Downing Drilling Inc. (Downing) of Grenville-sur-la-Rouge, Quebec.
- Boreholes NG-5 and NG-6 were advanced using a Boart Longyear LF-70 DD skid-mounted drill supplied and operated by Downing.

The boreholes were advanced using solid stem augers, 108 mm inner diameter hollow stem augers and/or NW casing and wash boring. Where coring through cobbles, boulders or bedrock was required, an NQ-size core barrel was used, with water obtained from the river. Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using 50 mm outer diameter split-spoon samplers driven by an automatic hammer, in general accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586).

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and a standpipe piezometer was installed in Borehole NG-1 to permit monitoring of the groundwater level. The piezometer consisted of a 50 mm diameter polyvinyl chloride (PVC) pipe, with a slotted screen, sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled with bentonite pellets to create a seal and cuttings were placed to the pavement structure. The piezometer installation details and water level readings are indicated on the borehole records contained in Appendix A. The piezometer was abandoned in accordance with Ontario Regulation 903 (as amended) on 19 September 2017.

The field work was supervised on a full-time basis by a member of WSP Golder's staff, who located the boreholes in the field, arranged for the clearance of underground services, directed the drilling and sampling operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to WSP Golder's Sudbury Laboratory for further examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Index and classification tests consisting of water content, Atterberg limits and grain size distribution were carried out on selected soil samples and uniaxial compressive strength (UCS) tests were carried out on selected bedrock core samples. The results of the laboratory testing on samples from the boreholes are presented on the borehole and drillhole records in Appendix A, and on figures in Appendix B.

Soil samples were obtained on 29 May 2017, from Boreholes NG-7 and NG-8, using appropriate sampling protocols and submitted to a specialist analytical laboratory under chain of custody procedures for testing for a suite of parameters including pH, resistivity, conductivity, sulphates and chlorides. The results of the analytical testing are presented in Table B1 in Appendix B.

Classification of the rock mass quality of the bedrock with respect to the Rock Quality Designation (RQD) and UCS are described based on Table 3.10 and Table 3.5, respectively, of the *Canadian Foundation Engineering Manual* (CFEM, 2006¹). The degree of weathering of the bedrock samples (i.e., fresh to slightly weathered) and the strength classification of the intact rock mass based on field identification (i.e., strong to very strong) are described in accordance with Table B.3 and Table B.6, respectively, of the International Society for Rock Mechanics (ISRM²) standard classification system.

The borehole locations and elevations were measured in the field by WSP Golder personnel, relative to existing site features and surveyed to a control point installed by LEA's surveyors. The borehole locations (referenced to the MTM NAD83 co-ordinate system), ground surface elevations (referenced to Geodetic datum) and borehole depths are presented on the borehole records in Appendix A and are summarized below.

Borehole	Location (MTM NAD 83, Zone13)		Location (World Geodetic System 84)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting	Latitude	Longitude		
NG-1	5482064.5	252206.8	49.474146	-84.725752	289.8	9.6
NG-2	5482075.2	252218.0	49.474243	-84.725599	289.7	14.8
NG-3	5482101.1	252243.1	49.474478	-84.725256	289.6	14.5
NG-4	5482112.6	252253.5	49.474583	-84.725114	289.6	9.8
NG-5	5482064.7	252222.7	49.474149	-84.725532	287.8	13.8
NG-6	5482094.4	252252.8	49.474419	-84.725121	286.9	11.5
NG-7	5482078.4	252214.2	49.474272	-84.725651	289.8	14.6
NG-8	5482104.2	252239.8	49.474506	-84.725302	289.6	14.5

¹ Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

² International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

4.0 SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain (NOEGTS)³ mapping, the Nagagamis Narrows Bridge site is located within a kame field/terrace/moraine deposit consisting primarily of sand and gravels.

Based on geological mapping by the Ontario Ministry of Northern Development and Mines (MNDM)⁴, the site is underlain by bedrock from the metasedimentary suite of rocks comprised of wacke, arkose, argillite, slate, marble, chert, iron formation and minor metavolcanic rock and bordered by muscovite-bearing granitic rocks comprising muscovite-biotite and cordierite-biotite granites and granodiorite-tonalite.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced in the vicinity of the Nagagamis Narrows bridge replacement, with the results of the laboratory tests carried out on selected soil and bedrock samples, are presented on the borehole records in Appendix A, and the laboratory test sheets in Appendix B. The results of the in situ field tests (i.e., SPT 'N' values) as presented on the borehole records and in Section 4 are uncorrected. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile and cross-sections on Drawings 1 and 2 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil and bedrock conditions will vary between and beyond the borehole locations. Descriptions of the subsurface conditions encountered in the boreholes are provided in the following sub-sections of this report.

Groundwater levels/conditions encountered in the boreholes during and shortly after drilling may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized. Groundwater and lake levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

4.2.1 Subsoil Conditions

A description of the soil deposits encountered in the boreholes is provided below.

Deposit/Layer Description	Boreholes	Deposit Surface Elevation (m)	Deposit Thickness (m)	N Values (blows)	Laboratory Testing
				Relative Density	
Asphalt	NG-1 to NG-4, NG-7 & NG-8	289.6 – 289.8	0.025 – 0.1	n/a	n/a
Reclaimed Asphalt Pavement (RAP)	NG-1 to NG-4	289.56 – 289.76	0.05 – 0.11	n/a	n/a
Gravelly Sand to Sand (Fill)	NG-1 to NG-4, NG-7 & NG-8 (containing additional asphalt/RAP layers in NG-3)	289.2 – 289.6	2.0 – 4.5	N = 2 – 68	w = 5% – 26% 5 – M (Fig. B1)
				Very Loose to Very Dense	

³ Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 42FNE.

⁴ Ontario Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey – Map 2543.

Deposit/Layer Description	Boreholes	Deposit Surface Elevation (m)	Deposit Thickness (m)	N Values (blows)	Laboratory Testing
				Relative Density	
Sandy Peat or Organic Silty Sand	NG-1, NG-3, NG-8	285.1 – 287.6	0.2 – 1.1	N = 2 Very Loose	w = 72% and 112% Oc = 3.7% and 13.5% 1 – MH (Fig. B2)
Sand ¹ (Silt and Sand in Borehole NG-8)	NG-1, NG-2, NG-4 to NG-8	284.0 – 287.8	0.9 – 5.0	N = 1 – 49 Very Loose to Dense	w = 8% – 50% 11 – M (Fig. B3) 1 – MH (Fig. B4) 1 – NP
Sandy Clayey Silt	NG-7	282.6	0.6	n/a	w = 41% 1 – AL (Fig. B5) 1 – MH (Fig. B6) w _l = 27% w _p = 19% I _p = 8%
TILL ² (Silty Sand or Sand and Gravel)	NG-1 to NG-8	281.7 – 284.7	> 1.5 – 6.2	N = 18 – 118 Compact to Very Dense	w = 6% – 18% 9 – MH (Fig. B7)

Where:

N = SPT 'N'-value; number of blows for 0.3 m of penetration

w = natural moisture content (%)

M = sieve analysis for particle size

MH = combined sieve and hydrometer analysis

AL = Atterberg Limit Tests

NP = non-plastic test result in Atterberg limits

w_p = plastic limit (%)

w_l = liquid limit (%)

I_p = plasticity index (%)

Oc = organic content test

Notes:

1) A 0.6 m sand and gravel layer was encountered at 3.8 m depth in NG-4.

2) Cobbles and boulders ranging from 75 mm to 230 mm were encountered in the till deposit in all of the boreholes.

4.2.2 Bedrock/Refusal

Bedrock was cored in Boreholes NG-2, NG-3, and NG-5 to NG-8 and the depth/elevation of the bedrock surface is presented below.

Borehole No.	Location	Depth to Bedrock (m)	Bedrock Surface Refusal Elevation (m)	Bedrock Coring (m)
NG-2	South Abutment	11.6	278.1	3.2 m
NG-3	North Abutment	11.1	278.5	3.4 m
NG-5	South Abutment (Detour)	10.4	277.4	3.4 m
NG-6	North Abutment (Detour)	8.4	278.5	3.1 m
NG-7	South Abutment	11.6	278.2	3.0 m
NG-8	North Abutment	11.0	278.6	3.5 m

The retrieved bedrock cores from the boreholes are described as fresh, fine to coarse-grained, grey to black/pink granite. More detailed descriptions of the bedrock cores are presented on the drillhole records in Appendix A, including data regarding the discontinuity frequency and type. Photographs of the bedrock core samples are shown on Figure B8 in Appendix B. The bedrock properties, as encountered in the cored boreholes and/or tested on selected samples, are summarized below. The UCS laboratory test sheet is presented in Table B2 in Appendix B.

Borehole No.	Total Core Recovery (TCR)	Rock Quality Designation (RQD)	Quality Classification (Table 3.10 of CFEM 2006 ⁵)	UCS (MPa)	Strength Classification (Table 3.5 of CFEM 2006)
NG-2	100%	100%	Excellent	146	(R5) Very Strong
NG-3	100%	100%	Excellent	128	(R5) Very Strong
NG-5	100%	90% - 100%	Excellent	94	(R4) Strong
NG-6	100%	95% - 100%	Excellent	87	(R4) Strong
NG-7	100%	100%	Excellent	180	(R5) Very Strong
NG-8	100%	100%	Excellent	118	(R5) Very Strong

4.3 Groundwater Conditions

The following table summarizes the unstabilized groundwater levels measured in the open boreholes upon completion of drilling, and groundwater levels measured in the piezometer on 12 June 2017. Water levels should be expected to vary depending on the time of year and precipitation events.

Borehole No.	Ground Surface Elevation (m)	Depth to Groundwater Level (m)	Approximate Groundwater Elevation (m)
NG-1 (In piezometer)	289.9	2.5	287.3
NG-2	289.7	2.7	287.0
NG-3	289.6	2.4	287.2
NG-4	289.6	3.0	286.6
NG-5	287.8	0.6	287.2
NG-6	286.9	0.0	286.9
NG-7	289.8	2.7	287.1
NG-8	289.6	2.1	287.5

The lake water level was surveyed by others at Elevation 286.9 m in November 2016 and by WSP Golder at Elevation 287.4 m in May 2017, at Elevation 287.2 m in June 2017 and at Elevation 286.8 m in August 2017.

⁵ Canadian Geological Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. André Bom, P.Eng. Mr. Kevin Bentley, P.Eng., a Designated MTO Foundations Contact for WSP Golder, conducted an independent quality control review of this report.

Signature Page



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[https://golderassociates.sharepoint.com/sites/163211/project files/6 deliverables/ph 1000-fdns and pvmts/02-nagagamis/final/22525553-r01-r-rev0-1000-nagagamis replacement fidr 20apr_23.docx](https://golderassociates.sharepoint.com/sites/163211/project%20files/6%20deliverables/ph%201000-fdns%20and%20pvmnts/02-nagagamis/22525553-r01-r-rev0-1000-nagagamis%20replacement%20fidr%20apr_23.docx)

PART B

**DETAIL FOUNDATION DESIGN REPORT
REPLACEMENT OF NAGAGAMISIS NARROWS
BRIDGE – SITE NO. 38N-0001/BO
HIGHWAY 631, TOWNSHIP OF FROST, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
DB CONTRACT # 2021-5168**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering recommendations for the proposed replacement of the Nagagamisis Narrows Bridge. The recommendations presented are based on interpretation of the factual data obtained from the boreholes advanced during the detail design subsurface investigation.

The discussion and recommendations presented are intended to provide the Design-Builder with sufficient information to complete the detail design of the proposed permanent bridge structure foundations and associated approach embankments, and provide adequate geotechnical input / parameters for design of temporary structures. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required for construction.

6.1 General

Based on the available information, the existing bridge consists of a three-span structure about 28 m in total length and supported by 4 sets of timber cribs (one at each abutment and two along the length of the bridge within the narrows).

According to the General Arrangement (GA) drawing (dated March 2023), the existing bridge will be replaced with a new 32 m long, single-span bridge on the existing alignment. The proposed grade at the new structure will be about 500 mm and 300 mm higher than the existing grade at the north and south abutments, respectively.

In order to construct the new bridge along the same alignment and allow for reduced impact to traffic on Hwy 631, a temporary detour bridge is proposed to be constructed about 13 m east of the existing bridge (as measured centreline to centreline). Based on the GA drawing dated March 2023, a 39.5 m long, single-span and single-lane Temporary Modular Bridge (TMB) is proposed to be constructed to carry traffic during removal of the existing and construction of the new bridge. The proposed grade at the TMB north and south approach embankments will be about 2.7 m and 1.9 m above existing grade, respectively. The detour alignment will extend approximately 200 m north and south of the TMB.

We understand that the detour embankment and the TMB structure are to be constructed in 2023, and open to traffic in 2024 during the replacement of the existing bridge; the temporary detour and TMB will be removed at the end of 2024 following completion of the construction of the new bridge.

6.2 Consequence and Site Understanding Classification

A “Typical” consequence level is considered appropriate for the Nagagamisis River Bridge replacement and TMB limit states design, as outlined in Section 6.5 of the Canadian Highway Bridge Design (CHBDC 2019) and its Commentary. Given the scope of work of the site-specific foundation field investigation and laboratory testing program, a “Typical” degree of site and prediction model understanding has been utilized. Accordingly, the appropriate corresponding Ultimate Limit States (ULS) and Serviceability Limit States (SLS) consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC (2019) have been used for design.

6.3 Foundations

The recommended foundations for the replacement bridge and TMB based on the subsurface conditions at this site are as follows:

- **Replacement bridge:** Steel H-piles are recommended to support the replacement bridge. Shallow foundations are not considered feasible for the anticipated loading of the new structure due to the relatively low geotechnical resistances and settlement performance for this option.
- **TMB:** Shallow foundations on a granular pad overlying the cohesionless deposit are recommended to support the relatively light TMB abutments compared to the heavier replacement bridge. Driven piles could also be considered but are not preferred given the temporary nature of the bridge and relative higher tolerance to movements of the TMB foundations which can be mitigated as necessary.

The following sections provide detailed foundation recommendations for the replacement bridge and TMB.

6.3.1 Deep Foundations – Steel H-Piles

The replacement bridge could be supported on steel HP310X110 piles driven to bedrock, which allows for an integral abutment design. However, due to the presence of cobbles and boulders within the till, which could cause the piles to “hang up” or be deflected from their intended vertical alignment, consideration should be given to using a heavier H-pile section, such as HP310X132 or HP360x132, to reduce the potential for damage to the piles during driving to the required tip elevation.

It is understood that a temporary cofferdam is proposed within the footprint of the replacement bridge abutments to control water inflow. It is further understood that as alternative to standard 3 m long corrugated steel pipes (CSPs for integral design), a permanent 610 mm diameter casing will be advanced. The proposed closed-box sheet pile cofferdam will require sheet piles driven to a suitable depth of penetration to mitigate base instability and minimize water inflow; dewatering and/or the use of a concrete tremie plug at the base of the excavation may be required to maintain the excavation integrity and stability. Alternatively, excavation of soils could be completed in wet conditions (i.e., without dewatering/unwatering and with the excavation flooded) within the cofferdam although specialized methods and/or dewatering/unwatering is anticipated to be required for placement of the concrete pile caps.

The following sections provide details regarding the tip elevation, geotechnical axial resistances, set criteria and pile driving notes, resistance to lateral loads and frost protection for driven steel H-piles.

6.3.1.1 Design Tip Elevation

The piles should be advanced to refusal on the bedrock at the estimated design tip elevations as follows:

Bridge	Foundation Element (Relevant Boreholes)	Proposed Underside of Pile Cap (m)	Estimated Pile Tip Elevation (m)	Estimated Design Pile Length (m)
Replacement Bridge	North Abutment (NG-3, NG-8)	285.5	278.5	7.0
	South Abutment (NG-2, NG-7)	285.5	278.1	7.4

Varying pile lengths from those presented above will need to be considered when ordering the piles due to the piles possibly “hanging up” on the cobbles and boulders, as well as the variability of the bedrock surface and the depth to bedrock. Piles would have to be predrilled to meet the minimum 5 m length for integral abutment design if piles are “hanging up” on cobbles and boulders.

6.3.1.2 Geotechnical Axial Resistance

For HP310x132 steel H-piles driven to bedrock, a factored ultimate geotechnical resistance of 2,300 kN per pile would normally be applicable for design. However, due to the potential of the piles hanging up on cobbles and/or boulders at this site and to avoid overdriving and damaging the piles, it is recommended that a factored ultimate geotechnical resistance of 1,800 kN per pile be used for design. The factored serviceability geotechnical resistance for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored ultimate geotechnical resistance; as such, the factored ultimate geotechnical resistance will govern for this foundation type.

If the piles do not reach the pile tip elevation (i.e., bedrock surface elevation), then it is likely that the piles have “hung-up” on an obstruction within the till deposit above the bedrock. If this occurs, the pile should be tested in the field using pile dynamic analyzer (PDA) testing (i.e., High Stain Dynamic Testing), and the designers should be contacted to determine if the measured pile capacity is sufficient for support of the permanent structure in conjunction with the results for the remainder of the piles at the foundation element. This specific requirement has been added to the “fill-in” portion of DBSP0903 (Construction Specification for Deep Foundations) which is included in Appendix C. In addition, detailed specifications for the PDA testing has been added to the “fill-in” portion of DBSP0903 in Appendix C.

6.3.1.3 Set Criteria and Pile Driving Note

Pile installation should be carried out in accordance with DBSP0903 (Construction Specification for Deep Foundations) which supersedes OPSS 903 for this contract and is included in Appendix C.

Based on the presence of the cobbles and boulders within the till layer underlain by bedrock, the piles should be fitted with rock points such as Titus Injector or Oslo Point as per Ontario Provincial Standard Drawing OPSD 3000.201 (HP310 Oslo Point), or equivalent, to assist in seating the piles and to minimize damage to the pile tip during driving.

The pile driving note that should be added to the drawings is Note 5 in Clause 3.3.3 of the Structural Manual (MTO, 2021) as follows:

- “Piles to be driven to bedrock”

6.3.1.4 Downdrag Loads

As the foundation soils are cohesionless and compact to very dense in relative density and minimal settlement is anticipated as a result of the proposed embankment loading, downdrag loads need not be considered for design of the pile foundations.

6.3.1.5 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. Where vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. The evaluation of the piles subjected to lateral loads should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects.

It is understood that an integral abutment foundation design has been chosen for the replacement bridge. Based on the shallow soil conditions at this site, the integral abutment design would include the installation of 3 m long driven casing (with soil augered out), as an alternative to the corrugated steel pipe (CSP) liners, with the annular space between the pile and the casing backfilled with uniformly graded, uncompacted sand, so that the upper portion of the H-piles will be free to flex and move laterally within the limits of the casing (see Appendix C). With this design, the passive lateral resistance over the length of the pile within the casing should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the casing should be based on the resistance provided by the surrounding soil conditions.

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory as outlined below. However, it should be noted that the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are typically less than 1 percent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behavior of the soil should be considered by the use of P-y curves.

The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory suggested in the 2019 CHBDC Commentary (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction, k_h , (kPa/m) is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (CFEM 1992).

$$k_h = \frac{n_h z}{B}$$

where:

- n_h = constant of horizontal subgrade reaction (kPa/m), as given below;
- z = depth (m) below the underside of the pile cap
- B = pile diameter or width (m)

The following values of n_h (Terzaghi, 1955) may be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) for structural analysis for a single vertical pile. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection).

Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)
North Abutment (NG-3, NG-8)	Casing (3 m)	285.5 (underside of pile cap) to 282.5	1,300
	Sand and Gravel to Gravelly Silty Sand Till, Compact to Very Dense	282.5 to 278.5	11,000
South Abutment (NG-2, NG-7)	Casing (3 m)	285.5 (underside of pile cap) to 282.5	1,300
	Sand and Gravel to Gravelly Silty Sand Till, Dense	281.5 to 278.1	11,000

The structural resistance of the pile should be evaluated to establish the governing case at ULS, although the design is often governed by limiting deformations at SLS. For SLS design, the horizontal reaction should be taken as that corresponding to a limiting horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments.

The upper zone of the soil (down to a depth below the pile cap equal to about $1.5 \times B$ (where B is the pile diameter)) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters between rows of driven steel H-piles. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2, 1986) as follows:

Pile Spacing in Direction of Loading (D = Pile Diameter)	Subgrade Reaction Reduction Factor, R
8D	1.00
6D	0.70
4D	0.40
3D	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above summary. Reduction for group effects is negligible when the centre-to-centre pile spacing exceeds about three pile diameters measured in the direction perpendicular to loading.

6.3.2 Shallow Foundations for TMB

It is recommended that the TMB abutments be supported on spread footings founded on a Granular B Type II pad overlying the loose to compact sand. It is understood that the Design-Builder is proposing to use precast footings for this structure. As such, any precast footings should follow the manufacturer's recommendations to ensure the base of the footing is in intimate contact with the foundation subgrade such that the load is evenly distributed to the granular pad. It is recommended that the detour alignment approach embankments be constructed from the abutment to at least 20 m behind the abutment at the same time / elevation as the top of granular pad (i.e., at least up to underside of proposed TMB abutment and preferably to temporary pavement structure subgrade level). The granular pad and approach embankment should be preloaded for at least one week before TMB abutment construction to reduce the potential for differential settlement. Construction considerations for the granular pad and detour approach embankments are discussed further in Sections 6.6.3 and 6.4.1. The recommended founding elevation of the granular pad at each foundation element is summarized below.

Foundation Element (Relevant Boreholes)	Existing Ground Surface at Borehole Location (m)	Proposed TMB Underside of Abutment Footing Elevation (m)	Recommended Approximate Sub-excavation/Footing Elevation of the Granular Pad (m)
North Abutment (NG-6)	286.9	288.4	285.9 (Recommended 2.5 m thick pad)
South Abutment (NG-5)	287.8	288.6	286.1 (Recommended 2.5 m thick pad)

6.3.2.1 Geotechnical Resistance

Spread footings founded at the elevations given in Section 6.3.2 should be designed based on the factored ultimate geotechnical axial resistance and factored serviceability geotechnical resistance given below.

Founding Stratum	Footing Width (m)	Factored Ultimate Geotechnical Axial Resistance (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
Granular pad over loose to compact sand, over dense sand, and gravel to gravelly silty sand till	1.9	150	110

The factored geotechnical resistances and the settlement are dependent on the footing size, depth of embedment and applied loads, and do not include settlement of the granular pad itself. The geotechnical resistances should, therefore, be reviewed if the selected footing width, founding elevation or granular pad thickness differs from those given above. In addition, the factored geotechnical resistances provided above are based on the loading applied evenly (i.e., intimate contact between base of footing and foundation subgrade) and perpendicular to the base of the footings; where applicable, inclination of the load should be taken into account.

All loose, softened, or disturbed subgrade soil should be removed immediately prior to placement of concrete. Construction and inspection of footings, including prior to placement of the granular pad, should be carried out in accordance with DBSP 0902 (Excavating and Backfilling – Structures).

6.3.2.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance should be calculated in accordance with Section 6.10.4 of the CHBDC (2019) applying the appropriate consequence and degree of site understanding factors as noted in Section 6.2. An unfactored coefficient of friction, $\tan \delta'_i$, of 0.5 may be used at the interface between the base of the precast concrete footing and the granular pad.

6.3.3 Frost Protection

The pile caps for the replacement bridge should be provided with a minimum of 2.6 m of soil cover for frost protection as per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario), or a combination of soil cover and rigid insulation. For polystyrene insulation, the MTO has typically adopted an equivalency of 25 mm of insulation for every 0.3 m reduction in soil cover. It is understood that rigid insulation will be placed horizontally below the pile cap with sufficient thickness and extending sufficient distance with sufficient soil cover to meet the minimum 2.6 m frost protection requirements; further, it is understood that insulation will be placed along the exterior vertical face of the abutment wall extending to the horizontal layer at the bottom of the abutment. The insulation installed horizontally should be founded on 100 mm of moderately compacted Granular B Type II leveling pad for support.

We understand the TMB will be constructed in the first construction season, left unopened over winter and opened to traffic in the second construction season. As the detour is not open to traffic and is only to be left in place one winter, we anticipate that frost protection would not be required based on the underlying low frost susceptible granular pad and the flexibility of the TMB which can be adjusted to accommodate some differential settlement prior to opening to traffic. However, if the TMB will be in operation over winter, it is recommended that frost protection be provided to mitigate frost-related differential movements over repeated freeze-thaw cycles, or that provision be made to adjust and re-level the TMB as required to ensure the structural integrity is not compromised.

6.3.4 Seismic Considerations

Subsurface ground conditions for seismic site characterization were established based on the results of the borehole investigations. Based on the anticipated foundation levels on/within the bedrock, the site may be classified as Site Class C in accordance with Table 4.1 of the CHBDC (2019), in the absence of any geophysical testing. Geophysics testing, if carried out, could provide a more favourable Site Class designation, but would also depend on the elevation of the abutment foundations. For example, Table 4.1 of the CHBDC (2019) indicates that Site Class A and B are not to be used if there is more than 3 m of soils between the rock and the underside of the bridge foundations (i.e., footings or pile caps).

Based on the information obtained from the NRCan (2015) Hazard Calculator for this site located at latitude 49.4744° and longitude -84.7257°, the following Site Class C values were obtained for the spectral acceleration for a return period of 2,475 years:

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
Sa (0.2) (g)	0.063
Sa (1.0) (g)	0.025

The structural designers have classified the new bridge and temporary bridge as “major” or “other”. Based on the values noted above, bridge classification, and in accordance with Table 4.10 of the CHBDC (2019), this site should be considered to be located in Seismic Performance Zone 1. In accordance with Section 4.4.5.1 of the CHBDC, no seismic analysis is required for structures located in Seismic Performance Zone 1.

A liquefaction assessment was completed for this site. The results indicate that the soils have a low potential for liquefaction during the 2,475-year design earthquake, and therefore the site soils may be considered to be non-liquefiable for design.

6.4 Approach Embankment Design and Construction

Based on the GA drawing provided by WSP, the proposed highway grade at the north and south approach embankments for the replacement structure will be at Elevation 290.1 m and 290.0 m, approximately 500 mm and 300 mm above the existing highway grade, respectively. The proposed detour north and south approach embankments will be at Elevation 289.6 m and 289.7 m, about 2.7 m and 1.9 m above existing grade along the proposed detour centreline, respectively.

The following sections address subgrade preparation and embankment construction, and stability and settlement analysis for the raised approach embankments on Highway 631 and the new/widened approach embankments on the detour. To improve settlement performance, consideration could be given to the existing fill and organic silty sand (north abutment/approach) being removed from below the footprint of the reconstructed embankment behind the abutments for the new permanent bridge; however, as this material has been here since construction of the existing bridge and only minor widening is required for the existing highway, this material could remain in place. The geometry of the proposed embankments, existing ground surface and existing river bed included in the stability and settlement analyses are based on the GA drawing. The piezometric conditions used in the analyses are based on the groundwater level as encountered during the subsurface investigation.

As discussed in Section 2, during the foundation investigation, surficial sloughing was observed at the east side of the north approach embankment; based on cross-sections of the existing highway embankment in this area, the existing slopes are oriented at approximately 1.8 horizontal to 1 vertical (1.8H:1V). It is understood that the preferred option is to maintain the existing embankment footprint/geometry to minimize environmental impacts on the existing waterway. As such, WSP is considering the use of rock fill for the approach embankments within the immediate vicinity of the abutments (i.e., within 20 m behind the abutment), to achieve adequate global and surficial stability while maintaining the existing footprint of the embankment. In this case, a portion of the existing granular fill will need to be removed and replaced with rock fill for the full embankment length where granular fill with side slopes of 2H:1V cannot be achieved. Granular fill will still be required as backfill behind the immediate abutment area for structural requirements and to facilitate pile driving.

6.4.1 Subgrade Preparation and Embankment Construction

Engineered fill for reconstruction of the raised/widened embankments behind the new abutments, and for construction of the new/widened detour embankments, should consist of Granular A or Granular B Type I or Type II meeting the specifications of OPSS.PROV 1010 (Aggregates). Rock fill can be considered for the embankment widening and detour approach embankments (up to the granular pad); however, it is recommended that rock fill not be used immediately behind the new permanent abutments. The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting), OPSS.PROV 206 (Grading) and DBSP 0902 (Structures). Embankment side slopes for granular fill should be constructed no steeper than 2H:1V. Embankment side slopes for rock fill should be constructed no steeper than 1.25H:1V. The top surface of the embankment shall be chinked with rock fragments and spalls to form the subgrade prior to the placement of the roadway subbase in order to minimize voids and prevent migration of the subbase material into the rock fill (OPSS.PROV 206).

The abutment front slopes and side slopes adjacent to the river require erosion protection in accordance with OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting). Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. Subject to confirmation and modifications as necessary based on the hydrology reports (by others), erosion protection could consist of a minimum 0.6 m thick layer of R-10 Rip Rap (180 mm size as per OPSS.PROV 1004 (Aggregates - Miscellaneous)), rock protection or concrete slope paving. The designer should address the potential for hydraulic scour below the pile caps in the design of the replacement bridge foundations and embankments.

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting), and OPSS.PROV 1004 (Aggregates – Miscellaneous) should be considered to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding. If rock fill is used, erosion protection is not considered to be required.

6.4.2 Embankment/Temporary Detour Stability

Slope stability analyses were carried out for the front slope of the proposed replacement bridge north approach embankment, which is considered to be the critical slope at this site. Figure 1 shows the north approach front slope embankment geometry in the context of the interpreted stratigraphic profiles on Drawing 1.

The Factor of Safety (FoS) is defined as the ratio of forces tending to resist failure to the driving forces tending to cause the failure. For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, ψ , and the geotechnical resistance factor ϕ_{gu} (i.e., $FoS = 1/(\psi * \phi_{gu})$). Accordingly, a target minimum FoS of 1.3 has been used for design of the temporary embankment side slopes, and FoS of 1.5 for the design of the final embankment configuration and vertical walls as per Table 6.2 of CHBDC (2019) for the total stress (short-term undrained) and effective stress (long-term drained) condition, as applicable.

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.23), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The stability analyses were performed to check that the target minimum FoS was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design.

For the new granular fill, the new rock fill at the north approach, the existing granular fill, and the cohesionless native soil deposits, effective stress parameters were employed in the analysis assuming drained conditions, and the parameters were estimated from empirical correlations using the in-situ SPT 'N'-values. The correlations proposed by Terzaghi and Peck (1967) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils. Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed works areas.

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
New Granular Fill (i.e., Granular A or B Type I or II)	21	35	-
New Rock Fill (North Approach Embankment)	19	40	-
Existing Granular Embankment Fill	20	32	-
Sand	20	30	-
Sand and Gravel to Gravelly Silty Sand Till	20	32	-

6.4.2.1 Results of Analysis

The stability analysis indicates that the front slope of the new north approach embankment will have a FoS greater than 1.3 against global instability in short-term conditions and 1.5 for the long-term, effective stress conditions, as shown on Figure 1. The stability analysis for the embankment side slopes for the south and north approach embankment reconstructed with rock fill (at 1.25H:1V) indicates that the FoS is greater than 1.5 for the long-term condition.

6.4.3 Embankment Settlement

For the approach embankments to the replacement bridge, based on the minimal grade raise and relative density of the native cohesionless soils, settlement of the foundation soils is anticipated to be relatively minor (i.e., 25 mm or less) provided the existing fill and organic materials are removed at the north approach prior to the embankment reconstruction/grade raise.

For the approach embankments to the TMB, to estimate the magnitude of the expected settlements of the native cohesionless soils, the immediate compression of the cohesionless deposits was modelled by estimating an elastic modulus of deformation based on the SPT “N”-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). The simplified stratigraphy together with the associated strengths and unit weights are summarized below. As the till deposit is generally noted to be dense to very dense, settlement of this deposit will be negligible under this low embankment height, and this stratigraphic unit has not been included below. To estimate the magnitude of the expected settlements, analyses were carried out on the critical section of the proposed TMB approach embankments using hand calculations.

Soil Type	γ (kN/m ³)	Settlement Parameters
Sand, Loose	20	Es = 3 MPa (north approach) Es = 10 MPa (south approach)

Settlement of new granular embankment fill that is properly placed and compacted, is considered nominal and would occur during or shortly after construction.

6.4.3.1 Settlement Performance Requirements

The settlement performance criteria for design of high fill embankments for the existing Highway 631 alignment (i.e., not for the detour) are in accordance with MTO Foundations Guideline, “*Embankment Settlement Criteria for Design*” (MTO, July 2010).

Where new embankments approach structural elements, the following post-construction settlement and differential settlement criteria are considered acceptable for settlements to occur within 20 years post-paving for the bridge approach embankments at this site (MTO, July 2010).

Location	Maximum Limits During Pavement Design Life	
	Distance from Transition Point (i.e., Abutment)	Total Post-Construction Settlement (mm)
Transition/Taper to Bridge Abutments	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	75

These criteria have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments.

The total settlement and differential settlement rate are considered to be applicable over a 20-year period following completion of construction (i.e., final paving). These performance criteria form part of the overall design performance for the embankment in the vicinity of the bridge replacement.

6.4.3.2 Results of Analysis

As noted above, based on the minimal grade raise on the approach embankments to the replacement bridge, settlement of the foundation soils is anticipated to be less than 25 mm.

At the TMB detour approach embankments, given the grade raise ranging from 1.9 m (south approach) to 2.7 m (north approach), the settlement of the native deposits below the south and north approach embankments for the proposed detour is expected to be between 25 and 50 mm, and the majority of this settlement will occur during or shortly after the detour embankment construction. Additional settlement (up to 25 mm) due to compression of the granular pad fill itself is also anticipated to occur during or shortly after construction, especially if the granular fill is placed below the water level as discussed in Section 6.6.3.

As a result, it is recommended that the detour approach embankments be constructed up the pavement subgrade level at the same time as the granular pad (supporting TMB abutments) and allowed a preload period of at least one week prior to placing / constructing the TMB footings and modular bridge units.

6.5 Lateral Earth Pressures

The lateral earth pressures acting on the abutment walls and any associated wing walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of walls for this site. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and OPSD 3190.100 (Walls, Retaining and Abutment, Wall Drain).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2019) Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall, with limitations on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 2.6 m behind the back of the wall (in accordance with Figure C6.31 (a) of the Commentary to the CHBDC). For unrestrained walls, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing in accordance with Figure C6.31(b) of the Commentary to the CHBDC (2019).

- For restrained walls, the pressures are based on the proposed embankment fill material behind the structure backfill zone, while for unrestrained walls, the pressures are based on the granular backfill; the following parameters (unfactored) may be used:

Fill Type	Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_0	Active, K_a
Granular 'A'	22 kN/m ³	0.43	0.27
Granular 'B' Type II	21 kN/m ³	0.43	0.27
Rock Fill	19 kN/m ³	0.36	0.22

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the foundation design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, should be calculated in accordance with Section C6.12.1 and Table C6.12 of the *Commentary to the CHBDC*, 2019.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.6 Construction Considerations

6.6.1 Excavations and Control of Groundwater and Surface Water

Prior to the construction of the new embankments, it is recommended that the existing fill and organic soils be removed from below the footprint of the proposed embankment within 20 m of the proposed replacement bridge and detour abutments. For the replacement bridge north abutment/approach, excavations will extend to approximately Elevation 284.0 m (5.6 m deep relative to the existing grade) to remove the organic silty sand. For the replacement bridge south abutment/approach, excavations will extend to Elevation 284.5 m (5.3 m deep relative to the existing grade) to the underside of the proposed pile cap. For the TMB, excavations will be shallow to remove surficial organics, if present in the footprint of the approach embankments, and/or to achieve a 2.5 m thickness for the granular pad.

Temporary protection systems will be required for protection of the existing bridge abutments and existing highway embankment during TMB/detour construction, as well as for protection of the TMB and detour embankments during sub-excavation and construction of the replacement bridge. Recommendations for temporary protection systems are provided in Section 6.6.2.

Temporary open-cut excavations must be carried out in accordance with the latest version of the Occupational Health and Safety Act (OHSA) and Regulation for Construction Activities. The existing fill and organic soils are classified as Type 3 and 4 soil according to the OHSA. Temporary excavations (i.e., those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V above the water level and 3H:1V below the water level. Open excavations below the water level are not recommended, except for the temporary excavation anticipated for the construction of the granular pad for the TMB abutments, as discussed in Section 6.6.3.

Excavations and groundwater / surface water control must be in accordance with the design-build specification DBSP0902. Temporary excavations for the replacement bridge abutments will extend below the groundwater / lake water level. Due to the close proximity of the abutments to the lake, a groundwater / surface water cut-off (cofferdam or similar measure) is recommended to minimize dewatering / unwatering requirements and the occurrence of potential environmental impacts, as discussed further in Section 6.6.2. Dewatering / unwatering of all excavations should be carried out in accordance with DBSP0902 using the minimum design storm return period as recommended by the hydrology specialist, with a preconstruction survey radius of 100 m. Groundwater control will be required to maintain the integrity of the abutment excavations, as well as the stability of the soils at the base and/or sides of the casing (alternative to CSP liner) against basal heave/disturbance and sloughing due to unbalanced groundwater pressures and the integrity of the sand fill once placed within the casing (alternative to CSP liners, installed in the wet). It is understood that the casings will be advanced under full head of water, drive the piles, and subsequently fill the annulus with sand fill in combination with the temporary protection system. A special provision should be included in the Contract to alert the contractor to the groundwater / surface water conditions and that the pile cap excavation must be unwatered and kept stable during pile cap construction, including placement of the casings for the integral abutments; an example Notice to Contractor and special provision is included in Appendix C. Installation of the piles after placement of sand fill within the casings is not recommended due to the risk of densifying the sand fill during pile installation and compromising the flexibility characteristics required for integral abutment design.

The native sand soils that will be exposed within the excavation at the abutments may be susceptible to disturbance from construction traffic and/or ponded water. A concrete working slab or concrete tremie plug (if designed as part of and in conjunction with the unwatering/temporary works) should be placed below the pile cap, above the subgrade.

Surface / lake water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation or temporary raised water levels, but all surface water should be directed away from the excavations. Seepage from the granular fills should be expected, particularly after precipitation events. It is anticipated that minor surface water seepage and seepage from the granular fills can be controlled by using properly filtered sumps within the excavation.

6.6.2 Temporary Protection Systems and Cofferdams

Temporary protection systems will be required to remove existing fill and organic soil below the new abutments/approaches, to allow for construction of the approach embankments for both the TMB and replacement bridge. Temporary protection systems shall be designed and constructed in accordance with DBSP 0539 (Temporary Protection Systems), provided that any existing adjacent structures or utilities can tolerate this magnitude of deformation. The lateral movement of the temporary shoring systems shall meet Performance Level 2 as specified in DBSP 0539.

It is considered that either a driven, interlocking sheet pile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at the abutments, based on the subsurface soil and groundwater conditions. An interlocking sheet pile system would contribute to both ground and, where applicable, groundwater control – it would provide for control of seepage of groundwater from the underlying till. For a soldier pile and lagging system, more extensive dewatering would likely be required, and in addition it may be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards.

The sheet piles or soldier piles would have to be driven or socketted to sufficient depth to provide the necessary passive resistance for the retained soil height, including any surcharge loads behind the protection system within at least a 1H:1V zone relative to the base of the excavation. Lateral support to the sheet piles or soldier piles could be provided in the form of rakers or temporary anchors.

The selection and design of the protection system will be the responsibility of the Contractor.

6.6.3 Granular Pad for TMB Shallow Foundation

The pad below and behind the TMB abutments for the approach embankments should be constructed using OPSS.PROV 1010 Granular 'B' Type II material. The granular pad should extend at least 1 m beyond the plan limits of the abutment, and be sloped no steeper than 1H:1V, downward and outward from the top of the pad to the subgrade. The granular pad should be constructed in accordance with OPSS.PROV 206 and DBSP0902 (Structures). Due to the shallow excavation below the water level at the TMB abutments, the granular pad could be constructed in wet conditions (i.e., without dewatering within the excavation), if applicable, with moderate compaction with the excavator bucket under the review of a qualified Foundation Specialist. The granular pad should be constructed concurrently with embankment construction and preloaded for at least one week before TMB abutment construction (see Notice to Contractor in Appendix C to be included in Contract documents) to reduce the potential for differential settlement occurring as indicated in Section 6.3.2. The founding level may need to be re-established / re-levelled after the preload period and prior to placement of foundations; further, adjustments to the bridge abutment supports over the footing may be required over the life of the structure depending on settlements. The granular pad will need to be protected from erosion / scour of the river during construction as required by the hydrologic / hydraulic assessment.

6.6.4 Obstructions

The native soils at this site are glacially derived and as such are very dense and contain coarse gravel, cobbles and boulders as noted on the borehole records, which could affect the installation of deep foundations, excavations for foundations and installation of cofferdams/temporary protection systems. A Notice to Contractor should be included in the Contract Documents to identify to the contractor the presence of cobbles and/or boulders within the overburden soils; an example is included in Appendix C.

6.6.5 Vibration Monitoring

Vibrations induced to a structure up to a maximum peak particle velocity (PPV) of 100 mm/s are generally considered applicable for bridge structures in good condition. However, as the existing bridge is in poor condition, it is recommended that a lower peak particle velocity be adopted for this site, at least during the start of sheet piles driven closest to the existing bridge if in operation. Based on vibration monitoring experience, it is considered unlikely that the vibrations induced by conventional construction activities will affect the performance of the existing structure, but may reach this threshold level. Therefore, vibration monitoring should be carried out during construction at this site adopting a PPV of 50mm/s at the existing bridge if it is to remain in operation during any pile driving activities (see Appendix C). Any vibration monitoring should be accompanied by settlement monitoring as described in the section below to assess whether the PPV threshold can be modified accordingly.

6.6.6 Existing Structure Monitoring

We recommend that the abutments of the existing structure be monitored for settlement and lateral movement during construction of the TMB and detour approach embankments, especially during construction works adjacent to the existing structure, such as excavation operations, installation of temporary protection/cofferdams and installation of deep foundations for the following reasons:

- The existing bridge is supported by timber cribbing, and is in poor condition.
- The existing structure is required to carry traffic during construction of the detour and TMB.

The structure monitoring program should be developed by the structural engineering team and a preconstruction condition survey of the existing bridge is recommended.

6.6.7 Analytical Testing for Construction Materials

The results of an analytical test carried out on soil samples from Boreholes NG-7 and NG-8 from the approximate foundation element elevations for the replacement structure are presented in Table B1 in Appendix B. The suite of parameters tested is intended to allow the design engineer to assess the requirements for the appropriate type of cement to be used in construction and the need for corrosion protection of steel reinforcing elements.

For potential sulphate attack on concrete, the results of the soil analysis were compared to Table 3 in CSA A23-1, and indicate that the relative degree of sulphate attack is low (less than the moderate range). However, given that the bridge will be exposed to de-icing salts it is recommended that C-1 class exposure concrete be considered. Further, the resistivity results indicate that the soil has a severe of corrosiveness ($R > 2000$) potential based on the Transportation Research Board Guidelines (Transportation Research Board, National Research Council, 1998 as referenced in the MTO Gravity Pipe Design Guidelines, 2014).

It should be noted that the creek water levels in the area are subject to seasonal fluctuations and variations due to precipitation events, and as a result the water and/or soil chemistry could also be variable. These recommendations are provided as guidance only; the structural designer should take the results of the laboratory testing and the potential for corrosion into consideration when selecting materials for bridge construction.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. André Bom, P.Eng. and Mr. Kevin Bentley, P.Eng., a Designated MTO Foundations Contact for WSP Golder, conducted an independent quality control review of this report.

Signature Page



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Geotechnical Engineer



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AB/KJB/ca

[https://golderassociates.sharepoint.com/sites/163211/project files/6 deliverables/ph 1000-fdns and pvmnts/02-nagagamisis/final/22525553-r01-r-rev0-1000-nagagamisis replacement fidr 20apr_23.docx](https://golderassociates.sharepoint.com/sites/163211/project%20files/6%20deliverables/ph%201000-fdns%20and%20pvmnts/02-nagagamisis/final/22525553-r01-r-rev0-1000-nagagamisis%20replacement%20fidr%20apr_23.docx)

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- ASTM International
- ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
- Commercial Software
- GeoStudio 2007 (Version 7.23) by Geo-Slope International Ltd.

Ontario Provincial Standard Drawings

OPSD 202.010	Slope Flattening
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

Ontario Provincial Standard Specifications

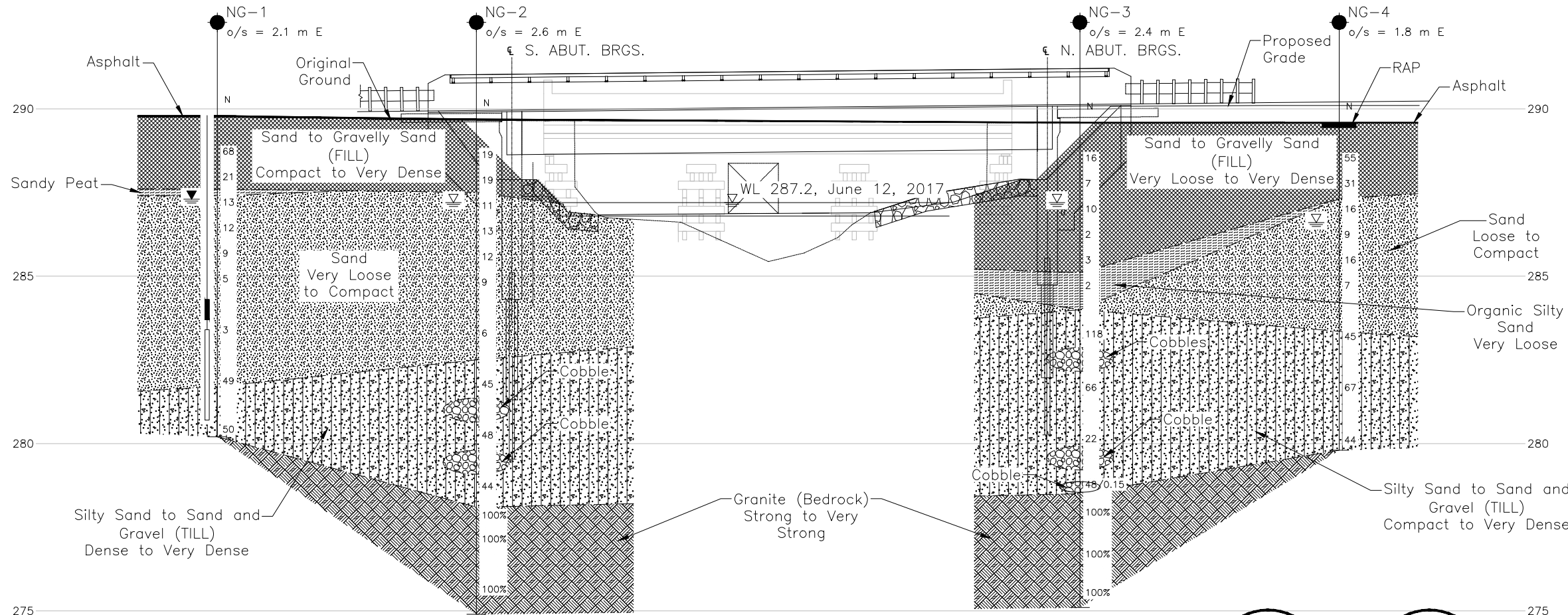
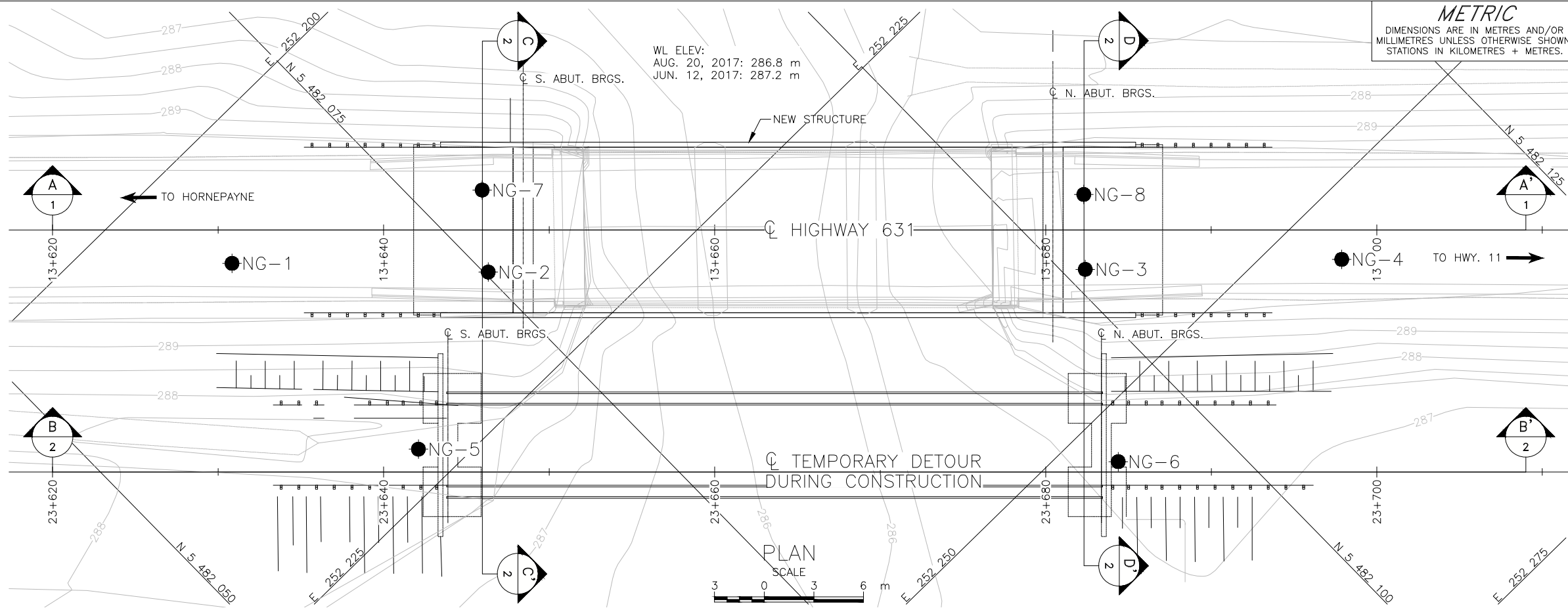
OPSS.PROV 206	Construction Specifications for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
OPSS 802	Construction Specification for Topsoil
OPSS 804	Construction Specification for Seed and Cover
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act

Ontario Regulation 903	Wells (as amended)
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Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects (as amended)
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A-A EXISTING HWY 631 PROFILE



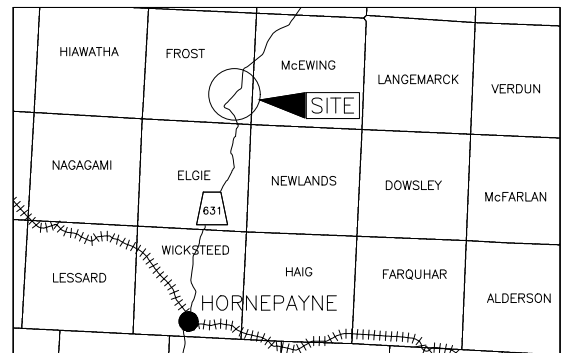
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2021-5168
WP No.



HIGHWAY 631
NAGAGAMIS NARROWS BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEY PLAN
SCALE
10 0 10 20

LEGEND

- Borehole - Current Investigation
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ▽ WL in piezometer, measured on JUNE 12, 2017
- ▽ WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 13)

No.	ELEVATION	NORTHING	EASTING
NG-1	289.8	5482064.5	252206.8
NG-2	289.7	5482075.2	252218.0
NG-3	289.6	5482101.1	252243.1
NG-4	289.6	5482112.6	252253.5
NG-5	287.8	5482064.7	252222.7
NG-6	286.9	5482094.4	252252.8
NG-7	289.8	5482078.4	252214.2
NG-8	289.6	5482104.2	252239.8

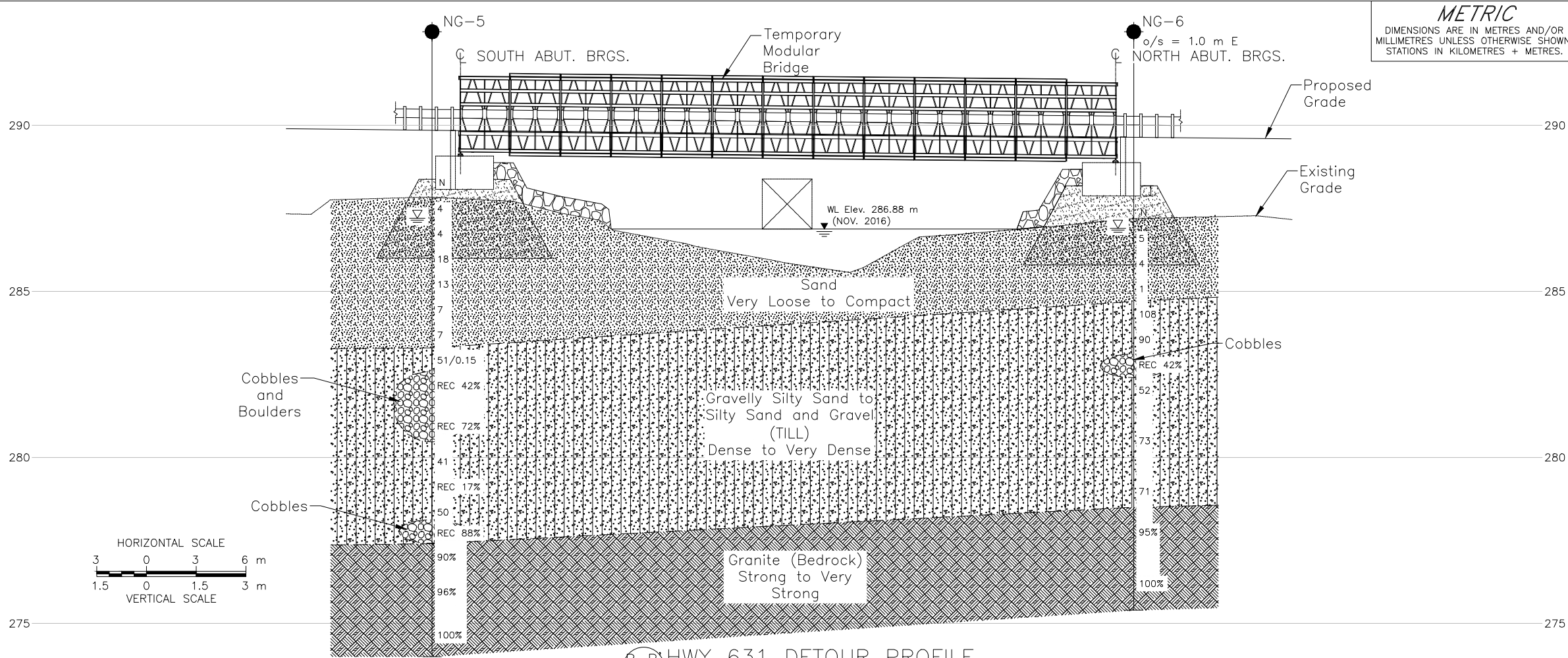
NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents. The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

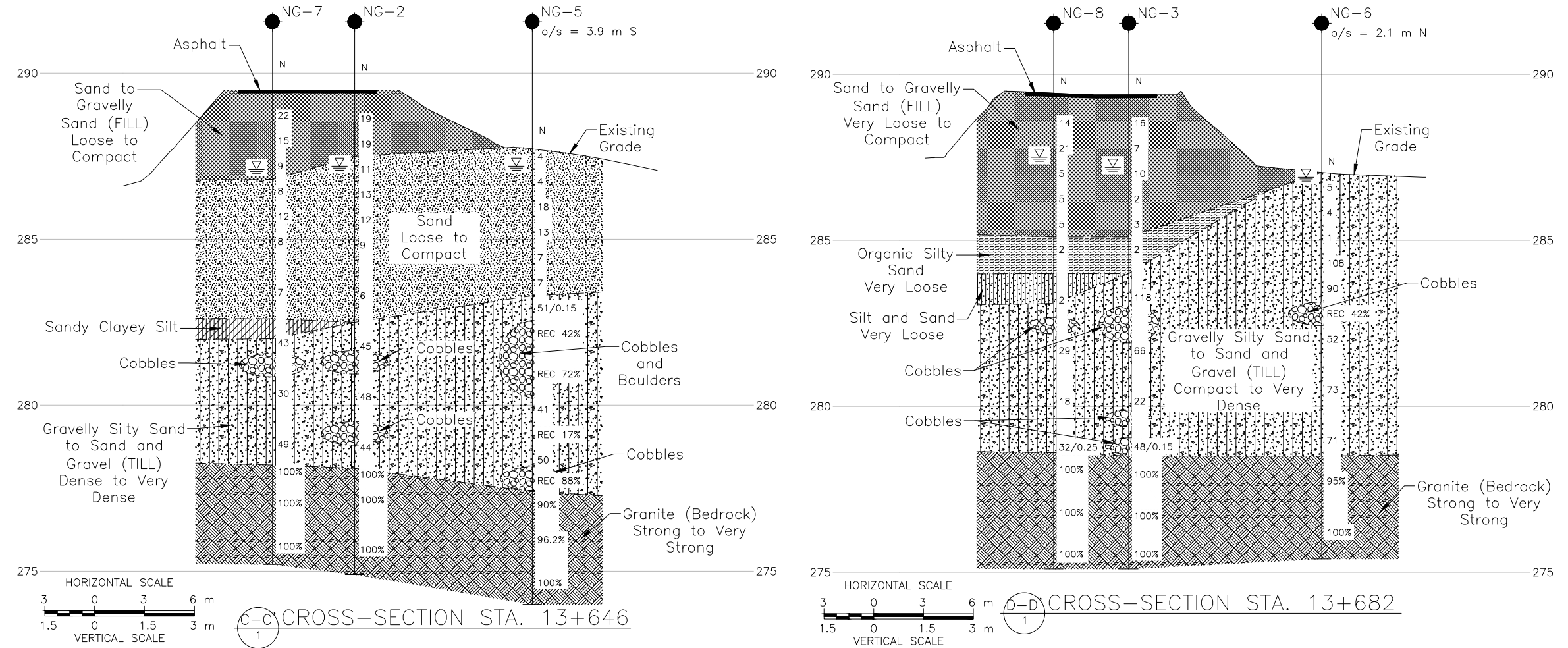
REFERENCE

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- Bridge General Arrangement provided in digital format by WSP, drawing file no. S221-09193-00-300-001GA.dwg, Received December 12, 2022.
- Detour Bridge General Arrangement provided in digital format by WSP, drawing file no. S221-09193-00-301-001GA.dwg, Received December 12, 2022.
- Base Plan provided in digital format by WSP, drawing file no. x17197 Nagagamis Base.dwg, Received December 12, 2022.
- Existing Highway 631 Alignment provided in digital format by WSP, drawing file no. 231122_SS_x17197 Nagagamis ALI_Ultimate.dwg, Received December 12, 2022.
- Detour Bridge Alignment provided in digital format by WSP, drawing file no. X17197 Nagagamis ALI_Detour.dwg, Received December 12, 2022.

NO.	DATE	BY	REVISION
Geocres No. 42F-62			
HWY. 631	PROJECT No. 22525553		DIST. .
SUBM'D. AB	CHKD. .	DATE: 4/18/2023	SITE: 38N-0001/B0
DRAWN: TR	CHKD. AB	APPD. KJB	DWG. 1



B-B HWY 631 DETOUR PROFILE



C-C CROSS-SECTION STA. 13+646

D-D CROSS-SECTION STA. 13+682

METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2021-5168 WP No.	SHEET
HIGHWAY 631 NAGAGAMIS NARROWS BRIDGE SOIL STRATA	



LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- REC Recovery (%)
- 100% Rock Quality Designation (RQD)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 13)

No.	ELEVATION	NORTHING	EASTING
NG-2	289.7	5482075.2	252218.0
NG-3	289.6	5482101.1	252243.1
NG-5	287.8	5482064.7	252222.7
NG-6	286.9	5482094.4	252252.8
NG-7	289.8	5482078.4	252214.2
NG-8	289.6	5482104.2	252239.8



NOTES

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NO.	DATE	BY	REVISION
Geocres No. 42F-62			
HWY. 631	PROJECT NO. 2252553		DIST. .
SUBM'D. AB	CHKD. .	DATE: 4/18/2023	SITE: 38N-0001/B0
DRAWN: TR	CHKD. AB	APPD. KJB	DWG. 2



PHOTOGRAPHS

**Photograph 1: Nagagamisis Lake Bridge
East elevation Looking North-West (May 2017)**



**Photograph 2: Nagagamisis Lake Bridge
Looking North at South-East embankment (May 2017)**





PHOTOGRAPHS

**Photograph 3: Nagagamisis Lake Bridge
North-East embankment bank looking South (May 2017)**



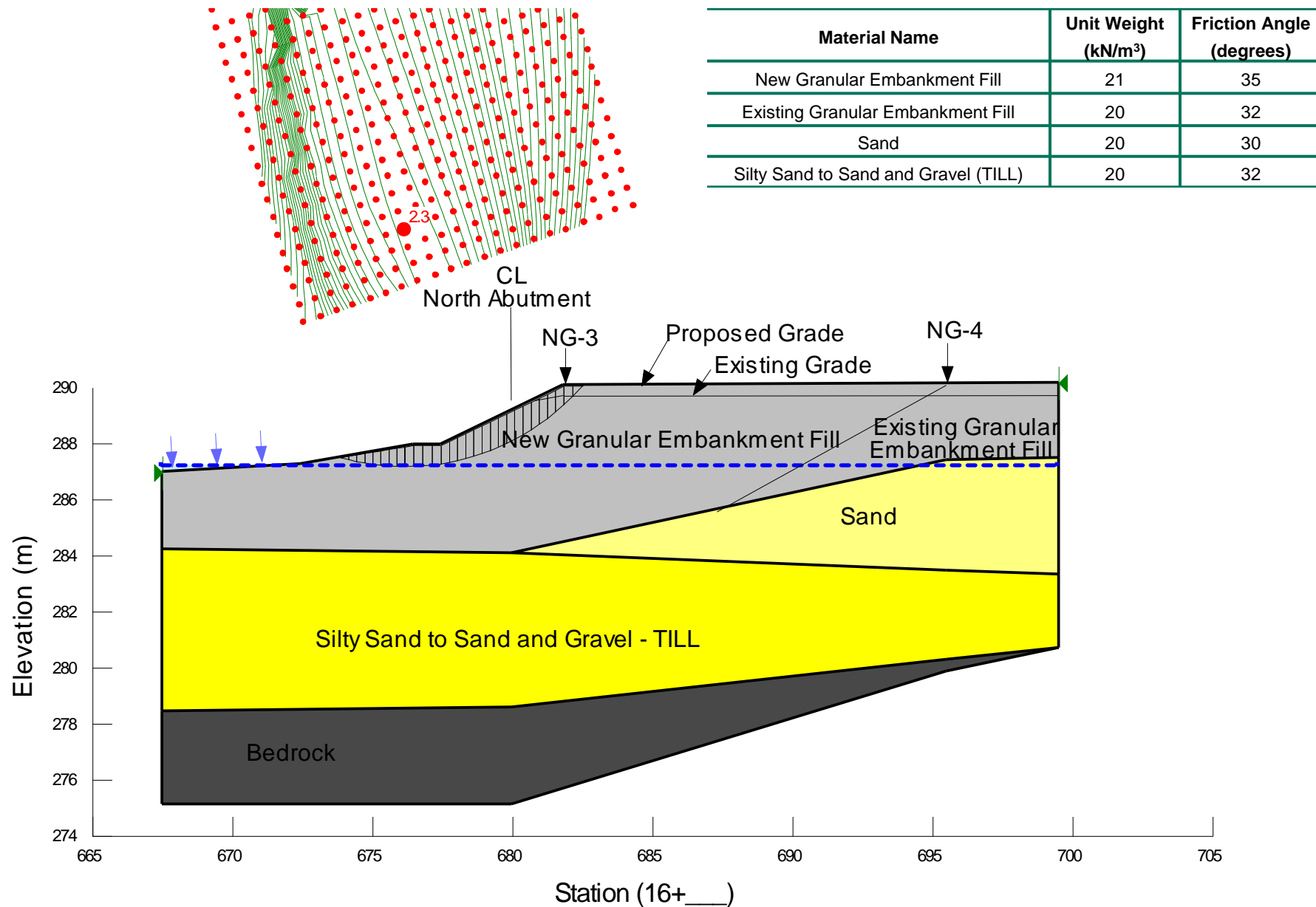
**Photograph 4: Nagagamisis Lake Bridge
East elevation looking South-West (May 2017)**





Global Stability Analysis Short -Term – Undrained Analysis North Abutment/Approach – Front Slope

Figure 1



APPENDIX A

Record of Boreholes

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (<i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (<i>i.e.</i> , some sand)
≤ 10	trace (<i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q_t*), porewater pressure (*u*) and sleeve friction (*f_s*) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, w _L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
Y	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

- Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
- SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 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	> 200	> 30

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
- SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index $= (w_L - w_P)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_P) / I_P$
I_c	consistency index $= (w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{a(e)}$	secondary compression index
C_a	rate of secondary compression
$C_{a(e)}$	modified secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

* Density symbol is ρ . Unit weight symbol is γ .
where $\gamma = \rho \cdot g$ (i.e., mass density multiplied by
acceleration due to gravity)

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING CLASSIFICATION

Fresh (W1): no visible sign of rock material weathering.

Slightly Weathered (W2): discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

Moderately Weathered (W3): less than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Highly Weathered (W4): more than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Completely Weathered (W5): 100% of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

Residual Soil (W6): all rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

AXJ Axial Joint	KV Karstic Void
BD Bedding	K Slickensided
BC Broken Core	LC Lost Core
CC Continuous Core	MB Mechanical Break
CL Closed	PL Planar
CO Contact	PO Polished
CU Curved	RO Rough
CT Coated	SA Slightly Altered
FLT Fault	SH Shear
FOL Foliation	SM Smooth
FR Fracture	SR Slightly Rough
GO Gouge	SY Stylolite
IN Infilled	UN Undulating
IR Irregular	VN Vein
JN Joint	VR Very Rough

ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250

PROJECT 1661607			RECORD OF BOREHOLE No NG-1				1 OF 1 METRIC							
W.P. 5312-14-01		LOCATION N 5482064.5; E 252206.8 MTM ZONE 13 (LAT. 49.474146; LONG. -81.725752)				ORIGINATED BY SA								
DIST _____ HWY 631		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers				COMPILED BY AC								
DATUM GEODETIC		DATE June 11, 2017				CHECKED BY AB								
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		WATER CONTENT (%)				
289.8	GROUND SURFACE						20 40 60 80 100	20 40 60						
0.0	ASPHALT (40 mm)													
0.2	RAP (110 mm)													
289.3	Gravelly sand (FILL)													
0.5	Brown Moist													
	Sand, trace to some gravel, trace to some silt (FILL)		1	SS	68									
	Compact to very dense													
	Brown Moist		2	SS	21									18 74 (8)
287.6	Sandy PEAT, some silt													
	Black Wet		3	SS	13									
	SAND, trace gravel, trace silt													
	Very loose to dense		4	SS	12									
	Brown to grey Wet													
	200 mm to 300 mm of sand heaving in augers at Samples 5 to 7.		5	SS	9									6 89 (5)
			6	SS	5									
			7	SS	3									
			8	SS	49									5 90 (5)
281.7	Silty SAND, some gravel, trace clay (TILL)													
8.1	Dense to very dense													
	Grey Wet													
	Augers grinding from 8.1 m to 9.1 m depth.													
			9	SS	50									10 53 33 4
280.2	END OF BOREHOLE													
9.6	Note: 1. Water level at a depth of 2.5 m below ground surface (Elev. 287.3 m) upon completion of drilling and measured in piezometer on June 12, 2017.													

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1661607 LEA_5015-E-0049_NE REGION02_DATA\GINT\1661607.GPJ GAL-MISS.GDT 11/8/17 TB\JUL

RECORD OF BOREHOLE No NG-2

1 OF 3 **METRIC**

PROJECT 1661607
 W.P. 5312-14-01 LOCATION N 5482075.2; E 252218.0 MTM ZONE 13 (LAT. 49.474243; LONG. -81.725599) ORIGINATED BY SA
 DIST HWY 631 BOREHOLE TYPE Solid Stem Augers, NW Casing and NQ Coring COMPILED BY AC
 DATUM GEODETIC DATE June 10 and 11, 2017 CHECKED BY AB


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	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	× REMOULDED	WATER CONTENT (%)					
289.7	GROUND SURFACE						20	40	60	80	100						GR SA SI CL
0.0	ASPHALT (40 mm)																
0.2	RAP (110 mm)																
289.0	Gravelly sand (FILL) Brown Moist																
0.7	Sand, some gravel, some silt (FILL) Compact Brown Moist		1	SS	19												
			2	SS	19												
287.5																	
2.2	SAND, trace gravel, trace silt Loose to compact Brown to grey Wet		3	SS	11											8 86	(6)
			4	SS	13												
			5	SS	12												
			6	SS	9											11 85	(4)
	150 mm to 200 mm of sand heaving in augers at Samples 6 and 7.																
			7	SS	6												
282.5																	
7.2	SAND and GRAVEL to Gravelly Silty SAND, trace clay (TILL) Dense Gret Wet		8	SS	45											57 36	6 1
	Four 100 mm diameter cobbles encountered from 8.2 m to 9.1 m depth.																
			9	SS	48												
			10	SS	44												
278.1	Two 75 mm diameter cobbles encountered at 11.2 m and 11.3 m depth.		1	RC	REC 100%												
11.6			2	RC													

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RQD = 100%
RQD = 100%

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\SMTO\1661607 LEA_5015-E-0049_NE REGION02_DATA\GINTV1661607.GPJ GAL-MISS.GDT 11/8/17 TBAJUL

PROJECT <u>1661607</u>		RECORD OF BOREHOLE No NG-2				2 OF 3 METRIC												
W.P. <u>5312-14-01</u>		LOCATION <u>N 5482075.2; E 252218.0 MTM ZONE 13 (LAT. 49.474243; LONG. -81.725599)</u>				ORIGINATED BY <u>SA</u>												
DIST <u> </u> HWY <u>631</u>		BOREHOLE TYPE <u>Solid Stem Augers, NW Casing and NQ Coring</u>				COMPILED BY <u>AC</u>												
DATUM <u>GEODETIC</u>		DATE <u>June 10 and 11, 2017</u>				CHECKED BY <u>AB</u>												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 (%)					
	--- CONTINUED FROM PREVIOUS PAGE ---																	
	GRANITE (BEDROCK)		2	RC	REC 100%	277											RQD = 100%	
	Bedrock cored from 11.6 m depth to 14.8 m depth. For coring details see Record of Drillhole NG-2.		3	RC	REC 100%	276												RQD = 100%
						275												
274.9 14.8	END OF BOREHOLE Note: 1. Water level at a depth of 2.7 m below ground surface (Elev. 287.0 m) upon completion of drilling.																	

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1661607 LEA_5015-E-0049_NE REGION02_DATA\GINT\1661607.GPJ GAL-MISS.GDT 11/8/17 TBJUL

PROJECT: 1661607
LOCATION: N 5482075.2; E 252218.0
MTM ZONE 13 (LAT. 49.474243; LONG. -81.725599)
INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: NG-2

SHEET 3 OF 3
DATUM: GEODETIC

DRILLING DATE: June 11, 2017
DRILL RIG: CME 55 Truck Mount
DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate										BD - Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage				PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular				PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break				BR - Broken Rock				NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
							RECOVERY		R.Q.D. %	FRACT INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
							TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
							FLUSH	100										100			100	100	100	100	100	100	100	100	100	100	100	100		100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100

DEPTH SCALE


1 : 60



LOGGED: SA
CHECKED: AB

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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>1661607</u>		RECORD OF BOREHOLE No NG-3				2 OF 3 METRIC												
W.P. <u>5312-14-01</u>		LOCATION <u>N 5482101.1; E 252243.1 MTM ZONE 13 (LAT. 49.474478; LONG. -81.725256)</u>				ORIGINATED BY <u>MR</u>												
DIST <u> </u> HWY <u>631</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing and NQ Coring</u>				COMPILED BY <u>AC</u>												
DATUM <u>GEODETIC</u>		DATE <u>June 10, 2017</u>				CHECKED BY <u>AC/AB</u>												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 (%)					
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>						
275.1	GRANITE (BEDROCK) Bedrock cored from 11.1 m depth to 14.5 m depth. For coring details see Record of Drillhole NG-3.		1	RC														RQD = 100%
14.5			2	RC	REC 100%	277												RQD = 100%
			3	RC	REC 100%	276												
	END OF BOREHOLE Note: 1. Water level 2.4 m below ground surface (Elev. 287.2 m) inside NW casing prior seating casing into bedrock.																	

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PROJECT: 1661607
LOCATION: N 5482101.1; E 252243.1
MTM ZONE 13 (LAT. 49.474478; LONG. -81.725256)
INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: NG-3

SHEET 3 OF 3
DATUM: GEODETIC

DRILLING DATE: June 10, 2017
DRILL RIG: LC CME 55 Track Mount
DRILLING CONTRACTOR: Gerge Downing Estate Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate										BD - Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage										PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular										PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break										BR - Broken Rock										NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
							RECOVERY										R.Q.D. %	FRACT INDEX METRES	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY										Diametral Point Load Index (MPa)	RMC -Q- AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							FLUSH	TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRACT INDEX METRES	TYPE AND SURFACE DESCRIPTION										Jr	Ja	Jn	k, cm/s	10 ⁻¹	10 ⁻²	10 ⁻³	10 ⁻⁴	10 ⁻⁵																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
												10 ⁻¹	10 ⁻²	10 ⁻³	10 ⁻⁴	10 ⁻⁵			10 ⁻⁶	10 ⁻⁷	10 ⁻⁸										10 ⁻⁹	10 ⁻¹⁰	10 ⁻¹¹	10 ⁻¹²	10 ⁻¹³	10 ⁻¹⁴	10 ⁻¹⁵	10 ⁻¹⁶			10 ⁻¹⁷	10 ⁻¹⁸	10 ⁻¹⁹	10 ⁻²⁰																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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DEPTH SCALE
1 : 60



LOGGED: MR
CHECKED: AC/AB

RECORD OF BOREHOLE No NG-4

1 OF 1 **METRIC**

PROJECT 1661607
W.P. 5312-14-01 LOCATION N 5482112.6; E 252253.5 MTM ZONE 13 (LAT. 49.474583; LONG. -81.725114) ORIGINATED BY SA
DIST HWY 631 BOREHOLE TYPE NW Casing and Wash Boring COMPILED BY AC
DATUM GEODETIC DATE June 12, 2017 CHECKED BY AB

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					w _p	w	w _L		
289.6	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALT (35 mm)																
0.3	RAP (110 mm)																
	Gravelly sand (FILL) Brown Moist																
	Sand, trace gravel, some silt (FILL) Dense to very dense Brown Moist		1	SS	55		289										
			2	SS	31		288						○				3 79 (18)
287.3																	
2.3	SAND, trace gravel Loose to compact brown Wet		3	SS	16		287										
			4	SS	9		286										
			5	SS	16		285						○				45 48 (7)
			6	SS	7		284										
							283						○				18 54 24 4
							282										
283.4	Silty SAND, some gravel, trace clay (TILL) Dense to very dense Grey Wet		7A	SS	45		281										
6.3			7B				280										
			8	SS	67												
			9	SS	44												
279.8	END OF BOREHOLE																
9.8	Note: 1. Water level at a depth of 3.0 m below ground surface (Elev. 286.6 m) in open borehole 20 minutes after completion of drilling.																

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE


SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1661607 LEA_5015-E-0049_NE REGION\02_DATA\GINT\1661607.GPJ GAL-MISS.GDT 11/8/17 TB\JUL

PROJECT 1661607		RECORD OF BOREHOLE No NG-5				1 OF 3 METRIC																																					
W.P. 5312-14-01		LOCATION N 5482064.7; E 252222.7 MTM ZONE 13 (LAT. 49.474149; LONG. -81.725532)				ORIGINATED BY MR																																					
DIST _____ HWY 631		BOREHOLE TYPE NW Casing and NQ Coring				COMPILED BY AC																																					
DATUM GEODETIC		DATE August 18, 2017				CHECKED BY AB																																					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)																											
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)																										
287.8	GROUND SURFACE							20	40	60	80	100																															
0.0	SAND, trace to some gravel, trace silt Loose to compact Brown to grey Wet		1	SS	4												5 91 (4)																										
			2	SS	4																																						
			3	SS	18												18 75 (7)																										
			4	SS	13																																						
			5	SS	7																																						
			6	SS	7												3 94 (3)																										
283.3	4.5		7	SS	51/0.15																																						
	Gravelly Silty SAND, trace clay (TILL) Dense to very dense Grey Wet Cobbles and boulders were encountered at the following depths and sizes: <table border="1" style="font-size: small;"> <thead> <tr> <th>Depth (m)</th> <th>Size (mm)</th> </tr> </thead> <tbody> <tr><td>5.7</td><td>75</td></tr> <tr><td>5.8</td><td>75</td></tr> <tr><td>5.9</td><td>105</td></tr> <tr><td>6.0</td><td>120</td></tr> <tr><td>6.1</td><td>460</td></tr> <tr><td>6.6</td><td>120</td></tr> <tr><td>6.7</td><td>120</td></tr> <tr><td>6.8</td><td>150</td></tr> <tr><td>9.8</td><td>460</td></tr> <tr><td>10.0</td><td>200</td></tr> <tr><td>10.3</td><td>90</td></tr> <tr><td>10.4</td><td>105</td></tr> </tbody> </table> No recovery in Samples 8 and 9 after 2 attempts.	Depth (m)	Size (mm)	5.7	75	5.8	75	5.9	105	6.0	120	6.1	460	6.6	120	6.7	120	6.8	150	9.8	460	10.0	200	10.3	90	10.4	105		-	RC	REC 42%												
Depth (m)	Size (mm)																																										
5.7	75																																										
5.8	75																																										
5.9	105																																										
6.0	120																																										
6.1	460																																										
6.6	120																																										
6.7	120																																										
6.8	150																																										
9.8	460																																										
10.0	200																																										
10.3	90																																										
10.4	105																																										
			-	RC	REC 72%																																						
			8	SS	41																																						
			-	RC	REC 17%																																						
			9	SS	50																																						
			-	RC	REC 88%																																						
277.4	10.4		1	RC	REC 100%												RQD = 90%																										
	GRANITE (BEDROCK) Bedrock cored from 10.4 m depth to 13.8 m depth. For coring details see Record of Drillhole NG-5.		2	RC	REC 100%												RQD = 96%																										

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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PROJECT <u>1661607</u>		RECORD OF BOREHOLE No NG-5				2 OF 3 METRIC												
W.P. <u>5312-14-01</u>		LOCATION <u>N 5482064.7; E 252222.7 MTM ZONE 13 (LAT. 49.474149; LONG. -81.725532)</u>				ORIGINATED BY <u>MR</u>												
DIST <u> </u> HWY <u>631</u>		BOREHOLE TYPE <u>NW Casing and NQ Coring</u>				COMPILED BY <u>AC</u>												
DATUM <u>GEODETIC</u>		DATE <u>August 18, 2017</u>				CHECKED BY <u>AB</u>												
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					W _p W W _L				WATER CONTENT (%)	
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60						
	GRANITE (BEDROCK)		2	RC	REC 100%													RQD = 96%
	Bedrock cored from 10.4 m depth to 13.8 m depth. For coring details see Record of Drillhole NG-5.		3	RC	REC 100%													RQD = 100%
274.0 13.8	END OF BOREHOLE Note: 1. Water level at a depth of 0.6 m below ground surface (Elev. 287.2 m) upon completion of drilling.																	


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PROJECT: 1661607
LOCATION: N 5482064.7; E 252222.7
MTM ZONE 13 (LAT. 49.474149; LONG. -81.725532)
INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: NG-5

SHEET 3 OF 3
DATUM: GEODETIC

DRILLING DATE: August 18, 2017
DRILL RIG: Boart Longyear LF-70 DD
DRILLING CONTRACTOR: George Downing Estate Drilling Ltd

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION	
								FLUSH	TOTAL CORE %			SOLID CORE %	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s				
	80 60 40 20 0	80 60 40 20 0							10 ⁻⁹ 10 ⁻⁸ 10 ⁻⁷ 10 ⁻⁶ 10 ⁻⁵														
	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate	BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage						PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular	PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break			BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.										
		REFER TO PREVIOUS PAGE		277.4																			
11	NW Boart Longyear LF-70 DD NQ CORING	GRANITE Strong Fresh Fine to medium grained Grey to black		10.4	1	GREY	100%																
12				2	GREY	100%																	UCS = 94 MPa
13				3	GREY	100%																	
14	END OF DRILLHOLE			274.0																			
15																							
16																							
17																							
18																							
19																							
20																							
21																							
22																							

DEPTH SCALE
1 : 60



LOGGED: MR
CHECKED: AB

PROJECT 1661607		RECORD OF BOREHOLE No NG-6				1 OF 3 METRIC										
W.P. 5312-14-01		LOCATION N 5482094.4; E 252252.8 MTM ZONE 13 (LAT. 49.474419; LONG. -81.725121)				ORIGINATED BY MR										
DIST HWY 631		BOREHOLE TYPE NW Casing and NQ Coring				COMPILED BY AC										
DATUM GEODETIC		DATE August 17, 2017				CHECKED BY AB										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT 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 (%)				
286.9	GROUND SURFACE						20	40	60	80	100	20	40	60		
0.0	SAND, trace to some gravel, trace organics Very loose to loose Brown to grey Wet		1	SS	5											
			2	SS	4										11	85 (4)
			3	SS	1											
284.7																
2.2	Gravelly Silty SAND to Silty SAND and GRAVEL, trace clay (TILL) Very dense Grey Wet 90 mm and 140 mm diameter cobbles were encountered at 3.7 and 3.8 m depths, respectively		4	SS	108											
			5	SS	90										37	38 20 5
			-	RC	REC 42%											
			6	SS	52											
			7	SS	73										28	44 24 4
			8	SS	71											
278.5																
8.4	GRANITE (BEDROCK) Bedrock cored from 8.4 m depth to 11.5 m depth. For coring details see Record of Drillhole NG-6.		1	RC	REC 100%											RQD = 95%
			2	RC	REC 100%											RQD = 100%
275.4																
11.5																

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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PROJECT <u>1661607</u>		RECORD OF BOREHOLE No NG-6		2 OF 3 METRIC	
W.P. <u>5312-14-01</u>		LOCATION <u>N 5482094.4; E 252252.8 MTM ZONE 13 (LAT. 49.474419; LONG. -81.725121)</u>		ORIGINATED BY <u>MR</u>	
DIST <u> </u> HWY <u>631</u>		BOREHOLE TYPE <u>NW Casing and NQ Coring</u>		COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 17, 2017</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	20	40	60					
	--- CONTINUED FROM PREVIOUS PAGE --- END OF BOREHOLE Note: 1. Water level at ground surface (Elev. 286.9 m) upon completion of drilling.																			

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PROJECT: 1661607
LOCATION: N 5482094.4; E 252252.8
MTM ZONE 13 (LAT. 49.474419; LONG. -81.725121)
INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: NG-6

SHEET 3 OF 3
DATUM: GEODETIC

DRILLING DATE: August 17, 2017
DRILL RIG: Boart Longyear LF-70 DD
DRILLING CONTRACTOR: George Downing Estate Drilling Ltd

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	CORING LOG														NOTES WATER LEVELS INSTRUMENTATION	
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)		RMC -Q' AVG.
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s				
																			JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate			
NOTE: For additional abbreviations refer to list of abbreviations & symbols.																						
9	Boart Longyear LF-70 DD NQ CORING	NW	REFER TO PREVIOUS PAGE		278.5																	
10		GRANITE Strong Fresh Fine to medium grained Grey to black Schist zone from 9.0 m to 9.1 m depth.		8.4	1	GREY	100%												UCS = 87 MPa			
11																						
12			END OF DRILLHOLE		275.4																	
13																						
14																						
15																						
16																						
17																						
18																						
19																						
20																						

DEPTH SCALE
1 : 60



LOGGED: MR
CHECKED: AB

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PROJECT 1661607		RECORD OF BOREHOLE No NG-7		1 OF 3 METRIC	
W.P. 5312-14-01		LOCATION N 5482078.4; E 252214.2 MTM ZONE 13 (LAT. 49.474272; LONG. -81.725651)		ORIGINATED BY SA	
DIST _____ HWY 631		BOREHOLE TYPE Solid Stem Augers, NW Casing and NQ Coring		COMPILED BY AC	
DATUM GEODETIC		DATE May 29, 2017		CHECKED BY AB	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	w _p		
289.8	GROUND SURFACE													GR SA SI CL
0.0	ASPHALT (50 mm)													
0.3	Gravelly sand (FILL)													
	ASPHALT (50 mm)													
	Sand, trace to some gravel, trace to some silt (FILL) Loose to compact Brown Moist to wet		1	SS	22									
			2	SS	15									9 78 (13)
			3	SS	9									
286.8														
3.0	SAND, trace to some gravel, trace silt Loose to compact Brown Wet Trace organics noted in Sample 4.		4	SS	8									14 79 (7)
			5	SS	12									
			6	SS	8									
			7	SS	7									5 92 (3)
282.6														
7.2	Sandy CLAYEY SILT, trace gravel Loose to compact Grey Wet													
282.0			8A											6 20 66 8
7.8	Gravelly Silty SAND to SAND and GRAVEL (TILL) Dense Grey Wet Cobbles less than 100 mm encountered between 7.6 m and 10.7 m depth.		8B	SS	43									
			9	SS	30									
			10	SS	49									36 51 13 0
278.2														
11.6			1	RC	REC 100%									RQD = 100%

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+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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PROJECT <u>1661607</u>		RECORD OF BOREHOLE No NG-7				2 OF 3 METRIC											
W.P. <u>5312-14-01</u>		LOCATION <u>N 5482078.4; E 252214.2 MTM ZONE 13 (LAT. 49.474272; LONG. -81.725651)</u>				ORIGINATED BY <u>SA</u>											
DIST <u> </u> HWY <u>631</u>		BOREHOLE TYPE <u>Solid Stem Augers, NW Casing and NQ Coring</u>				COMPILED BY <u>AC</u>											
DATUM <u>GEODETIC</u>		DATE <u>May 29, 2017</u>				CHECKED BY <u>AB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 (%)				
--- CONTINUED FROM PREVIOUS PAGE ---																	
	GRANITE (BEDROCK)		2	RC	REC 100%												RQD = 100%
	Bedrock cored from 11.6 m depth to 14.6 m depth. For coring details see Record of Drillhole NG-7.		3	RC	REC 100%												
275.2	END OF BOREHOLE																
14.6	Note: 1. Water level at a depth of 2.7 m below ground surface (Elev. 287.1 m) upon completion of drilling.																

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PROJECT: 1661607
LOCATION: N 5482078.4; E 252214.2
MTM ZONE 13 (LAT. 49.474272; LONG. -81.725651)
INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: NG-7

SHEET 3 OF 3
DATUM: GEODETIC

DRILLING DATE: May 29, 2017
DRILL RIG: CME 55 Truck Mount
DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate										BD - Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage				PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular				PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break				BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.				NOTES WATER LEVELS INSTRUMENTATION
							RECOVERY		R.Q.D. %	FRACT INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG.													
							TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s																
							FLUSH	80 80																									

DEPTH SCALE

1 : 60



LOGGED: SA

CHECKED: AB

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PROJECT 1661607		RECORD OF BOREHOLE No NG-8		1 OF 3 METRIC	
W.P. 5312-14-01		LOCATION N 5482104.2; E 252239.8 MTM ZONE 13 (LAT. 49.474506; LONG. -81.725302)		ORIGINATED BY MR	
DIST _____ HWY 631		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing and NQ Coring		COMPILED BY AC	
DATUM GEODETIC		DATE May 29, 2017		CHECKED BY AB	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							W _P	W	W _L		
289.6	GROUND SURFACE							20	40	60	80	100				GR	SA	SI	CL
0.0	ASPHALT (50 mm)																		
289.3	Gravelly sand (FILL)																		
0.3	Sand, trace silt, trace asphalt (FILL) Loose to compact Brown to grey Wet																		
			1	SS	14														
			2	SS	21														
			3	SS	5														
			4	SS	5														
	Trace organics at 3.8 m depth.		5	SS	5														
285.1																			
4.5	ORGANIC Silty SAND, trace wood Very loose Black Wet		6	SS	2														
284.0																			
5.6	SILT and SAND, trace clay, trace gravel, trace organics Very loose Grey Wet		7A	SS	2														
283.1			7B																
6.5	SAND and GRAVEL, trace to some silt to Gravelly Silty SAND (TILL) Compact to dense Grey Wet A 125 mm cobble encountered at 6.6 m below ground surface. A 250 mm cobble encountered at 7.1 m below ground surface.																		
			8	SS	29														
			9	SS	18														
			10	SS	32/0.25														
278.6																			
11.0	GRANITE (BEDROCK)																		
	Bedrock cored from 11.0 m depth to 14.5 m depth. For coring details see Record of Drillhole NG-8.		1	RC	REC 100%														

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1661607 LEA_5015-E-0049_NE REGION02_DATAGINT\1661607.GPJ GAL-MISS.GDT 11/8/17 TB\JUL

PROJECT <u>1661607</u>		RECORD OF BOREHOLE No NG-8				2 OF 3 METRIC											
W.P. <u>5312-14-01</u>		LOCATION <u>N 5482104.2; E 252239.8 MTM ZONE 13 (LAT. 49.474506; LONG. -81.725302)</u>				ORIGINATED BY <u>MR</u>											
DIST <u> </u> HWY <u>631</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing and NQ Coring</u>				COMPILED BY <u>AC</u>											
DATUM <u>GEODETIC</u>		DATE <u>May 29, 2017</u>				CHECKED BY <u>AB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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 (%)				
--- CONTINUED FROM PREVIOUS PAGE ---																	
	GRANITE (BEDROCK)		2	RC	REC 100%	277											RQD = 100%
	Bedrock cored from 11.0 m depth to 14.5 m depth. For coring details see Record of Drillhole NG-8.		3	RC	REC 100%	276											
275.1 14.5	END OF BOREHOLE Note: 1. Water level 2.1 m below ground surface (Elev. 287.5 m) inside augers prior to switching NW casing.																


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PROJECT: 1661607
LOCATION: N 5482104.2; E 252239.8
MTM ZONE 13 (LAT. 49.474506; LONG. -81.725302)
INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: NG-8

SHEET 3 OF 3
DATUM: GEODETIC

DRILLING DATE: May 29, 2017
DRILL RIG: CME 55 Truck Mount
DRILLING CONTRACTOR: George Downing Estate Drilling Ltd

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY			Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
								FLUSH	TOTAL CORE %			SOLID CORE %	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k, cm/s				
11		REFER TO PREVIOUS PAGE		278.6																	
	NW LC CME 55 Truck Mount NQ Coring	GRANITE Very strong Fresh Fine to medium grained Light grey		11.0	1	GREY	100%														UCS = 118 MPa
12					2	GREY	100%														
13					3	GREY	100%														
14		END OF DRILLHOLE		275.1																	
15				14.5																	
16																					
17																					
18																					
19																					
20																					
21																					
22																					
23																					

DEPTH SCALE
1 : 60



LOGGED: MR
CHECKED: AB

APPENDIX B

Laboratory Test Results

Table B1 - Summary of Analytical Testing of Soil Sample

Parameter	Units	South Abutment (Borehole NG-7)	North Abutment (Borehole NG-8)
Resistivity	ohm-cm	3,900	3,200
Conductivity	µmho/cm	257	314
pH	pH	8.12	7.05
Sulphate	µg/g	Not Detected	Not Detected
Chloride	µg/g	81	43

Notes:

Reviewed by: AB

1. Sample obtained 29 May 2017.

2. Analytical testing carried out by Maxxam Analytics Inc.

Golder Associates Ltd.

33 Mackenzie Street, Suite 100
 Sudbury, Ontario, Canada P3C 4Y1
 Telephone: (705) 524-6861
 Fax: (705) 524-1984



TABLE B2 - SUMMARY OF ROCK CORE TEST DATA

PROJECT NO.: 1661607

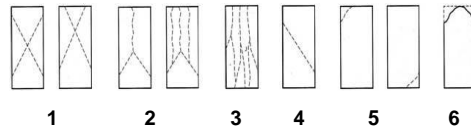
JOB NAME: Nagagamisis Narrows Bridge

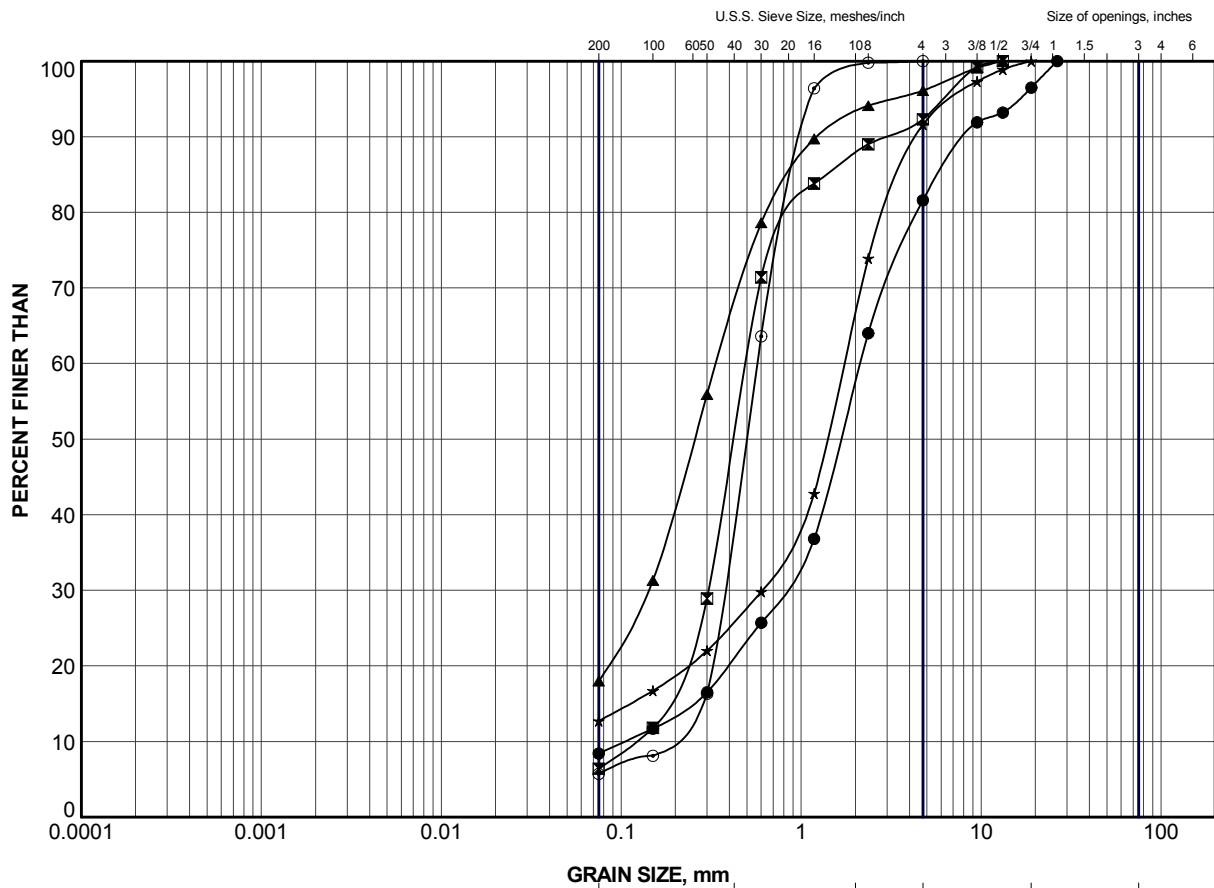
TYPE OF UNIT: Bedrock Core

BOREHOLE	NG-2	NG-3	NG-5	NG-6	NG-7	NG-8
GOLDER LAB #	C1012	C1017	C1528	C1532	C647	C641
DATE TESTED	Jul. 25, 2017	Jul. 25, 2017	Sept. 19, 2017	Sept. 19, 2017	Jun. 9, 2017	Jun. 9, 2017
TESTED BY	JM/DM	JM/DM	JP	JP	EHS	EHS
DEPTH OF TESTED CORE (m)	13.1	11.3	11.9	9.8	11.9	12.5
LENGTH (mm)	100.9	97.5	100.3	101.0	95.8	96.1
DIAMETER (mm)	47.0	47.3	47.0	47.0	47.1	47.6
DENSITY (kg/m3)	2625	2625	2585	2568	2661	2554
COMPRESSIVE STRENGTH (MPa)	145.6	128.0	94.0	86.7	180.1	117.9
TYPE OF FRACTURE	1	1	2	3	1	1

Checked by : AB

Type of Fracture





LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NG-1	2	288.0
⊠	NG-3	3	287.0
▲	NG-4	2	287.8
★	NG-7	2	288.0
⊙	NG-8	3	287.0

PROJECT

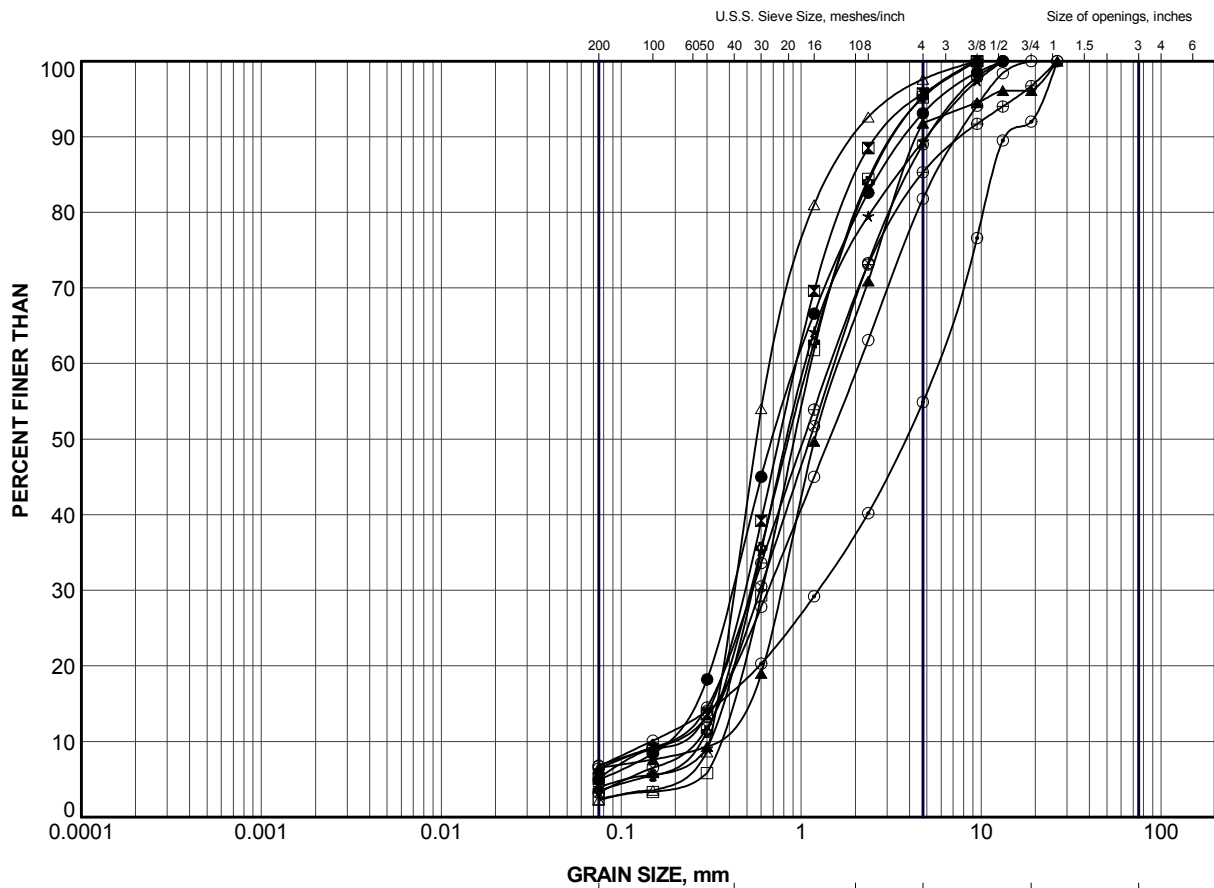
HIGHWAY 631
NAGAGAMISIS NARROWS BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION
SAND (FILL)



PROJECT No.		1661607	FILE No.		1661607.GPJ
DRAWN	TB	Sep 2017	SCALE	N/A	REV.
CHECK	AB	Sep 2017	FIGURE B1		
APPR		Sep 2017			



CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NG-1	5	285.7
⊠	NG-1	8	281.9
▲	NG-2	3	287.1
★	NG-2	6	284.8
⊙	NG-4	5	285.5
⊕	NG-5	1	287.5
○	NG-5	3	286.0
△	NG-5	6	283.7
⊗	NG-6	2	285.8
⊕	NG-7	4	286.5
□	NG-7	7	283.4

PROJECT

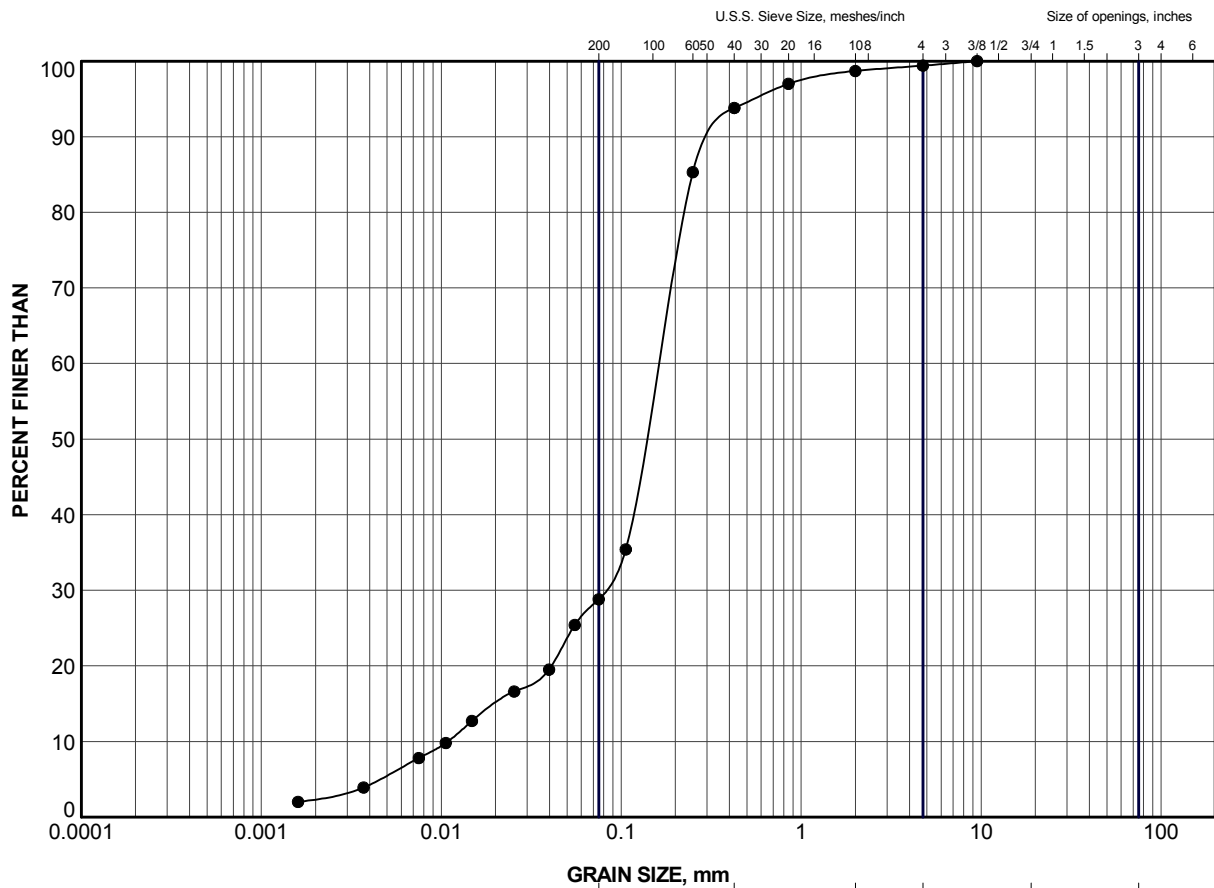
HIGHWAY 631
NAGAGAMISIS NARROWS BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION
SAND (and SAND and GRAVEL LAYER)




PROJECT No.		1661607	FILE No.		1661607.GPJ
DRAWN	TB	Oct 2017	SCALE	N/A	REV.
CHECK	AB	Oct 2017	FIGURE B3		
APPR		Oct 2017			

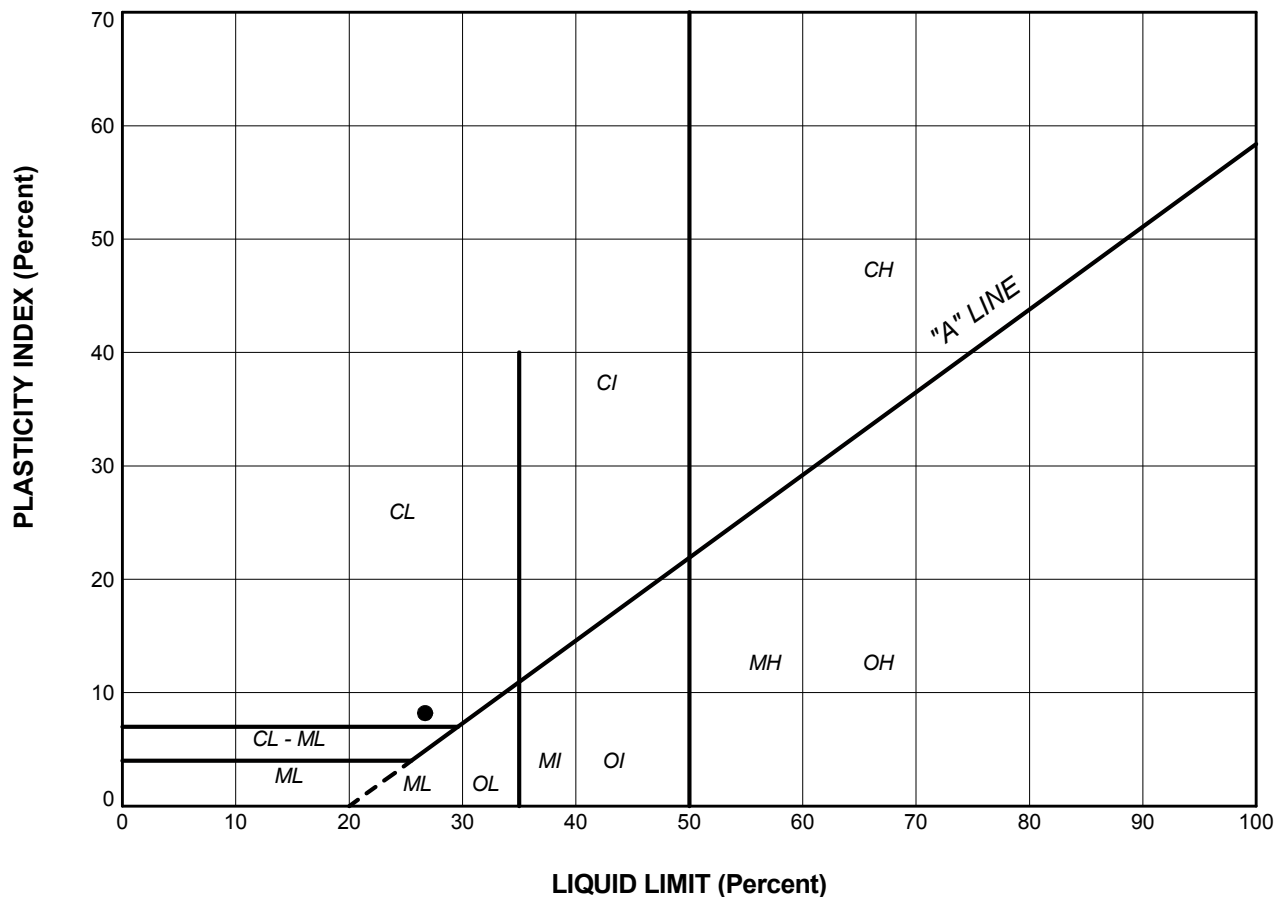


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NG-8	7A	283.3

PROJECT					
HIGHWAY 631 NAGAGAMISIS NARROWS BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION SILT and SAND					
PROJECT No.		1661607		FILE No.	
DRAWN		TB		Sep 2017	
CHECK		AB		Sep 2017	
APPR				Sep 2017	
SCALE		N/A		REV.	
 Golder Associates SUDBURY, ONTARIO				FIGURE B4	

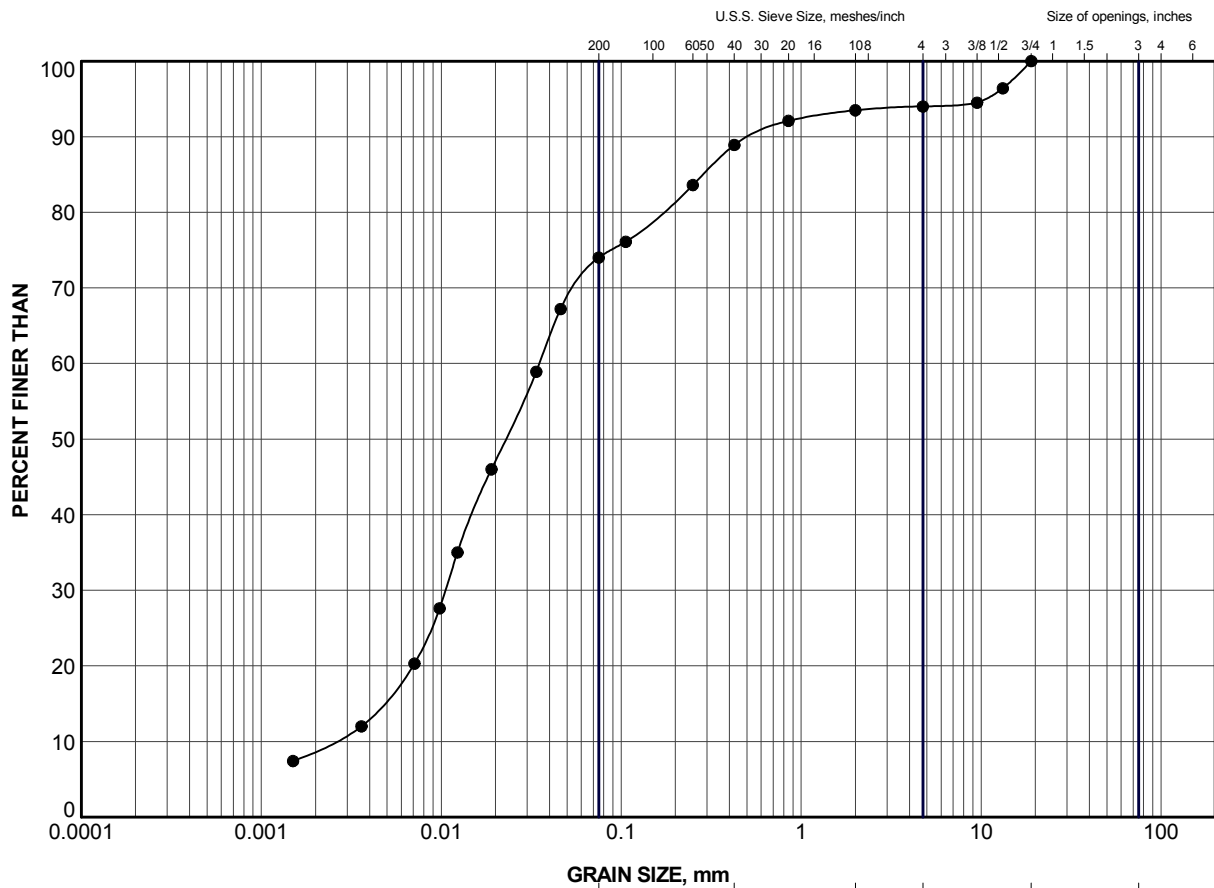


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	NG-7	8A	26.7	18.5	8.2

PROJECT					
HIGHWAY 631 NAGAGAMISIS NARROWS BRIDGE					
TITLE					
PLASTICITY CHART SANDY CLAYEY SILT					
PROJECT No.		1661607		FILE No.	
DRAWN		TB		Oct 2017	
CHECK		AB		Oct 2017	
APPR				Oct 2017	
SCALE		N/A		REV.	
FIGURE		B5			




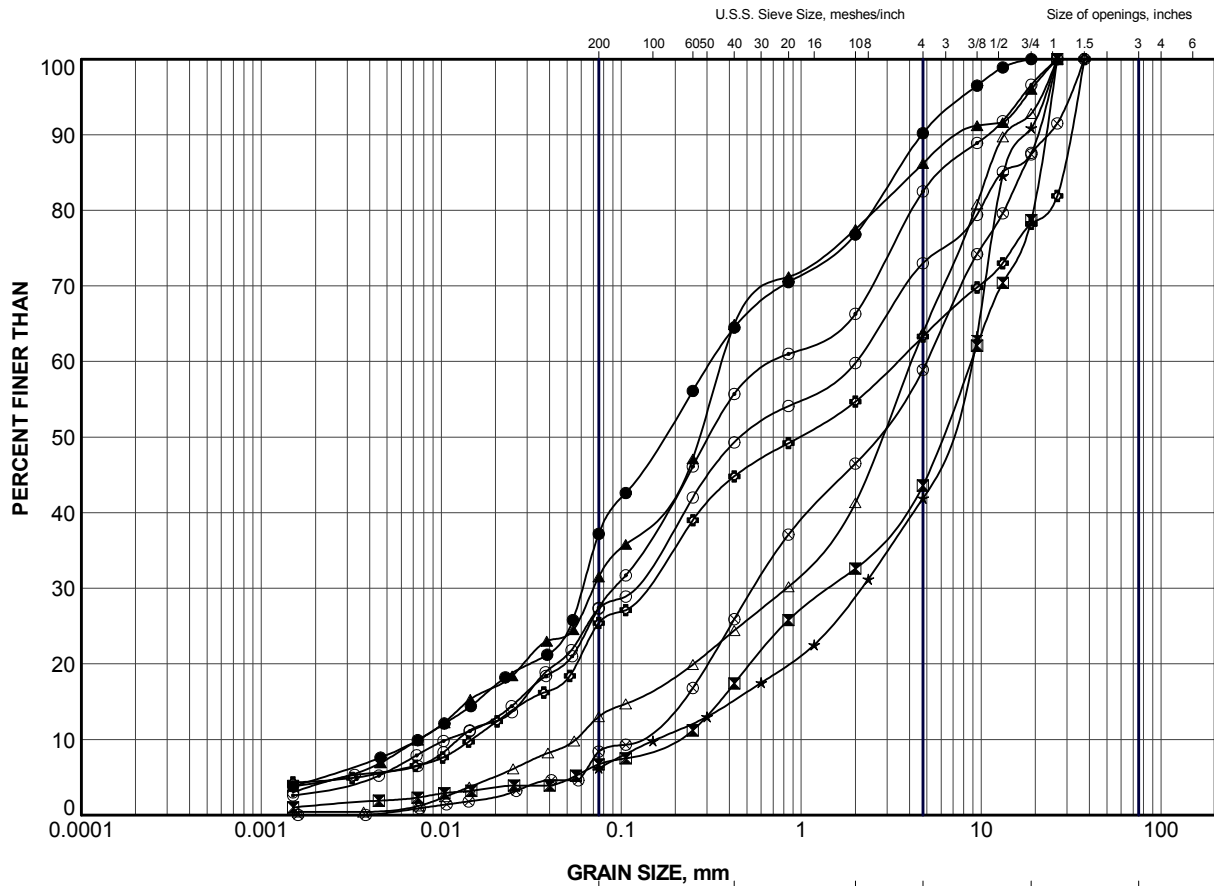


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NG-7	8A	282.1

PROJECT						HIGHWAY 631 NAGAGAMISIS NARROWS BRIDGE					
TITLE						GRAIN SIZE DISTRIBUTION SANDY CLAYEY SILT					
PROJECT No.			1661607			FILE No.			1661607.GPJ		
DRAWN	TB	Oct 2017	SCALE	N/A	REV.						
CHECK	AB	Oct 2017									
APPR		Oct 2017									
 Golder Associates SUDBURY, ONTARIO						FIGURE B6					



CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NG-1	9	280.4
⊠	NG-2	8	281.8
▲	NG-3	8	281.7
★	NG-3	10	278.8
⊙	NG-4	7B	283.1
⊕	NG-6	5	283.6
○	NG-6	7	280.5
△	NG-7	10	278.8
⊗	NG-8	10	278.7

PROJECT

HIGHWAY 631
NAGAGAMISIS NARROWS BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION
SILTY SAND to SANDY GRAVEL (TILL)



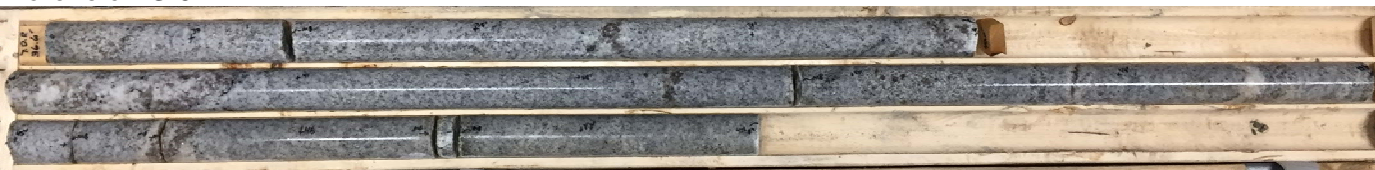
PROJECT No.		1661607	FILE No.		1661607.GPJ
DRAWN	TB	Sep 2017	SCALE	N/A	REV.
CHECK	AB	Sep 2017	FIGURE B7		
APPR		Sep 2017			

Borehole NG-2



Box 1: 11.6 m – 14.8 m

Borehole NG-3



Box 1: 11.1 m – 14.5 m

Borehole NG-5



Box 1&2: 10.4 m – 13.8 m

Borehole NG-6



Box 1&2: 8.4 m – 11.5 m

Borehole NG-7



Box 1: 11.6 m – 14.6 m

Borehole NG-8



Box 1: 11.0 m – 14.5 m

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT

**Highway 631
Nagagamisis Narrows Bridge**

TITLE

Bedrock Core Photographs



**Golder
Associates**

PROJECT No. 1661607

FILE No. ---

DESIGN

TB

SEP 17

SCALE

NTS

REV.

CADD

--

SEP 17

CHECK

AC

SEP 17

REVIEW

FIGURE B8

APPENDIX C

Non Standard Special Provisions

AMENDMENT TO OPSS 902, NOVEMBER 2019

Special Provision No. DBSP0902

OPSS 902, November 2019, is deleted in its entirety and replaced with the following:

CONSTRUCTION SPECIFICATION FOR EXCAVATING AND BACKFILLING - STRUCTURES

902.01 SCOPE

This specification covers the requirements for excavating and backfilling for structures, including dewatering.

902.02 REFERENCES

This specification refers to the following specifications, standards, or publications:

Design Build Special Provisions

DBSP 0539 Temporary Protection Systems

Ontario Provincial Standard Specifications, Construction

OPSS 206	Grading
OPSS 501	Compacting
OPSS 510	Removal
OPSS 517	Dewatering
OPSS 805	Temporary Erosion and Sediment Control Measures

Ontario Provincial Standard Specifications, Material

OPSS 1004	Aggregates - Miscellaneous
OPSS 1010	Aggregates - Base, Subbase, Select Subgrade, and Backfill Material
OPSS 1205	Clay Seal
OPSS 1860	Geotextiles

902.03 DEFINITIONS

For the purpose of this specification, the following definitions apply:

Bedding means granular material to be placed in areas so designated in the Contract Documents.

Channel means a natural or artificial watercourse.

Culvert means a structure that provides an opening through an embankment and to which roadway loads are distributed through fill or that is designated as a culvert in the Contract Documents.

Design Engineer means the Engineer retained by the Contractor who has sealed and signed the Issued for Construction Drawings and/or Working Drawings required to complete all or part of the work specified in the contract.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Earth means all soils except those defined as rock, and excludes stone masonry, concrete and other manufactured materials.

Foundation means that portion of the ground below the structure base or footings that supports the structure or that portion of the ground supporting the pile caps.

Groundwater Control System means as defined in OPSS 517.

Rock means natural beds or massive fragments, of the hard, stable, cemented part of the earth's crust, igneous, metamorphic, or sedimentary in origin, which may or may not be weathered, and includes boulders having a volume of 1 cubic m or greater.

Sediment means as defined in OPSS 517.

Soil means all loose or moderately cohesive organic or inorganic deposits of the earth's crust such as silt, sand, gravel, or clay or any of their mixtures.

Structure means any bridge, tunnel, concrete culvert, retaining wall, dock, guide way, or sign support.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Where dewatering is required, the Contractor shall be responsible for the design of the dewatering system for the intended purpose.

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a [* Designer Fill-In, See Notes to Designer] year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within [100] metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.05 MATERIALS

902.05.01 Granular

Granular material to be used for backfill, bedding, and frost tapers shall be according to OPSS 1010.

The 19.0 mm clear stone to be used for wall drains shall be according to OPSS 1004.

902.05.02 Native Backfill

Native and imported material shall be approved by the design Engineer. All material shall be free from frozen lumps, cinders, ashes, refuse, vegetable or organic matter, rocks and boulders over 150 mm in any dimension, and other deleterious material.

902.05.03 Clay Seal

Clay seal shall be according to OPSS 1205.

902.05.04 Geotextile

Geotextile shall be according to OPSS be and be of the type, class, and filtration opening size (FOS) range specified in the Contract Documents.

902.06 EQUIPMENT

902.06.01 Compaction Equipment

Compaction equipment shall be according to OPSS 501.

902.07 CONSTRUCTION

902.07.01 Removals

Removals shall be according to OPSS 510.

902.07.02 Removal of Ice and Snow

All ice and snow shall be removed from all portions of the work area before any excavation and backfill operations proceed. Frozen materials shall not be incorporated into the work. Material shall not be placed over frozen ground, ice, or snow.

902.07.03 Protection Systems

Protection systems shall be according to DBSP 0539.

Protection systems shall be installed:

- a) Where the stability, safety, or function of an existing structure, roadway, railway, or other facility can be impaired by an excavation or temporary slope.
- b) To permit excavation where there is a necessity to retain the sidewalls of an excavation and to permit dewatering by restricting water flow and facilitating safe execution of the work.

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any watercourse, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

902.07.05 Excavation

902.07.05.01 General

Deterioration of the foundation soil or rock, surface water from entering and eroding the face of the excavation, and build up of hydrostatic pressures that may have harmful effects upon the temporary or permanent structures shall be prevented.

902.07.05.02 Excavation for Foundations

Excavation for footings, working slabs, and granular pads shall be to the neat lines specified in the Contract Documents.

The bottom of the excavation on which the footing, working slab, or granular pad is to rest shall not be disturbed. In soft conditions, construction of the footing or structural slab shall commence immediately after the final removal of material to the foundation level has been completed.

In the case of concrete culverts of the open footing type, no excavation shall be made between the footings below the level of the stream bed or the top of the footings, whichever elevation is lower, unless authorized in writing by the design Engineer and a Notice to Proceed has been issued by the Contract Administrator.

The elevation of the bottom of the footing, working slab, or granular pad shall not be changed without the approval of the design Engineer and a Notice to Proceed has been issued by the Contract Administrator.

Over excavated areas beyond the excavation limits shall be restored to their original conditions at the Contractor's expense. Over excavated areas shall be backfilled with a material suitable for the particular

application and approved by the design Engineer. Concrete fill shall be used for over excavation in rock. Where material other than concrete is used, the material shall be compacted to the dry density specified in OPSS 501.

Any additional excavation, not anticipated in the original design, shall be justified by the design Engineer in writing for the Contract Administrator's review and approval. A Request to Proceed with the additional excavation is required and the additional excavation shall not start until a Notice to Proceed is issued by the Contract Administrator.

902.07.05.03 Excavation for Backfill and Frost Tapers

Excavation for backfill and frost tapers shall be to the neat lines specified in the Contract Documents.

Over excavated areas shall be restored with material approved by the design Engineer. The material shall be compacted to the dry density specified in OPSS 501.

902.07.05.04 Preservation of Channel

Where a channel cross-section is altered, it shall be restored to its original condition.

902.07.06 Backfilling

902.07.06.01 General

Frozen materials shall not be incorporated into the work.

Footings shall be protected against frost action.

Other than the backfill placed to the tops of the footings, no fill shall be placed against an abutment, wing wall, retaining wall, or concrete culvert until the concrete has reached 70% of its design strength.

Backfilling around culverts, arches, rigid frames, and piers shall proceed simultaneously and evenly on both sides of the structure. The differential in surface elevation of the backfill material on each side of the structure and individual component shall not be greater than 500 mm.

All voids around abutments, piers, or other permanent work shall be backfilled to the level of the surrounding ground or to the grade specified in the Contract Documents, whichever is the lower before the general backfilling commences.

When rock fill is to be placed around a structure the structure shall be protected to prevent damage from the rock fill.

The minimum height of fill specified in the Contract Documents shall be placed before traffic or construction equipment shall pass over the culvert.

Granular material shall be placed within the lines and grades specified in the Contract Documents.

Wall drains shall have a 0.05 m³ pocket of 19 mm minimum size clear stone wrapped in geotextile placed over the opening of the wall drain at the backfilled side of the wall.

902.07.06.02 Compaction

Backfill shall be placed according to OPSS 206, except the Modified Layer Compaction Method shall not apply, and compacted according to OPSS 501.

Only hand operated vibratory type compaction equipment shall be used for compaction of fill material within the restricted zone behind all earth retaining structures.

902.07.07 Clay Seal

When specified in the Contract Documents, clay seals shall be placed to the dimensions shown.

902.07.08 Inspection for Dewatering, Excavation and Backfilling

A Request to Proceed shall be submitted to the Contract Administrator prior to the commencement of dewatering of the excavation for the structure and completion of the excavation for the foundation.

The next operation after the completion of the excavation for the foundation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

A Request to Proceed shall be submitted to the Contract Administrator upon completion of the excavation for the structure and frost tapers and prior to the commencement of backfilling of excavation.

The commencement of backfilling of excavation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

902.07.09 Management of Excess Material

Management of excess material shall be according to the Contract Documents.

902.08 QUALITY ASSURANCE - Not Used

902.09 MEASUREMENT OF PAYMENT - Not Used

902.10 BASIS OF PAYMENT - Not Used

NOTES TO DESIGNER:

- * Fill in the design storm return period according to MTO Drainage Design Standard TW-1.
- ** Fill in the preconstruction survey distance as recommended by the MTO foundation engineer.

WARRANT: In Design-Build contracts with Excavation, and Backfilling of Structures, including dewatering.

CUSTODIAN: Tony Sangiuliano, Senior Foundation Engineer, Foundations Section, Structural Standards and Specifications Office and Felipe Mendoza, Senior Contract Innovations Analyst, Special Planning Initiatives Office.

AMENDMENT TO OPSS 903, APRIL 2016

Special Provision No. DBSP0903

OPSS 903, April 2016, is deleted in its entirety and replaced with the following:

CONSTRUCTION SPECIFICATION FOR DEEP FOUNDATIONS

903.01 SCOPE

This specification covers the requirements for the supply and installation of deep foundation units.

903.02 REFERENCES

This specification refers to the following specifications, standards, or publications:

Design-Build Special Provisions

DBSP 0904	Concrete Structures
DBSP 0909	Prestressed Concrete - Precast Girders
DBSP 1350	Concrete – Materials and Production

Ontario Provincial Standard Specifications, Construction

OPSS 904	Concrete Structures
OPSS 905	Steel Reinforcement for Concrete
OPSS 911	Coating Structural Steel Systems

Ontario Provincial Standard Specifications, Material

OPSS 1302	Water
OPSS 1440	Steel Reinforcement for Concrete
OPSS 1350	Concrete – Materials and Production

CSA Standards

G40.20/40.21-13 (R2018)	General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Steel
CAN3-056-1962 (R2006)	Round Timber Piles
O80 Series-08	Wood Preservation
W47.1-09 (R2014)	Certification of Companies for Fusion Welding of Steel
W48-18	Filler Metals and Allied Materials for Metal Arc Welding
W59-18	Welded Steel Construction (Metal Arc Welding)
W178.1-18	Certification of Welding Inspection Organizations
W178.2-18	Certification of Welding Inspectors

Canadian General Standards Board (CGSB)

48.9712-2006	Non-destructive Testing, Qualification and Certification of Personnel
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ASTM International

A 252-98(2007)	Welded and Seamless Steel Pipe Piles
A 328/A 328M-07	Steel Sheet Piling
D 1143/ D 1143M-07	Standard Test Methods for Deep Foundations under Static Axial Compressive Load
D 3689-07	Standard Test Methods for Deep Foundations under Static Axial Tensile Load
D 3966-07	Standard Test Method for Deep Foundations under Lateral Loads

American Petroleum Institute (API)

API 13A	Drilling Fluid Materials, 17th Edition, 10.00.08
RP 13B-1	Standard Procedure for Field Testing Water Based Drilling Fluids, 4th Edition,

Joint Publications of the Society for Protective Coatings (SSPC) and National Association of Corrosion Engineers (NACE)

SSPC-SP6/NACE No. 3-2007	Commercial Blast Cleaning
SP10/NACE No.2	Near-White Blast Cleaning

International Organization for Standardization/International Electrotechnical Commission (ISO/IEC)

17025	General Requirements for the Competence of the Testing and Calibration Laboratories
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903.03 DEFINITIONS

For the purpose of this specification, the following definitions apply:

Anvil means the component of a diesel hammer that acts as an impact block for the ram

Bedrock means a natural solid bed of the hard, stable, cemented part of the earth's crust, igneous, metamorphic, or sedimentary in origin that may or may not be weathered.

Caisson Pile means a cast in place deep foundation unit with or without an enclosing liner formed by placing concrete in a bored or excavated hole.

Cap Block means a material placed on top of the helmet to cushion the blow of the hammer and to attenuate the peak impact energy without causing excessive loss of the impact energy.

Casing means open ended enclosing cylindrical steel tubing or pipe permanently installed in the ground. Casings are structurally required and can be used to stabilize and excavated hole.

Deep Foundation Unit means a structural member, driven or otherwise, installed in the ground to transfer the loads from a structure to soil or rock and derives supporting resistance from the surrounding soil or rock or from the soil or rock strata below its tip or a combination of both.

Design Engineer means the Engineer retained by the Contractor who has sealed and signed the Issued for Construction Drawings and/or Working Drawings required to complete all or part of the work specified in the contract.

Design Engineer's Designee means a foundations Engineer who under the direct supervision of the design Engineer performs monitoring of the deep foundation work specified in the Contract Documents or as required by the design Engineer.

Displacement Caisson Pile means a pile formed in the ground by driving a casing or liner with a concrete plug or an expendable metal plate attached to it and replacing the displaced soil with unreinforced or reinforced concrete.

Driven Pile means one of the following pile types: steel H, tube, or sheet piles; wooden pile; or precast reinforced concrete pile that has been installed by means of a pile driver.

Driving Shoe means reinforcement attached to the bottom of the pile and designed to protect the pile during driving or to penetrate a hard stratum.

Driving to a Set means driving the pile to the requirement that satisfies pile driving criteria correlated to a required pile resistance.

Follower means a removable extension that transmits the hammer blows to the head of the pile.

Helmet means a formed steel cap that fits over the top of a pile head to retain in position a resilient cap block.

Jetting means the use of a jet of water at high pressure directed into the ground below the pile tip to assist its penetration

Liner means open ended enclosing steel tubing or pipe temporarily installed in the ground to facilitate the construction of caisson piles

Pile means a relatively slender structural element that is installed, wholly or partly in the ground by driving, drilling, auguring, jetting, or other means.

Pile Cap means a footing, or some other structural component used to transfer the load to the piles as well as maintaining them in position.

Pile Cushion means a pad of resilient material placed between the helmet and the top of a precast reinforced concrete or wooden pile to minimize damage to the head during driving.

Pile Group means the piles supporting a pile cap.

Pumped Concrete means a method of transporting concrete through hose or pipe by means of positive and continuous pressure.

Ram means the moving or driving part of an air, steam, diesel, or drop pile hammer that delivers an impact blow to an anvil and to the pile.

Retapping means verifying that the specified resistance previously attained has been sustained by imparting appropriate hammer energy to the pile and monitoring pile penetration.

Rock Points means a specially designed steel tip fitted to piles to enable them to be driven into hard, sound sloped bedrock.

Sheet Pile means a pile that is designed to interlock with adjacent piles and form a continuous wall for the purpose of resisting mainly lateral forces and to reduce seepage.

Slurry means a drilling fluid, consisting of water mixed with one or more of various solids or polymers, used to maintain the stability of the side walls and bottom of an excavation.

Tremie means a hopper with a vertical pipe used for placing concrete under water. The foot of the pipe is always submerged in concrete except during commencement of concreting and the upper level of the concrete in the pipe is always above water level.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.01 Design Requirements

903.04.01.01 Concrete

The Contractor is responsible for providing plastic concrete with suitable characteristics for installation. The concrete shall be flowable, non-segregating concrete that does not exhibit rapid slump loss.

903.04.02 Submission Requirements

903.04.02.01 General

All submissions shall bear the seal and signature of the design Engineer experienced in the field of deep foundations.

When welded field splices are used, the Contractor shall submit the welding procedures to the Contract Administrator for the purpose of quality assurance and documentation. The Canadian Welding Bureau shall have approved the welding procedures.

903.04.02.02 Preconstruction Survey

Prior to commencing the work, the Contractor shall submit to the Contract Administrator, a condition survey of property and structures that may be affected by the work. The survey shall include the locations and conditions of adjacent properties; buildings; underground structures; Utility services; and structures, such as walls abutting the site.

903.04.02.03 Materials

903.04.02.03.01 Mill Certificates

The Contractor shall submit to the Contract Administrator at the time of delivery 1 copy of the mill certificates, indicating that the steel meets the requirements for the appropriate standards for H-piles, tube piles, and sheet piles.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificates verified by testing by a Canadian laboratory. The laboratory shall be certified by an organization accredited by the Standards Council of Canada to comply to comply with the requirements of ISO/IEC 17025 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the

specified material requirements. The stamp shall include the appropriate material specification number, the date (i.e., yyyy-mm-dd), and the signature of an authorized officer of the Canadian testing laboratory.

903.04.02.03.02 Concrete

The Contractor shall submit a suitable, site-specific concrete mix design that meets the requirements of the hardened concrete to the Contract Administrator 14 Days prior to construction, for the purpose of quality assurance and documentation.

903.04.02.03.03 Slurry

The Contractor shall submit the following to the Contract Administrator 14 Days prior to construction, for the purpose of quality assurance and documentation:

- a) The type, source, and physical and chemical properties of the bentonite or polymer.
- b) The source of water.
- c) Method of mixing slurry.
- d) The water solids ratio and the mass and volumes of the constituent parts, including any chemical admixtures or physical treatment employed to produce slurry with the required physical properties.
- e) Details of procedure to be used for monitoring the quality of the slurry.
- f) A test report showing the properties of the slurry and certifying that the slurry meets the requirements of API RP 13B-1.
- g) Method of disposal of the slurry.

903.04.02.04 Installation

903.04.02.04.01 Driven Piles

The Contractor shall submit the following to the Contract Administrator 14 Days prior to construction, for the purpose of quality assurance and documentation:

- a) A schedule of work identifying time and sequence of activities. A Request to Proceed to allow for quality assurance oversight by the Contract Administrator.
- b) Type of equipment, anvil, helmet, and hammer details, including the hammer energy assumed by the Contractor, stated potential energy (rated energy) of the hammer, operating efficiency, and weight of ram.
- c) Working Drawings of precast concrete piles showing the pile dimensions, concrete strength, tendon arrangement, working stresses and arrangement of steel reinforcement, schedules, elongation calculations, method and sequence of casting, complete specifications and details of the prestressing steel, and lift anchors and lifting point locations.
- d) The method of maintaining the steel reinforcement cages in position, when steel reinforcement cages are used in tube piles.

- e) Procedure for monitoring pile installation.
- f) Details of the method of attaching proprietary driving shoes.
- g) When load testing is specified in the Contract Documents, details of the full-scale test, including site preparation and the details of the load application, components, equipment, testing apparatus, and method of monitoring.
- h) Information pertinent to establishing the resistance of a pile when the wave equation analysis method is used.

903.04.02.04.02 Caisson Piles

The Contractor shall submit the following to the Contract Administrator 14 Days prior to construction, for the purpose of quality assurance and documentation:

- a) A schedule of work identifying time and sequence of activities. A Request to Proceed to allow for quality assurance oversight by the Contract Administrator.
- b) Detailed procedures for caisson excavation in overburden and rock.
- c) Detailed procedures for casing and liner installation and for the withdrawal of the liner.
- d) Detailed procedures for slurry displacement method of excavation, including disposal of slurry upon completion.
- e) Detailed procedures for tremie concrete, including the size of tremie delivery pipe.
- f) Detailed procedure for placing concrete in the dry.
- g) Method of maintaining the steel reinforcement cages in position in the caisson.
- h) Details of filling the annular void around a casing.
- i) Details of procedure to be used for monitoring installation.
- j) When load testing is specified in the Contract Documents, details of the full-scale test, including site preparation, details of the load application, components, equipment, testing apparatus, and method of monitoring.

903.04.02.04.03 Displacement Caisson Piles

The Contractor shall submit the following to the Contract Administrator 14 Days prior to construction, for the purpose of quality assurance and documentation:

- a) A schedule of work identifying time and sequence of activities. A Request to Proceed to allow for quality assurance oversight by the Contract Administrator.

- b) Type of equipment, anvil, helmet, and hammer details, including the hammer energy assumed by the Contractor, stated potential energy (rated energy) of the hammer, operating efficiency, maximum stroke or drop, and weight of the ram.
- c) Details of procedures used for installation of displacement caisson piles, including detailed procedures for liner installation and withdrawal.
- d) Method of maintaining the steel reinforcement cages in position in the pile.
- e) Details of procedure to be used for monitoring pile installation.
- f) When load testing is specified in the Contract Documents, details of the full-scale test, including site preparation, and the details of the load application, components, equipment, testing apparatus, and method of monitoring.

903.04.02.04.04 Steel Reinforcement Cages

The Contractor shall submit Working Drawings showing the fabrication details of the steel reinforcement cages, including the lifting points and lifting lugs, to the Contract Administrator 14 Days prior to fabrication, for the purpose of quality assurance and documentation.

903.05 MATERIALS

903.05.01 Wooden Piles

Wooden piles shall be according to CAN3-056 and shall be clean and peeled. Treated piles shall be pressure treated with creosote according to CAN/CSA O80.

Wooden piles shall be provided with collars sufficiently strong to prevent splitting of the head of the wooden pile during driving.

903.05.02 Steel Piles

903.05.02.01 H-Piles

Steel H-piles shall be according to CAN/CSA G40.20/G40.21, Grade 350 W.

903.05.02.02 Tube Piles

Steel tube piles shall be according to ASTM A 252, minimum Grade 2.

903.05.02.03 Sheet Piles

Steel sheet piles shall be according to ASTM A 328M.

903.05.02.04 Straightness Tolerance for Steel Piles, Casings, and Liners

Steel piles, casings, and liners shall conform to a straightness tolerance of 1.5 mm maximum per metre of length.

Steel sheet piles shall be sufficiently straight to prevent binding in the interlock during driving.

903.05.03 Driving Shoes and Rock Points

Rock points and driving shoes shall be as specified in the Contract Documents.

Driving shoes shall transfer the driving stresses to the pile over the full cross-sectional area of the pile.

Where precast concrete piles are driven into dense or hard material, a steel driving shoe cast into the concrete shall be provided.

Where wooden piles are driven into dense material, a steel plate driving shoe shall be provided to prevent damage to the bottom of the pile.

903.05.04 Casing for Caissons

Casings shall be according to ASTM A 252, Grade 2. If welded, they shall be welded by the electric arc method according to CSA W59.

The casing wall thickness specified is the minimum that shall be supplied. The wall thickness shall be increased as required to ensure the casing is not damaged during handling and installation.

903.05.05 Steel Reinforcement

Steel reinforcement shall be according to OPSS 1440.

903.05.06 Concrete

903.05.06.01 General

Concrete shall be according to DBSP 1350.

903.05.06.02 Tube Piles

Concrete shall have a slump of 150 to 180 mm.

903.05.06.03 Caisson Piles

Concrete shall have a slump of 150 to 180 mm. When approved by the Contractor's Engineer in writing, admixtures may be used. Where the liner is to be withdrawn, sufficient retarder shall be added to prevent arching of concrete during liner withdrawal and to prevent setting of concrete until after the liner is withdrawn.

903.05.07 Precast Concrete Piles

The production of precast reinforced concrete piles shall be according to DBSP 0904, OPSS 905, and DBSP 0909.

Steel reinforcement shall be placed such that direct loading during the ram stroke shall not occur.

Lifting anchors shall be at least 25 mm clear from reinforcement or prestressing steel in the pile.

Concrete in precast reinforced concrete piles shall be according to DBSP 1350 and have a nominal minimum 28-Day compressive strength of 45 MPa.

Concrete for precast reinforced concrete piles shall be cured according to DBSP 0904.

Concrete for precast reinforced concrete piles shall be placed in smooth mortar-tight forms that are supported to prevent excessive deformation or settlement during placing or curing.

Unformed surfaces shall be finished smooth.

When removed from the form, the pile shall present true, smooth, even surfaces free from honeycombs and voids. The pile shall be straight so that a line stretched from butt to tip on any face shall not be more than 25 mm from the face of the pile at any point.

Each precast reinforced concrete pile shall have the date of manufacture (i.e., yyyy-mm-dd) inscribed on it.

903.05.08 Slurry

903.05.08.01 Solids

Bentonite and polymers shall be according to API Spec 13A.

903.05.08.02 Water

Water shall be according to OPSS 1302.

903.05.08.03 Slurry Composition

The slurry shall consist of a stable colloidal suspension of pulverized solids or polymers thoroughly mixed with water. The density, viscosity, sand content, and pH of the slurry being used during excavation shall be according to API RP 13B-1.

903.06 EQUIPMENT

903.06.01 Hammers

Hammers shall be capable of installing the piles, casings, and liners to the depth or resistance specified in the Contract Documents, without damage to the portions that are not cut off.

The hammer used to chisel the rock point into the rock shall be capable of delivering a controlled blow in 10% increments ranging in energy from zero to the maximum hammer energy.

For precast reinforced concrete piles, the heaviest hammer practicable shall be employed and the stroke limited so as not to damage the piles. When choosing the size of the hammer, consideration shall be given to whether the pile is to be driven to a resistance or to a given depth.

903.06.02 Helmets and Striker Plates

The head of steel piles shall be protected by a striker plate or a helmet. Helmets shall have adequate and suitable cushioning material. Helmets and striker plates shall distribute the blow of the hammer evenly throughout the cross-section of the pile head.

903.06.03 Leads

Pile driver leads shall be built to afford freedom of movement for the hammer and shall be held in position at the top and bottom by guys, stiff braces, or other approved means to ensure support of the pile, casing, or liner while it is being driven. Swinging leads shall not be permitted.

Batter piles, casings, or liners shall be driven with leads aligned parallel to the axis of the pile, casing, or liner. The leads shall be equipped with a fixed, rigid, adjustable kicker.

903.06.04 Followers

When use of followers are specified in the Contract Documents, followers shall be of type, size, shape, length, and weight as to permit driving the pile, casing, or liner at the location and to the required depth or ultimate resistance specified in the Contract Documents. The follower shall be provided with a socket or hood carefully fitted to the top of the pile, casing, or liner to minimize loss of energy and to prevent damage to the pile, casing, or liner, and shall have sufficient rigidity to prevent "whip" during driving.

When followers are permitted, an identical follower shall be used when the set is being determined.

903.07 CONSTRUCTION

903.07.01 Transporting, Storing, and Handling Piles, Casings, Liners, and Reinforcing Steel Reinforcement Cages

903.07.01.01 General

Piles, casings, liners, and steel reinforcement shall be transported, stored, and handled in such a manner that damage is prevented and the strength of the components is not affected by deterioration or deformation.

Components shall be lifted and placed using appropriate lifting equipment, temporary bracing, guys, or stiffening devices so that the components are at no time overloaded, unstable, or unsafe.

Material shall be supported to prevent unequal settlement when stacked.

903.07.01.02 Wooden Piles

Cant hooks, dogs, pile pulls, or use of other lifting methods that might damage the integrity of the pressure treated surface shall not be used. Cuts or breaks in the surface of treated piling shall be given three brush coats of hot creosote oil. Bolt holes shall be treated with three applications of hot creosote oil applied with a bolt hole treater.

903.07.01.03 Handling Holes in Steel Piles

Unless otherwise approved by the design Engineer, holes shall only be made in the portion of the pile to be cut off or in the portion of the pile to be encased in concrete.

When other holes are approved to be cut in a pile, they shall be covered by splice plates placed on both sides of the section. The thickness and the mechanical properties of the plate material shall be at least equivalent to the pile material.

903.07.01.04 Precast Reinforced Concrete Piles

Precast concrete piles shall be handled only from the designated lifting points.

When lifting or transporting precast reinforced concrete piles lift anchors, slings, or other approved means shall be used. Care shall be taken when lifting and transporting to avoid any overstressing of the pile or cracking of the concrete.

Precast reinforced concrete piles shall be so handled to avoid breaking or chipping their edges.

Lift anchors shall be removed, and the holes filled with a non-shrink grout or epoxy installed according to the manufacturer's recommendations.

903.07.01.05 Caisson Casings and Liners

Casings and liners shall be handled and stored in such a manner to avoid damage or distortion to them. The casings and liners shall be maintained circular within $\pm 2\%$ of the casing or liner diameter.

903.07.02 Driven Piles

903.07.02.01 Pile Driving Requirements and Restrictions

Piles shall not be driven until embankment work or excavation work has been completed to the underside of the footing. When driving of the piles is completed, all material between the piles shall be removed to the correct elevation and any holes or voids created shall be filled to the correct elevation with compacted material approved by the Contractor's design Engineer or design Engineer's designee.

Piles shall be installed at the locations specified in the Contract Documents and to the set or depth specified without being damaged. Damage to the pile, casing, or liner during driving shall be prevented by limiting the drop or energy and number of blows of the hammer. The hammer, helmet, cap block, striker plate, and pile shall be coaxial and shall sit squarely upon each other.

A shorter stroke shall be used, and proper precaution shall be taken when there is a danger of damaging or over driving the piles, casing, or liners under conditions such as:

- a) In the early stages of driving a long pile where a hard layer near the ground surface has to be penetrated.
- b) Where there is very soft material of a considerable depth and a large penetration is achieved at each hammer blow.
- c) Where it is anticipated the pile shall meet refusal on rock or other impenetrable soil.
- d) When piles are driven onto sloping bedrock.

Damage to adjacent structures, Utilities, and fresh concrete shall be prevented during pile installation. Piles shall not be driven within a radius of 8 m of concrete that has been in place for less than 72 hours. Piles shall not be driven within a radius of 15 m of concrete that has been in place for less than 72 hours without the approval of the design Engineer.

The tops of all piles shall be either square to the longitudinal axis of the pile or horizontal as indicated in the Contract Documents.

Piles shall not be forced into their proper position using excessive manipulation. Pile damage due to excessive driving shall be avoided.

903.07.02.02 Driving Shoes and Rock Points

Driving shoes and rock points shall be installed in locations specified in the Contract Documents.

Driving shoes shall be welded in accordance with the Contract Documents.

When driving shoes are specified in the Contract Documents, the Titus H bearing pile point, standard model, may be substituted for the driving shoes.

When Oslo points are specified in the Contract Documents, the Titus H bearing pile point, rock injector model, may be substituted for the pile points.

Where proprietary driving shoes are used, they shall be welded or otherwise attached to the driven piles according to the manufacturer's specifications.

903.07.02.03 Splicing

903.07.02.03.01 General

Any damaged material shall be cut-off prior to splicing.

903.07.02.03.02 Wooden Piles

Wooden piles shall not be spliced.

903.07.02.03.03 H-Piles, Tube Piles, and Sheet Piles

Welding shall be according to CSA W59 and shall be done by a qualified welder employed by a firm certified according to CSA W47.1, Division 1 or Division 2.1.

Steel H-piles and steel tube piles may be spliced providing that the pieces being spliced are not less than 3 m long, except for piles at integral abutments for which the pieces being spliced shall not be less than 7 m long. Splices in piles located into a watercourse shall only be introduced under the low water level, unless the piles are encased in concrete.

Sheet piles shall not be spliced without approval by the design Engineer.

903.07.02.03.04 Precast Reinforced Concrete Piles

Precast reinforced concrete piles shall only be spliced when specified in the Contract Documents and the splices shall only be made with approved mechanical splicing devices.

903.07.02.04 Concrete in Steel Tube Piles

Concrete in steel tube piles shall be placed according to DBSP 0904.

903.07.02.05 Cutting Off Piles

903.07.02.05.01 General

Driven piles shall be cut to the elevation as specified in the Contract Documents.

The length of pile supplied shall be sufficient to ensure there is no damaged material below the cut off. Damaged material at the pile head shall be cut off.

Piles shall not be cut off until retapping, re-driving, and specified load testing are complete.

903.07.02.05.02 Wooden Piles

Where wooden piles are broomed, splintered, or otherwise damaged below the cut-off elevation, the pile shall be considered defective and shall be replaced.

903.07.02.06 Protective Coating for Steel H and Steel Tube Piles

Exposed steel H and steel tube piles shall have a coal tar epoxy protective coating applied from an elevation 600 mm below the low water level or finished ground surface up to the top of the exposed steel.

The steel surfaces shall be cleaned according to SSPC-SP10 prior to application of a coal tar epoxy system that shall be according to OPSS 911.

903.07.02.07 Monitoring Driven Piles

903.07.02.07.01 General

The Contractor shall submit a Request to Proceed to Contract Administrator. The driving of piles shall be carefully monitored and controlled and pile driving records produced for each pile under the direction of the Contractor's design Engineer or design Engineer's designee. A pile driving record shall be submitted to the Contract Administrator for the purpose of quality assurance and documentation.

The Contractor shall not overdrive the piles. When driving to a specified ultimate resistance, or driving to bedrock, the Contractor shall drive the piles to the anticipated tip elevation. If a pile does not reach set at the anticipated tip elevation, the Contractor shall notify the design Engineer for review and decision prior to proceeding with driving of that pile.

In soils where there is a possibility of piles moving upward due to ground heave, elevations of completed pile tops shall be measured at time intervals determined by the design Engineer or design Engineer designee while nearby piles are being installed. The readings shall be recorded and submitted to the design Engineer and Contract Administrator (for the purpose of quality assurance and documentation) as the work proceeds.

903.07.02.07.02 Driving to a Specified Elevation

Piles shall be driven to an elevation specified in the Contract Documents. Driving piles to other elevations shall only be done when approved in writing by the design Engineer.

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be determined using the High Strain Dynamic Testing (as required, where piles do not reach bedrock) at end of initial driving as specified in the Contract Documents. If the specified ultimate resistance is not achieved, retap/restrike shall be conducted after initial driving as specified in the Contract Documents.

A Request to Proceed shall be submitted to the Contract Administrator after the design ultimate resistance is achieved.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

903.07.02.07.03.02 Driving to a Set

The founding elevation shall be established by driving to a set determined in accordance with the dynamic formula specified in the Contract Documents or by the application of the wave equation analysis procedure that verifies the pile resistance. This set shall be established on the first pile of every ten piles driven in a pile group.

The other piles shall be controlled by the pile penetration rate in blows per millimetre that correlates to the set.

When new conditions, such as change in hammer size, change in pile size, or change in soil material occur, new sets shall be determined.

903.07.02.07.03.03 Driving to Bedrock

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

Where rock points are used, the rock points shall penetrate into the rock. Piles driven using rock points shall be driven to ensure adequate seating on the bedrock without damaging the pile.

Driving of piles on sloping bedrock shall be stopped when initial contact is made with the bedrock. The bedrock elevation shall be recorded. Driving shall then continue, commencing with energy of 10% of the maximum energy of the hammer. The pile shall be driven in sets of 20 blows at this energy until no penetration is observed. Twenty additional blows shall be applied, and, if no penetration is observed, the energy shall be increased by an additional 10% and the above procedure repeated.

Driving shall continue with these stepped increases in energy and with the same series of blows as described above, until the pile has been seated on the bedrock.

If unrealistic excessive penetration per blow is observed, driving shall be stopped, and this excessive penetration immediately reported to the design Engineer and Contract Administrator.

The design Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

903.07.02.07.04 Wave Equation Analysis

When requested by the Contract Administrator, the Contractor shall supply all equipment, material, and personnel to conduct the wave equation analysis procedure. The design Engineer shall review the results of the analysis and submit a report with recommendations to the Contract Administrator.

903.07.02.07.05 Hammer Performance

When requested by the Contract Administrator, the Contractor's design Engineer or design Engineer designee shall verify the hammer performance using the pile driving analyzer or other approved equivalent. Hammer performance shall be verified to ensure that the actual potential energy (rated energy) is not less than 90% of the stated potential energy. The Contractor shall provide all instrumentation, access, and assistance for the testing and monitoring. The Contractor shall provide a copy of the hammer performance verification results.

903.07.02.07.06 Retapping Tests on Piles

In each pile group, 10% of the piles rounded up to the next whole number, but no fewer than two piles, shall be retapped no sooner than 24 hours after installation of the individual pile to confirm that the ultimate axial resistance has been sustained.

Retapping of piles driven to bedrock is not required.

903.07.02.07.07 Retapping and Redriving Piles

When the retapping tests indicate that the ultimate axial resistance has not been achieved on any one pile, all piles in the group shall be retapped.

Where the retapping reveals that the ultimate axial resistance of the piles has not been achieved, the piles that have not achieved the ultimate axial resistance shall be redriven to the specified resistance.

Where piles have risen, the piles shall be redriven to the original depth.

903.07.02.08 Jetting

Jetting shall be carried out in such a manner that the resistance of the piles already in place and the safety of adjacent structures shall not be impaired. Jetting shall be stopped at least 1 m above the final expected pile-tip elevation and at least 1 m above the tip elevation of any piles previously driven within 2 m of the jet. Where piles are to be end bearing on rock, jetting may be carried to the rock surface.

The driving and jetting of precast reinforced concrete piles shall not be carried out simultaneously.

903.07.03 Caisson Piles

903.07.03.01 General

Caissons shall be constructed as specified in the Contract Documents.

The final bearing elevation shall be as specified in the Contract Documents or as determined by the design Engineer. When permanent casings are not specified, the caisson shall be constructed in a drilled hole with or without the use of a temporary liner or slurry as determined by the Contractor.

903.07.03.02 Excavation

903.07.03.02.01 General

Sidewall stability and basal stability shall be maintained throughout the excavation and concrete placement operation. Soil cave-in into the excavation hole shall be prevented.

The bottom of the excavation shall be cleaned before the start of concrete placement.

Excavation methods shall be such that the sides and bottom of the hole are straight and free of loose material that might prevent intimate contact of the concrete with undisturbed soil or bedrock.

Except when founded on sloping rock, the caisson bottom shall be level. On sloping rock, the caisson bottom may be stepped, with each step not greater than $\frac{1}{4}$ the diameter of the bearing area.

903.07.03.02.02 Casings

When an auger is used to excavate for a casing, the diameter of the auger shall be no greater than the outside diameter of the casing.

903.07.03.02.03 Liners

The diameter of the excavation for the installation of liners shall not exceed the diameter of the liner by more than 150 mm.

903.07.03.02.04 Slurry Method

The level of slurry in the excavation shall be sufficient to prevent the intrusion of water and to maintain a stable wall with no cave-in, sloughing, or basal heave.

Slurry shall be tested as specified in API RP 13B-1. The Contractor shall provide all test equipment required for the tests. A slurry sampler capable of obtaining samples at any depth within the caisson hole shall always be available.

At least 1 set of tests shall be completed every 4 hours during the slurry operation. Samples shall be taken from the mud tank and from within the caisson at a depth within 300 mm of the bottom.

903.07.03.03 Inspection of the Excavation

The bottom of excavations shall be visually inspected. The bottom of excavations shall be inspected with a Mini Shaft Inspection Device (Mini-SID) and/or Shaft quality inspection device (SQUID), lowered to the base of the shaft.

903.07.03.04 Dewatering

Where dewatering is required, the Contractor shall affect a dewatering scheme in such a manner as to prevent any disturbance to the base founding material. The dewatering shall not create subsidence or cause ground loss that may adversely affect the work or adjacent structures.

903.07.03.05 Backfilling Liners Left in Place

The annular space between a liner permanently left in place and shaft excavation shall be filled with concrete or fluid grout.

903.07.03.06 Steel Reinforcement

Steel reinforcement steel shall be installed according to OPSS 905. Steel reinforcement cages shall be checked to ensure conformance to the Working Drawings prior to installation and during placement of concrete.

The steel reinforcement cage shall be fabricated in one piece.

Welding of steel reinforcement and use of splices shall not be done unless specified in the Contract Documents.

The steel reinforcement shall not be displaced or distorted during the construction of the caisson.

903.07.03.07 Concrete

903.07.03.07.01 General

A Request to Place Structural Concrete shall be submitted to the Contract Administrator prior to concrete placement.

The placement of concrete shall not proceed until the Contract Administrator has issued a Notice to Proceed to the Contractor.

Concrete shall be placed in the caisson according to DBSP 0904, and as specified herein.

The reinforcement shall not be displaced or distorted during the construction of the caisson.

When casing or liner withdrawal is part of the design, arching of concrete during casing or liner withdrawal shall be prevented.

903.07.03.07.02 Concrete Placed in the Dry

The concrete may be placed free fall provided the fall is vertically down the centre of the opening and transverse ties, spacers or other objects do not impede the free fall. In the event of interference with the concrete free fall, an elephant trunk or other means shall be used to prevent concrete segregation.

Concrete shall be placed in a continuous operation from the bottom to the top of the caisson or, where columns are cast integral with the caisson, to the elevation of the bottom of the column steel reinforcement cage. The concrete shall be vibrated for the last 1.5 m of the pour.

903.07.03.07.03 Concrete Placed Under Water or Under Slurry

Tremie or pumped concrete shall be carried out in one continuous operation. The tremie or pumping operation shall be a continuous flow of concrete that prevents the inflow of water or slurry.

Where tremie concrete is to be placed in a caisson under water, the Contractor shall maintain an adequate head of water within the excavations to prevent the inflow of water through the base or walls of the caisson as the concrete is being placed.

Where tremie is placed under slurry, the caisson shall be filled with concrete entirely by tremie and the method of deposition shall not be changed part way up the caisson.

When concrete placement is not started within 6 hours of acceptance of the excavation, the excavation shall be redrilled, cleaned, and the slurry tested before concrete placement commences.

903.07.03.07.04 Withdrawal of Liners

Arching of concrete during withdrawal of the liner shall be prevented.

During withdrawal, the bottom of the liner shall have a minimum embedment into the concrete being placed and a sufficient head of concrete shall always be maintained above the bottom of the liner to prevent intrusion of soil and water into the hole.

During withdrawal, upward or downward movement of the steel reinforcement shall be monitored. Upward or downward movement shall be restricted to 150 mm.

A theoretical concrete level shall be calculated based on the quantity of concrete placed and the caisson dimensions, and this theoretical level shall be compared to the actual level of concrete in the caisson to provide a check for possible separation of shaft concrete during liner withdrawal.

903.07.03.07.05 Founding Elevation

The final founding elevation shall be as specified in the Contract Documents or an elevation approved in writing by the design Engineer. When casings are not specified in the Contract Documents, the caisson shall be constructed in a drilled hole with or without the use of a liner or slurry as determined by the Contractor.

Mini-Sid and/or SQUID shall be used to verify the founding soil.

Except when founded on sloping unweathered bedrock, the caisson bottom shall be level. On sloping unweathered bedrock, the caisson bottom may be stepped, with each step not greater than one quarter the diameter of bearing area.

The bearing area of the caisson pile shall be approved by the design Engineer. A Request to Place Structural Concrete shall be submitted to the Contract Administrator prior to placing concrete. Complete access to inspect the bearing area of the caisson pile prior to the placement of concrete shall be given to the Contract Administrator. The placement of concrete shall not proceed until the Contract Administrator has issued a Notice to Proceed to the Contractor.

903.07.04 Displacement Caisson Piles

Work shall be carried out in accordance with the displacement caisson pile suppliers' installation procedures. A permanent liner shall be used when specified in the Contract Documents.

The sequence of installation shall be such as to prevent damage to any recently completed piles.

The pile shall not be founded above or below the specified pile tip elevation without approval in writing from the design Engineer.

The Contractor's design Engineer or design Engineer's designee shall witness the pile installation operation.

903.07.05 Tolerances

903.07.05.01 Driven Piles

- a) Cut-off elevation ± 25 mm.
- b) Deviation from vertical not more than 1H:50V, except in the case of a pile cap or footing supporting only a single row of piles the deviation shall not be more than 1H:75V in the direction of the span.
- c) The deviation from the specified inclination for battered piles shall not exceed 1H:25V.
- d) The centre of the pile at the junction with the pile cap shall be within 150 mm measured horizontally of that specified except in the case of a pile cap or footing supported on a single row of piles the deviation shall not be more than 75 mm measured horizontally in the direction of the span.

903.07.05.02 Caissons and Displacement Caisson Piles

- a) Cut-off elevation ± 25 mm.
- b) Horizontal location at cut-off not more than 5% of shaft diameter or 75 mm, whichever is less.
- c) Vertical alignment not more than 2% of the caisson length from vertical for vertical caissons, or 2% of the caisson length from the specified inclination for battered caissons.

903.07.06 Load Test

When a load test is specified in the Contract Documents, the testing shall be according to ASTM D 1143M for piles under vertical static load, ASTM D 3689 for piles under tensile load, and ASTM D 3966 for piles under lateral loads. The Contractor's design Engineer, or design Engineer's designee, shall organize and notify the Contract Administrator of the scheduled test. The Contractor's design Engineer, or design Engineer's designee, and the Contract Administrator shall witness the pile load test. All records and results of the pile load test shall be submitted to the Contract Administrator for the purpose of quality assurance and documentation.

The Contractor shall provide all necessary personnel, equipment, and material to make adjustments during the tests and shall have at least one skilled worker present for the complete duration of each test. The Contractor shall ensure that this worker shall have demonstrated experience in load testing of piles.

The Contractor shall do all necessary grading work to ensure a level dry working area at the test location and shall erect an adequate enclosure sufficient to provide complete protection from adverse weather conditions for the complete duration of the tests, including all temporary work required to obtain access to the site for the personnel, equipment, and materials.

On completion of the tests, the Contractor shall clear and restore the site to the satisfaction of the Contract Administrator. Piles that are not part of the finished work shall be cut off 1.2 m below ground level or 0.6 m below stream bed level. Any resulting void shall be backfilled with suitable fill material.

903.07.07 Repair of Welds

Any section of weld that does not meet the requirements of the Contract Documents shall be removed and rewelded.

903.08 Quality Assurance

903.08.01 Visual Inspection of Welds

Complete access to visually inspect the welds shall be given to the Contract Administrator.

All welds shall conform with the requirements of CSA W59 and the Contract Documents. A representative sample of splice welds, not less than 30% of the welds will be selected by the Contract Administrator for visual inspection. The selected splice welds shall be taken from different piles.

If the sample welds do not pass the visual inspection and need to be repaired, the visual inspection by the Contract Administrator may be increased up to 100% of the welds.

903.08.02 Non-Destructive Testing of Welds

The Contract Administrator shall be notified in writing, 48 hours in advance of installing piles which will require weld splicing. The Contract Administrator shall be immediately notified in writing, if there are any schedule changes for each pile requiring weld splicing.

A Request to Proceed shall be submitted to the Contract Administrator after the completion of splice welds for each construction stage of work.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

Radiographic or ultrasonic testing shall be carried out by the Contract Administrator using procedures according to CSA W59.

Ultrasonic or radiographic testing shall be carried out on the entire length of selected splice welds chosen at random by the Contract Administrator.

The welds selected for the random ultrasonic or radiographic testing shall be taken from different piles and shall include 10% of the splice welds, rounded to the next highest number, but no fewer than two.

If any welds do not pass the ultrasonic or radiographic-testing and need to be repaired, these non-destructive testing requirements may be increased up to 100% of the welds.

903.08.03 Repaired Welds

All welds that have been repaired shall be visually inspected and shall undergo non-destructive testing performed by the Contract Administrator.

903.08.04 Non-Destructive Test Reports and Visual Inspection Reports

Results from completed Visual Inspection Reports and Non-Destructive Test Reports will be provided upon request.

Costs associated with any required removals and replacement or repairs of defective welds, following the visual inspection or non-destructive testing by the Contract Administrator, shall be the Contractor's responsibility at no additional cost to the Owner. No additional payment will be made for labour and equipment provided by the Contractor, and the Contractor will pay the Owner \$500, for each weld requiring additional re-testing.

903.08.03 Displacement Caisson Piles

A Request to Proceed shall be submitted to the Contract Administrator before the installation of displacement caisson piles.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

903.09 MEASUREMENT FOR PAYMENT – Not Used

903.10 BASIS OF PAYMENT – Not Used

NOTES TO DESIGNER:

- * Fill-in one of the following, as recommended by the foundations engineer in consultation with the MTO Foundations Office:
 - Dynamic Formula, or
 - High-Strain Dynamic Testing

WARRANT: In Design-Build contracts with Deep Foundations.

CUSTODIAN: Tony Sangiuliano, Senior Foundation Engineer, Foundations Section, Structural Standards and Specifications Office and Felipe Mendoza, Senior Contract Innovations Analyst, Special Planning Initiatives Office.

OBSTRUCTIONS

Notice to Contractor

The Contactor is hereby notified that the native soils at the site of the Nagagamisis Narrows bridge are glacially derived and as such are very dense and should be expected to contain cobbles and boulders, as encountered at a number of boreholes advanced at this site, which could affect excavations and the installation of deep foundations and/or temporary shoring and roadway protection systems. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for sub-excavation and installation of the foundation and temporary shoring and roadway protection systems.

STEEL CASINGS FOR INTEGRAL ABUTMENTS (Alternative to CSPs) – Item No.

Non-Standard Special Provision

MATERIAL

Steel casings shall be in accordance with OPSS.PROV 1802.

The steel casing shall be of the diameter and wall thickness specified on the Contract Drawings.

The steel casing shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract Drawings; field cutting and splicing of steel casings will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each steel casing shall be parallel to each other.

Handling and storage of steel casing shall be in accordance with the manufacturer's recommendations. Damaged steel casings shall be rejected.

Sand Fill

The sand fill for backfilling the steel casings shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Weight
2 mm	#10	100%
600 µm	#30	80% to 100%
425 µm	#40	40% to 80%
250 µm	#60	5% to 25%
150 µm	#100	0% to 6%

CONSTRUCTION

The steel casing shall be positioned such that the piles are centrally positioned within the casing.

The Contractor shall ensure the full perimeters of the top of the steel casing at each abutment are at the elevation and orientation shown on the working drawings.

The steel casing at each pile shall be constructed to the following tolerances:

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of steel casing from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the steel casing from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the steel casing and pile (or in the wet if required, provided the water is adequately displaced). No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the steel casing.

Basis of Payment

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

UNWATERING OF STRUCTURE EXCAVATION - Item No.

Notice to Contractor

Construction of the abutments for the new permanent bridge and the temporary modular bridge (TMB) will require excavations to extend below the groundwater level and the adjacent lake water level. The embankment fill, organic soil and sand within the excavation may slough, run, boil or cave into the excavation unless appropriate groundwater controls are in place. The Contractor is to design and install an appropriate excavation protection and unwatering system (if required) to enable construction and prevent disturbance to the founding soils for the abutment pile caps, to ensure the basal stability of the soils at the base of the corrugated steel pipe (CSP) liners for the integral abutments, and to avoid disturbing the sand fill placed within the CSP liners for the integral abutments.

TEMPORARY MODULAR BRIDGE - GRANULAR PAD CONSTRUCTION AND WAIT PERIOD FOR EMBANKMENT SETTLEMENT

Notice to Contractor

The granular pad below and behind the Temporary Modular Bridge (TMB) abutments for the approach embankments should be constructed using OPSS.PROV 1010 Granular 'B' Type II material. The granular pad should extend at least 1 m beyond the plan limits of the abutment, and be sloped no steeper than 1H:1V, downward and outward from the top of the pad to the subgrade. The granular pad should be constructed in accordance with OPSS.PROV 206 and DBSP0902 (Structures).

Due to the shallow subexcavation anticipated to be less than 1 m to 1.5 m below the lake water level (measured to be at Elevation 286.9 m in November 2016 and Elevation 287.4 m in May 2017) at the TMB abutments, the granular pad may be constructed in wet conditions (i.e., without dewatering within the subexcavation), provided the excavation remains stable, with suitable compaction of the Granular 'B' Type II (e.g. with the excavator bucket) under the review of a qualified Foundation Specialist.

The granular pad should be constructed concurrently with embankment construction and preloaded for at least one week before TMB footing / abutment construction. The top of the granular pad (foundation subgrade) may need to be re-established and re-levelled after settlements have occurred during the preload wait period. The granular pad must be protected from erosion / scour during the temporary design life of the structure.

Vibration Monitoring for Existing Structure - Item No.

Non-Standard Special Provision

Scope

Vibration monitoring should be completed during the driving of sheet piles adjacent to the existing bridge abutments when the bridge is in operation. Vibration monitoring equipment shall be capable of measuring and recording ground vibration Peak Particle Velocity (PPV) up to 200 mm/s in the vertical, transverse, and radial directions. The equipment shall have been calibrated within the last 12 months either by the manufacturer or other qualified agent. Vibration monitoring threshold should be PPV of 50mm/s for the existing bridge. A vibration monitoring summary letter / memorandum shall be prepared which documents the vibrations recorded during sheet pile driving.

Basis of Payment

Payment at the contract price for the above item shall include all labour, equipment and material required to do the work.



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