June 27, 2018

FOUNDATION INVESTIGATION AND DESIGN REPORT

REPLACEMENT OF NAGAGAMISIS NARROWS BRIDGE - SITE NO. 38N-001
HIGHWAY 631, TOWNSHIP OF FROST, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5569-09-00, WP 5312-14-01

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GEOCRES NO.: 42F-054

REPORT

Report Number: **1661607 - R03**

Distribution:

1 Copy + 1 PDF Copy: Ministry of Transportation, Ontario, North Bay, Ontario (Northeastern Region)

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PART A

FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF NAGAGAMISIS NARROWS BRIDGE – SITE NO. 38N-001
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the replacement of the Nagagamisis Narrows Bridge (Site No. 38N-001). 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 purpose of this investigation is to establish the subsurface soil and bedrock conditions at the proposed bridge location, including the associated approach embankments and detour alignment, by borehole drilling, rock coring and laboratory testing on selected soil and rock core samples.

The Terms of Reference and Scope of Work for the Foundation Investigation are outlined in MTO's Request for Proposal dated April 2016. Golder's proposal for foundation engineering services associated with replacement of this structure is contained in Section 17.8 of LEA's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundations engineering services for this project, dated November 1, 2016.

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.

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 The Nagagamisis Narrows Bridge is 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.

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

3.0 INVESTIGATION PROCEDURES

The field work was carried out on May 29, 2017, between June 10 and June 12, 2017, and on August 17 and 18, 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. 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 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 September 19, 2017.

The field work was supervised on a full-time basis by a member of 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 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 May 29, 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, 2006[1]). 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 (ISRM[2]) standard classification system.

The borehole locations and elevations were measured in the field by Golder personnel, relative to existing site features and surveyed to point HCP-101. The borehole locations (referenced to the MTM NAD83 co-ordinate

[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.

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, Zone12)		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

4.0 SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain (NOEGTS)[3] mapping, the Nagagamisis 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 Nagagamisis 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.

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[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

Groundwater 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) / Relative Density	Laboratory Testing
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 **Very Loose to Very Dense**	w = 5% – 26% 5 – M (Fig. B1)
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 June 12, 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

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

Borehole No.	Ground Surface Elevation (m)	Depth to Groundwater Level (m)	Approximate Groundwater Elevation (m)
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 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.

5.0 CLOSURE

The field drilling program was supervised by Mr. Shane Albert and Mr. Mathew Riopelle. This Foundation Investigation Report was prepared by Ms. Aronne-Kay De Souza, EIT, and the technical aspects were reviewed by Mr. André Bom, P.Eng., a geotechnical engineer and Associate of Golder. Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer, Principal of Golder and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.

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PART B

DETAIL FOUNDATION DESIGN REPORT
REPLACEMENT OF NAGAGAMISIS NARROWS BRIDGE – SITE NO. 38N-001
HIGHWAY 631, TOWNSHIP OF FROST, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5569-09-00, WP 5312-14-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the Nagagamisis Narrows Bridge (Site No. 38N-001) located on Highway 631 north of Hornepayne, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation of the subsurface information and recommendations presented in this Foundation Design Report (Part B) are intended to provide MTO's designers with sufficient information to assess the feasible foundation alternatives and to design the proposed bridge structure foundations and approach embankments, and associated works for a detour alignment/structure on the north side of the existing highway.

The discussion and recommendations contained in this Foundation Design Report (Part B) shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. The contractor must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A), as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of MTO to provide recommendations on the foundation aspects for the design of the replacement of the Nagagamisis Narrows Bridge.

The existing bridge is shown in plan on Drawing 1 and consists of a three-span structure about 28 m in length supported by timber cribs. The existing bridge will be replaced with a 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. A 40 m long, single-span and single-lane Temporary Modular Bridge (TMB) is proposed to be constructed 13 m to the east of the existing bridge (as measured centreline to centreline) to carry traffic during 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.

Based on discussions with LEA we understand that the detour and the TMB structure are to be constructed in Year 1, and open to traffic in Year 2 during the replacement of the existing bridge; the temporary detour and TMB will be removed at the end of Year 2 following completion of the construction of the new bridge.

6.2 Consequence and Site Understanding Classification

The replacement bridge is being designed in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC 2014).

In accordance with Section 6.5 of CHBDC (2014) and its Commentary, the proposed bridge and its foundation system is considered to be classified as having a "typical consequence level" associated with exceeding limits states design.

The degree of understanding, based on the scope of the current foundation investigation and design, is considered 'typical' as described in Clause 6.5.3.2 of CHBDC (2014). The appropriate corresponding Ultimate Limit States

(ULS) and Serviceability Limit States (SLS) consequence factors, Ψ, geotechnical resistance factors at ULS (ϕ_{gu}) and SLS (ϕ_{gs}), respectively, from Tables 6.1 and 6.2 of the CHBDC have been used for design in this report.

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. Drilled steel casings are also feasible to support the replacement bridge; however, we understand they are not preferred over steel H-piles from a structural perspective at this site, as they preclude the use of integral abutments. As such, drilled steel casings for the replacement bridge are not discussed further in this report. 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 feasible to support the relatively light TMB abutments (compared to the heavier replacement bridge). We understand that piles are not preferred from a structural perspective when compared with shallow foundations. As such, steel H-piles for the TMB are not discussed further in this report, although they would achieve similar geotechnical resistances to piles for the permanent structure.

A comparison of the alternative foundations options based on advantages, disadvantages, risks and relative costs is provided in Tables 1 and 2 following the text of this report for the replacement bridge and the TMB respectively. 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 (replacement bridge). 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 could 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 from LEA that HP310X132 are preferred over HP360X132 at this site for the greater axial compressive resistance they will afford in the integral abutment configuration.

Based on LEA’s General Arrangement drawing, it is understood that a cofferdam is proposed within the footprint of the replacement bridge abutments. Sub-excavation of the existing fill behind both abutments and approach embankments (and organic silt at the north abutment/approach) is discussed in Section 6.6.1. Driving sheet piles to a suitable depth of penetration will be required to mitigate base instability and minimize water inflow; dewatering and/or the use of a concrete tremie plug at the base of the excavation will be required to maintain the excavation integrity and stability. Alternatively, excavation of subsoils could be completed in wet conditions (i.e., without dewatering, with the excavation flooded) within the cofferdam. Sub-excavation for the TMB abutments is further discussed in Section 6.6.1.

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)	284.7	278.5	6.2
	South Abutment (NG-2, NG-7)	284.5	278.1	6.4

There should be a provision made in the Contract for dealing with varying pile lengths 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. The lengths given above should be considered minimum lengths.

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, 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 Hiley formula or pile dynamic analyzer (PDA) 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. An NSSP to amend OPSS 903 has been developed to address this requirement, for inclusion in the Contract Documents (see Appendix C for reference). In addition, MTO has recently developed SP903S06 to provide for detailed specifications for PDA testing and this document should be included in the contract (see Appendix C for reference).

6.3.1.3 Set Criteria and Pile Driving Note

Pile installation should be carried out in accordance with OPSS 903 (Deep Foundations).

Based on the presence of the cobbles and boulders within the till layer, 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 2 in Clause 3.3.3 of the Structural Manual (MTO, 2016) as follows:

- "Piles to be driven to bedrock".

The piles should be tapped to confirm they are seated on the 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

The design of steel pile foundations subjected to lateral loads should take into account such factors as the batter of the pile (if any), 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 moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. Lateral loading could be resisted fully or partially by the use of battered piles.

It is understood that an integral abutment foundation design is being considered for the replacement bridge. Based on the shallow soil conditions at this site, the integral abutment design should include the installation of 3 m long corrugated steel pipe (CSP) liners, with the annular space between the pile and the liner backfilled with uniformly graded, loose sand (as per the NSSP in Appendix C), so that the upper portion of the H-piles will be free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand.

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 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 2014 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).

For cohesionless soils:

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

where:

n_h = constant of horizontal subgrade reaction (kPa/m), as given below;
z = depth (m)
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)	CSP Liners (3 m)	284.7 (underside of pile cap) to 281.7	1,300
	Sand and Gravel to Gravelly Silty Sand Till, Compact to Very Dense	281.7 to 278.5	11,000
South Abutment (NG-2, NG-7)	CSP Liners (3 m)	284.5 (underside of pile cap) to 281.5	1,300
	Sand and Gravel to Gravelly Silty Sand Till, Dense	281.5 to 278.1	11,000

Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal reaction should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (CHBDC (2014) Commentary Section 6.11.2.2).

The upper zone of the soil (down to a depth below the pile cap equal to about 1.5xB (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 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 pad overlying the loose to compact sand. The recommended founding elevation of the granular pad at each foundation element is summarized below. Construction considerations for a granular pad are discussed further in Section 6.6.3.

Foundation Element (Relevant Boreholes)	Existing Ground Surface at Borehole Location (m)	LEA's Proposed TMB Underside of Abutment Footing Elevations (m)	Recommended Approximate Sub-excavation/Founding Elevation of the Granular Pad (m)
North Abutment (NG-6)	286.9	287.9	285.9 (Recommended 2 m thick pad)
South Abutment (NG-5)	287.8	288.1	286.1 (Recommended 2 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	3	200	125

The factored geotechnical resistances and the settlement are dependent on the footing size, depth of embedment and applied loads. 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 perpendicular to the base of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.10.4 and Section C6.10.4 of the CHBDC and its Commentary.

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 OPSS 902 (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 6.10.5 of the CHBDC (2014) applying the appropriate consequence and degree of site understanding factors as noted in Section 6.2. An unfactored coefficient of friction, tan δ_i', of 0.55 may be used at the interface between the base of the cast-in-place 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 adopted an equivalency of 25 mm of insulation for every 0.3 m reduction in soil cover.

As noted above, we understand the TMB will be constructed in the first construction season, left 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 granular pad and the flexibility of the TMB. However, if the TMB will be in operation for more than one 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 re-level the TMB as required.

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 E in accordance with Table 4.1 of the CHBDC (2014), 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 (2014) 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

Based on the values noted above and in accordance with Table 4.10 of the CHBDC 2014, 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 LEA, 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, respectively, 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, respectively, about 2.7 m and 1.9 m above existing grade along the proposed detour centreline.

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, we recommend that the existing fill and organic silty sand (north abutment/approach) be removed from below the footprint of the reconstructed embankment behind the abutments for the new permanent bridge, and for the new/widened detour embankment sections. 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 provided by LEA. The piezometric conditions used in the analyses are based on the groundwater level as encountered during the subsurface investigation.

During our foundation investigation, we observed surficial sloughing at the east side of the north approach embankment; based on LEA's 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, LEA is considering the use of rock fill for the south approach embankment within the immediate vicinity of the south abutment (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, 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. If required, rock fill can also be used for the north approach embankment immediately behind the north abutment / granular fill.

6.4.1 Subgrade Preparation and Embankment Construction

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). The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading). Embankment side slopes for granular fill should be constructed no steeper than 2H:1V. Embankment side slopes for rock fill at the north approach, where required, 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) will be required 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.

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 (2014) 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 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 north and south approaches, settlement of the approximately 3 m high rock fill embankment will 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 the detour embankment construction.

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 (2014) Section 6.12.3 and Figure 6.6. 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 C6.20 (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 C6.20(b) of the Commentary to the CHBDC (2014).
- 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_o	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 C6.6 of the *Commentary to the CHBDC*, 2014..

- 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 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 m thickness for the granular pad.

Temporary protection systems will be required for protection of the existing bridge abutments and existing highway embankments 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.

Open-cut excavations must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA 2016) and Regulation for Construction Activities. The existing fill and organic soils are classified as Type 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. Open excavations below the water level are not recommended, except for the shallow excavation anticipated for the construction of the granular pad for the TMB abutments, as discussed in Section 6.6.3.

Excavations for the replacement bridge abutments will extend below the groundwater level. Due to the proximity of the abutments to the lake, a groundwater cut-off (cofferdam or similar measure) is recommended to minimize dewatering requirements and the occurrence of potential environmental impacts, as discussed further in Section 6.6.2. Dewatering of all excavations should be carried out in accordance with Special Provision FOUND003 (Dewatering Structure Excavations) using a survey radius of 500 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 of the CSP liners against basal heave/disturbance due to groundwater pressures, and the integrity of the sand fill once placed within the CSP liners. A Notice to Contractor should also be included in the Contract to alert the contractor to the groundwater conditions and that the excavation must be unwatered and kept stable during pile cap construction, including placement of the CSP liners for the integral abutments; an example Notice to Contractor is included in Appendix C.

The sand 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. We anticipate that the CSPs will be installed after coring through the concrete for an integral abutment structure, although the CSPs may be able to be installed in appropriate templates prior to placement of the tremie concrete.

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation, 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 should be designed and constructed in accordance with OPSS.PROV 539 (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 should meet Performance Level 2 as specified in OPSS.PROV 539. If excavations must be completed for removals in close proximity to the existing or new foundations, it is recommended that such protection systems meet Performance Level 1b as specified in OPSS.PROV 539.

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. 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 to reduce the potential for differential settlement occurring.

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. An NSSP 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. 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 initially.

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.

6.6.7 Analytical Testing for Construction Materials

The results of an analytical test carried out on soils 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. Adam Core, P.Eng. and Mr. André Bom, P.Eng. Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer, Principal of Golder and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.

Report Signature Page

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Commercial Software

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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.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
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)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects (as amended)

Table 1: Evaluation of Foundation Alternatives for Replacement Bridge

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven steel H-piles	1	■ Straightforward construction. ■ Higher axial resistance compared to spread footings. ■ Allows for integral abutment design.	■ Potential for "hanging up" or damage to piles on cobbles and boulders within cohesionless deposits; larger or heavier pile sections recommended to minimize damage. ■ Requires shoring system to excavate through the fill and organic soils (below groundwater level) adjacent to proposed TMB for pile cap construction.	■ Relative costs higher than shallow foundations. ■ Mobilization of piling equipment relatively expensive. ■ Crane pad improvement works may be required to support pile driving equipment, depending on location relative to existing highway embankment ■ Larger or heavier pile due to cobbles and boulders more expensive than standard piles.	■ Potential for variation in pile length to reach bedrock in order to achieve design resistances ■ Larger piles to minimize risk of "hanging" up on cobbles and boulders; a low to moderate risk remains that the piles may not reach bedrock, and a lower geotechnical resistance has been recommended for use in design, supplemented with provision for driving one additional pile, appropriately mitigating this risk
Small diameter drilled steel casings socketted into bedrock using DTH drilling	2	■ Highly suited to penetrate through the till including cobbles and boulders to bedrock. ■ Higher axial resistance compared to steel H-piles and spread footings. ■ Reduced vibrations on TMB bridge compared with pile driving. ■ Better suited for installation on steeply sloping bedrock surface, although this is not an issue at this site.	■ Requires specialized drilling equipment. ■ Not compatible with integral abutment design. ■ Would require more onerous management of cuttings/drilling fluid to prevent discharge of these materials into the river. ■ Requires similar shoring system and excavations as steel H-piles.	■ Mobilization of specialized equipment relatively expensive. ■ Higher cost than steel H-piles due to requirement for casings to remain in place. ■ Higher cost due to need to dispose of drilling fluid/cuttings off site.	■ Potential impact on river water quality due to cuttings/drilling fluid release. ■ Requires off-site disposal area for disposal of drilling fluid and cutting.

Table 1: Evaluation of Foundation Alternatives for Replacement Bridge

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Shallow foundations founded on the compact to very dense till	NP	■ Conventional construction, except for deep excavation that would be required for this site.	■ Not practical to sub-excavate to the relatively deep depth to reach the till to achieve adequate geotechnical axial resistances ■ Would require more significant dewatering or other means of groundwater control compared with other options, due to depth of excavation required ■ Not suitable for integral abutment design. ■ Variable till conditions at abutments potentially results in differential settlement.	■ Typically lower relative cost than deep foundations. ■ Cost would rise substantially due to difficulties associated with relatively deep shoring and unwatering system installation/ operation.	■ Potential difficulties installing shoring and unwatering system; and increased costs for unwatering, potential for claims. ■ Potential impact of deep excavations on “perched” footings for TMB. ■ Potential for differential settlement between foundation units.

NP = Not Practical

Table 2: Evaluation of Foundation Alternatives for Temporary Modular Bridge

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Shallow foundations founded on a granular pad	1	■ Conventional construction. ■ TMB structures are more tolerant to total and differential settlements, if applicable, compared to permanent structures. ■ Granular 'B' Type II pad can be placed in wet conditions if necessary.	■ Reduced capacity when compared with driven steel H-piles. ■ Greater potential for differential settlement/movement relative to steel H-pile foundations; however, this risk is low and is not expected to be a significant disadvantage for the temporary structure ■ Some risk of being impacted by excavations for subexcavation of existing fill/organic soil at the approaches for the replacement structure	■ Typically lower relative cost than deep foundations.	■ Low risk of differential settlement between TMB foundation units. ■ Low risk of being impacted by excavations for adjacent permanent structure, provided appropriate protection systems are implemented where required
Driven steel H-piles	2	■ Straightforward construction. ■ Higher axial resistance compared to spread footings. ■ Similar foundation support systems as replacement bridge. Pile foundations have lower risk of being affected by adjacent deep excavations for subexcavation of organic materials below approach embankments for replacement structure.	■ Potential for "hanging up" on cobbles and boulders within cohesionless deposits; larger pile sections required.	■ Relative costs higher than shallow foundations. ■ Mobilization of piling equipment relatively expensive.	■ Some risk of vibrations from pile driving affecting existing bridge, which is in poor condition.

NP = Not Preferred

MINISTRY OF TRANSPORTATION, ONTARIO

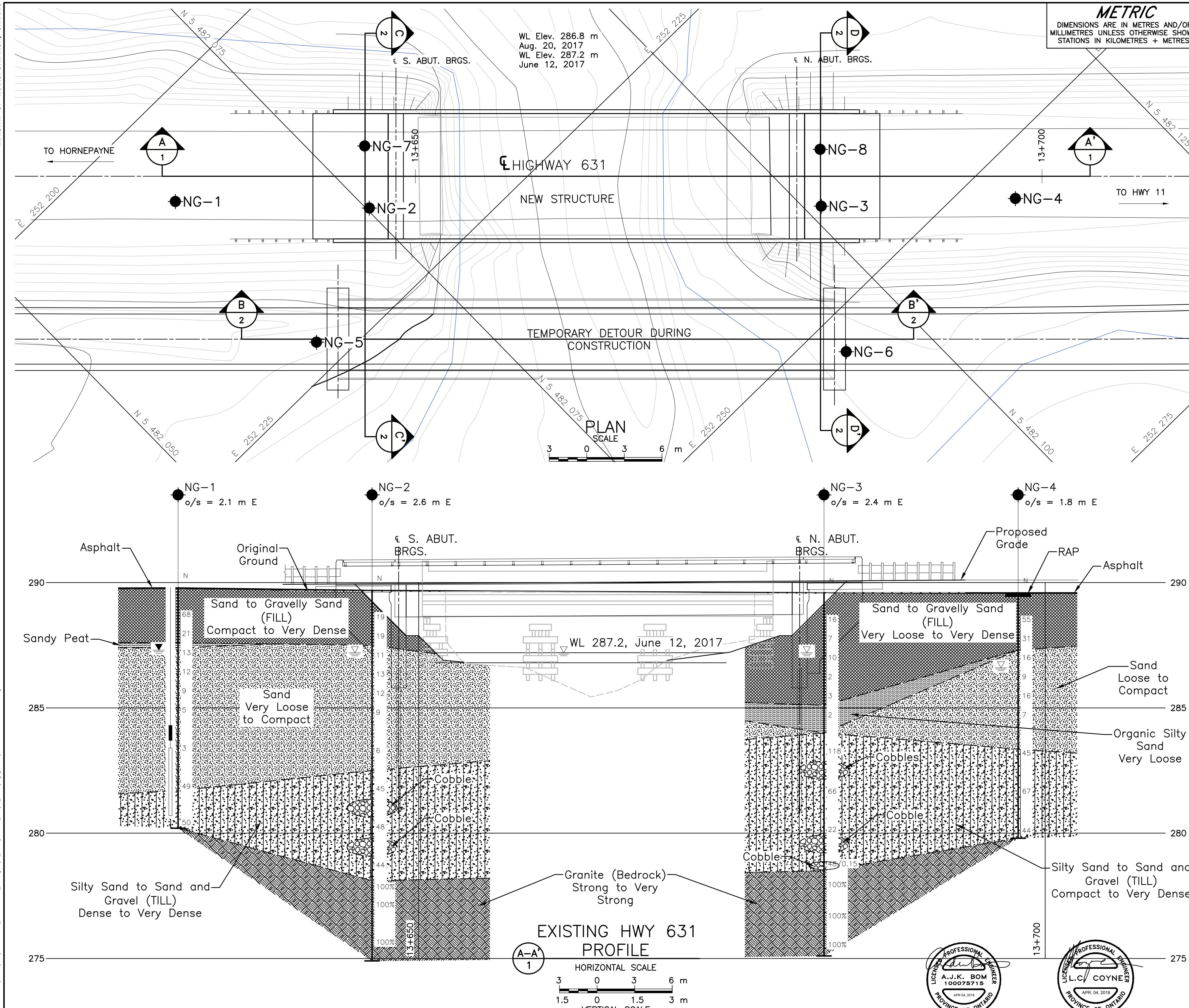
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

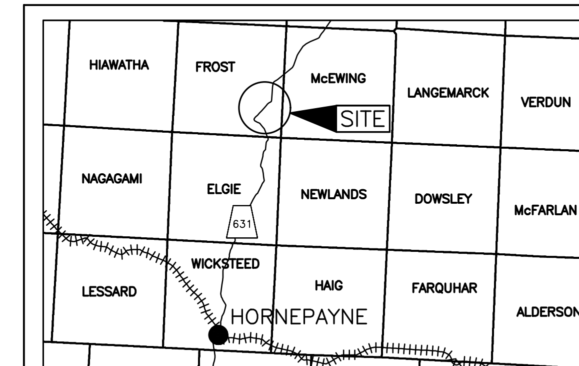
CONT No.
WP No. 5312-14-01

SHEET

HWY 631
NAGAGAMISIS NARROWS BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates





KEY PLAN
SCALE
10 0 10 20 km

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 JUN 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.

Cobbles shown on this drawing are based on the conditions encountered in the boreholes. Cobbles and/or boulders are to be expected randomly throughout the till deposit.

REFERENCE

Base plans provided in digital format by LEA Consulting LTD., drawing file nos. 17197-Nagagamisis-General Arrangement.dwg and 17197-Detour General Arrangement-D1.dwg, received AUG 15, 2017.

LICENSED PROFESSIONAL ENGINEER A.J.K. BOM 100075715 APR 04, 2018 PROVINCE OF ONTARIO

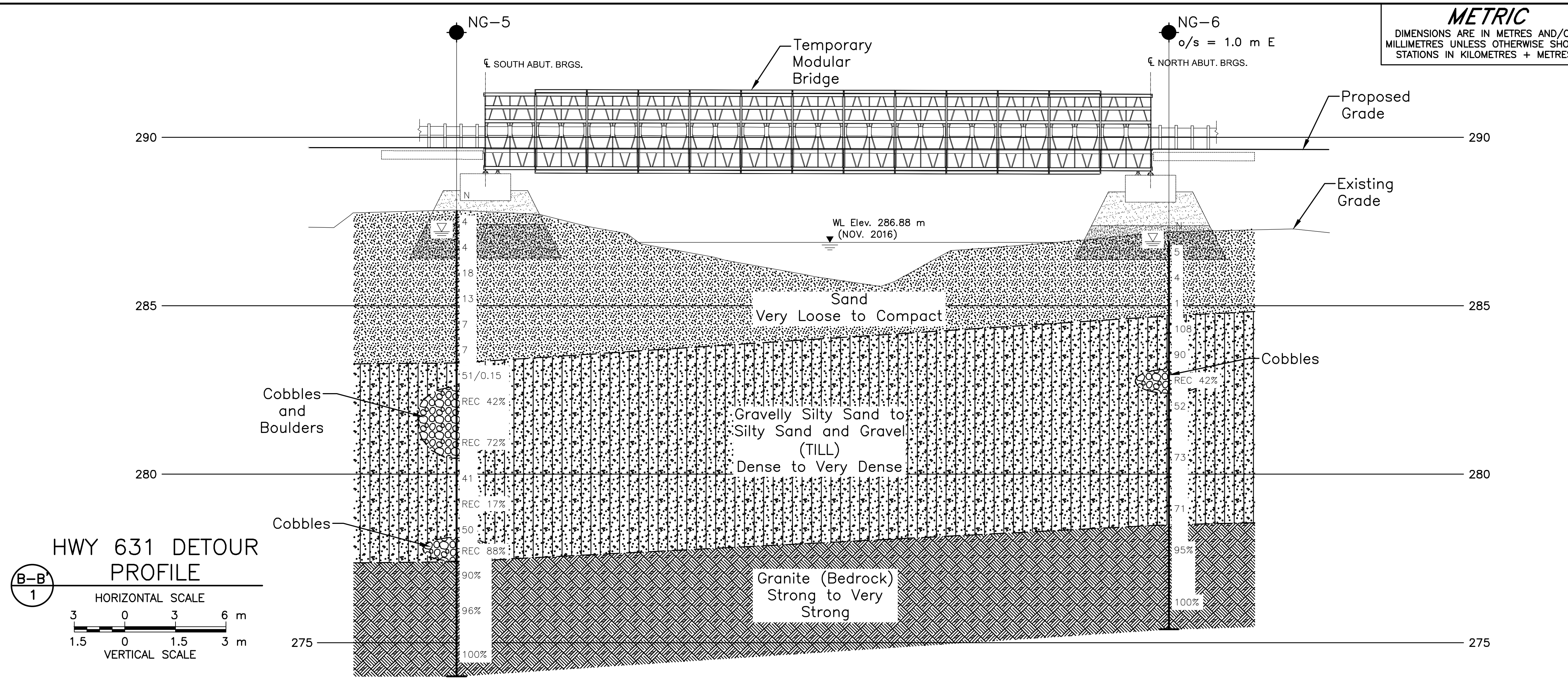
LICENSED PROFESSIONAL ENGINEER L.C. COYNE APR. 04, 2018 PROVINCE OF ONTARIO

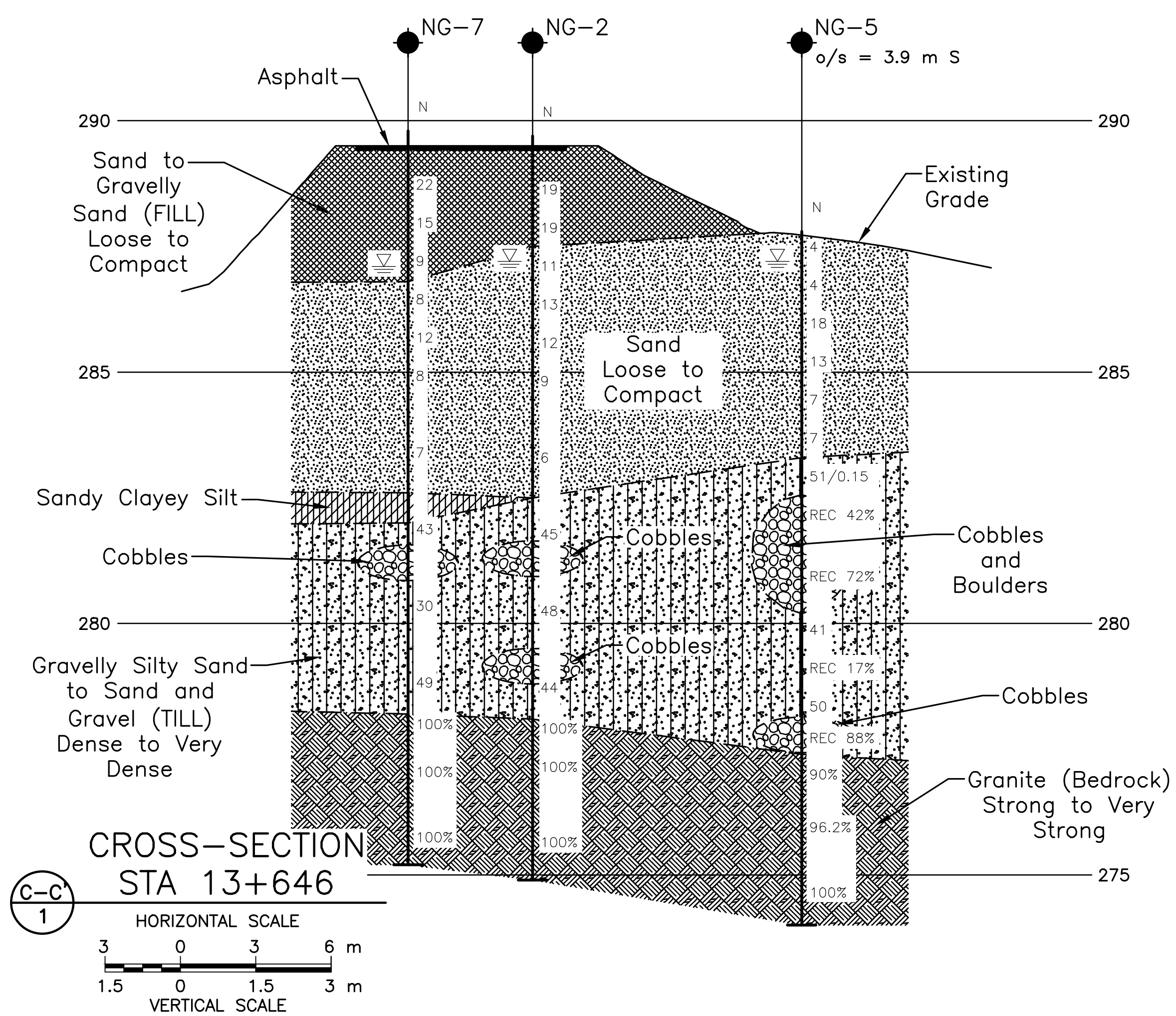
NO.	DATE	BY	REVISION
.	.	.	

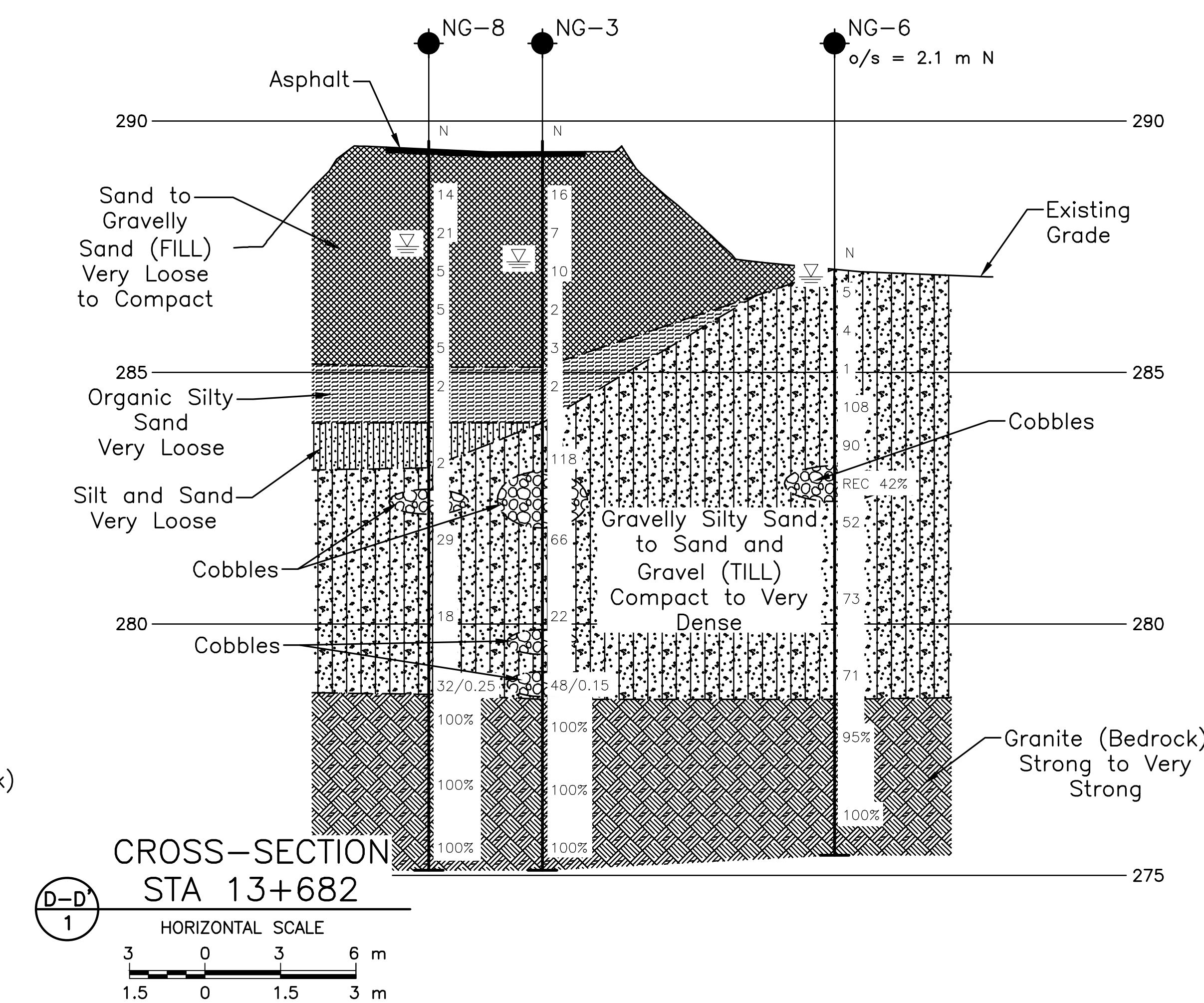
Geocres No. 42F-054

HWY. 631	PROJECT NO. 1661607		DIST.
SUBM'D.	CHKD. AC	DATE: 4/3/2018	SITE: 38N-001
DRAWN: JJL/TB	CHKD. AB	APPD. LCC	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO







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

CONT No.
WP No. 5312-14-01

HWY 631
NAGAGAMISIS NARROWS BRIDGE
SOIL STRATA

SHEET

Golder Associates

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 (NAD83 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

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LICENSED PROFESSIONAL ENGINEER A.J.K. BOM 100075715 APR. 04, 2018 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE APR. 04, 2018 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 42F-054

HWY. 631		PROJECT NO. 1661607	DIST.
SUBM'D.	CHKD. AC	DATE: 4/3/2018	SITE: 38N-001
DRAWN: TB	CHKD. AB	APPD. LCC	DWG. 2

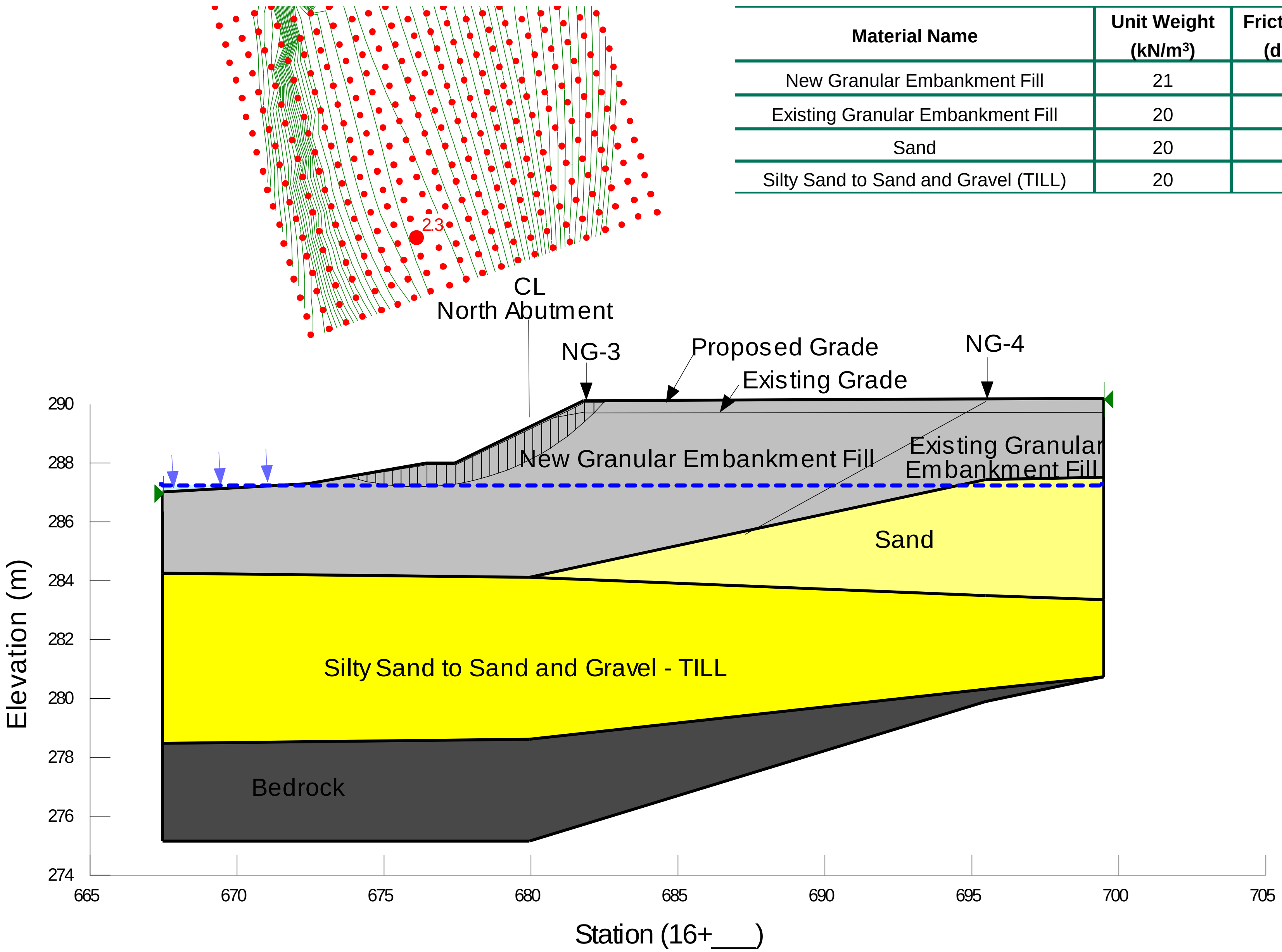
Photograph 1: Nagagamisis Lake Bridge
East elevation Looking North-West (May 2017)

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

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)

Material Name	Unit Weight (kN/m³)	Friction Angle (degrees)
New Granular Embankment Fill	21	35
Existing Granular Embankment Fill	20	32
Sand	20	30
Silty Sand to Sand and Gravel (TILL)	20	32



APPENDIX A

Record of Boreholes

LIST OF SYMBOLS

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
υ	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

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

(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 $= (w_l - w_p)$
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_α	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
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan\delta$
c′	effective cohesion
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	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. 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.) drive open sampler for a distance of 300 mm (12 in.)

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, 60º 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

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index Relative Density	**N Blows/300 mm or Blows/ft**
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
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
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	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	**Modifier**	**Example**
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

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, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

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^o 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

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

Foundation Design

RECORD OF BOREHOLE No NG-1

 METRIC

PROJECT 1661607
W.P. 5312-14-01
LOCATION N 5482064.5; E 252206.8 MTM ZONE 12 (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

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
289.8	GROUND SURFACE						
0.0	ASPHALT (40 mm)						
0.2	RAP (110 mm)						
289.3 / 0.5	Gravelly sand (FILL) Brown Moist						
	Sand, trace to some gravel, trace to some silt (FILL) Compact to very dense Brown Moist	1	SS	68	289		
		2	SS	21	288	○	18 74 (8)
287.6 / 2.4	Sandy PEAT, some silt Black Wet						
	SAND, trace gravel, trace silt Very loose to dense Brown to grey Wet	3	SS	13	287		
		4	SS	12		○	
	200 mm to 300 mm of sand heaving in augers at Samples 5 to 7.	5	SS	9	286	○	6 89 (5)
		6	SS	5	285		
					284		
		7	SS	3	283		
281.7 / 8.1		8	SS	49	282	○	5 90 (5)
	Silty SAND, some gravel, trace clay (TILL) Dense to very dense Grey Wet. Augers grinding from 8.1 m to 9.1 m depth.				281		
280.2 / 9.6		9	SS	50		○	10 53 33 4

END OF BOREHOLE

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.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

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

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

Golder Associates

RECORD OF BOREHOLE No NG-2

 METRIC

PROJECT 1661607

W.P. 5312-14-01 LOCATION N 5482075.2; E 252218.0 MTM ZONE 12 (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

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
289.7	GROUND SURFACE							
0.0	ASPHALT (40 mm)							
0.2	RAP (110 mm)							
	Gravelly sand (FILL) Brown Moist							
289.0 / 0.7	Sand, some gravel, some silt (FILL) Compact Brown Moist	1	SS	19		289		
		2	SS	19		288	○	
287.5 / 2.2	SAND, trace gravel, trace silt Loose to compact Brown to grey Wet	3	SS	11	▽	287	○	8 86 (6)
		4	SS	13		286		
		5	SS	12				
	150 mm to 200 mm of sand heaving in augers at Samples 6 and 7.	6	SS	9		285	○	11 85 (4)
						284		
		7	SS	6		283		
282.5 / 7.2	SAND and GRAVEL to Gravelly Silty SAND, trace clay (TILL) Dense Gret Wet							
	Four 100 mm diameter cobbles encountered from 8.2 m to 9.1 m depth.	8	SS	45		282	○	57 36 6 1
						281		
		9	SS	48		280		
	Two 75 mm diameter cobbles encountered at 11.2 m and 11.3 m depth.	10	SS	44		279		
278.1 / 11.6		1	RC	REC 100%		278		RQD = 100%
		2	RC					RQD = 100%

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

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

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

Golder Associates

RECORD OF BOREHOLE No NG-2

 METRIC

PROJECT 1661607
W.P. 5312-14-01
LOCATION N 5482075.2; E 252218.0 MTM ZONE 12 (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

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	GRANITE (BEDROCK) Bedrock cored from 11.6 m depth to 14.8 m depth. For coring details see Record of Drillhole NG-2.		2	RC	REC 100%		277				RQD = 100%
			3	RC	REC 100%		276				RQD = 100%
274.9							275				
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\MTO\1661607 LEA_5015-E-0049_NE REGION\02_DATA\GINT\1661607.GPJ GAL-MISS.GDT 11/8/17 TB/JJL

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: NG-2

PROJECT: 1661607
LOCATION: N 5482075.2; E 252218.0
MTM ZONE 12 (LAT. 49.474243; LONG. -81.725599)
INCLINATION: -90° AZIMUTH: ---

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

DATUM: GEODETIC

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 METRES	DISCONTINUITY DATA B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE																		
	CME 55 Truck Mount; NQ Coring; NW	GRANITE Very strong Fresh Fine to medium grained Grey to black / pink		278.1 11.6	1	GREY 100														
12					2	LIGHT GREY 100														
13																				UCS = 146 MPa
14				274.9	3	LIGHT GREY 100														
15		END OF DRILLHOLE		14.8																
16																				
17																				
18																				
19																				
20																				
21																				
22																				
23																				

DEPTH SCALE
1 : 60

LOGGED: SA
CHECKED: AB

RECORD OF BOREHOLE No NG-3

 METRIC

PROJECT 1661607
W.P. 5312-14-01
DIST ___ HWY 631
DATUM GEODETIC
LOCATION N 5482101.1; E 252243.1 MTM ZONE 12 (LAT. 49.474478; LONG. -81.725256)
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing and NQ Coring
DATE June 10, 2017
ORIGINATED BY MR
COMPILED BY AC
CHECKED BY AC/AB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
289.6	GROUND SURFACE							
0.0	ASPHALT (25 mm)							
	RAP (100 mm)							
	Gravelly sand (FILL)							
	ASPHALT (50 mm)					289		
0.6	RAP (50 mm)							
	Gravelly sand (FILL)							
	Sand, trace gravel (FILL) Very loose to compact Grey Wet	1	SS	16				
		2	SS	7		288		
					▽			
		3	SS	10		287	~20	8 86 (6)
		4	SS	2		286		
		5	SS	3				
285.1 4.5	ORGANIC Silty SAND, trace clay, trace gravel Very loose Black Wet	6	SS	2		285		OC = 13.5% 2 68 29 1
284.0 5.6	Silty SAND to SAND and GRAVEL, trace clay (TILL) Compact to very dense Brown to grey Wet					284		
		7	SS	118		283		
	Cobbles encountered at the following depths and sizes:					282		
		8	SS	66			~13	14 54 28 4
						281		
		9	SS	22		280		
						279		
		10	SS	48/0.15			~17	58 36 (6)
278.5 11.1		1	RC	REC 100%		278		RQD = 100%

Depth (m)	Size (mm)
6.6	230
6.8	100
6.9	130
7.5	150
10.5	150
11.0	150

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No NG-3

 METRIC

PROJECT 1661607

W.P. 5312-14-01 LOCATION N 5482101.1; E 252243.1 MTM ZONE 12 (LAT. 49.474478; LONG. -81.725256) 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 June 10, 2017 CHECKED BY AC/AB

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	GRANITE (BEDROCK)		1	RC							RQD = 100%
	Bedrock cored from 11.1 m depth to 14.5 m depth. For coring details see Record of Drillhole NG-3.		2	RC	REC 100%		277				RQD = 100%
			3	RC	REC 100%		276				RQD = 100%
275.1 / 14.5	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.										

$+^{3}$, $\times^{3}$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

PROJECT: 1661607

RECORD OF DRILLHOLE: NG-3

LOCATION: N 5482101.1; E 252243.1
MTM ZONE 12 (LAT. 49.474478; LONG. -81.725256)
INCLINATION: -90° AZIMUTH: ---

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

DATUM: GEODETIC

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE		278.5																	
	LC CME 55 Track Mount; NW; NQ Coring	GRANITE Very strong Fresh Fine to medium grained Light grey to black		11.1	1		GREY 100														UCS = 128 MPa
12																					
13					2		GREY 100														
14				275.1 14.5	3		GREY 100														
		END OF DRILLHOLE																			
15																					
16																					
17																					
18																					
19																					
20																					
21																					
22																					
23																					

DEPTH SCALE
1 : 60

Golder Associates

LOGGED: MR
CHECKED: AC/AB

RECORD OF BOREHOLE No NG-4

 METRIC

PROJECT 1661607			
W.P. 5312-14-01	LOCATION N 5482112.6; E 252253.5 MTM ZONE 12 (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

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	Water content w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
289.6	GROUND SURFACE						
0.0	ASPHALT (35 mm)						
	RAP (110 mm)						
0.3	Gravelly sand (FILL) Brown Moist						
	Sand, trace gravel, some silt (FILL) Dense to very dense Brown Moist	1	SS	55			
		2	SS	31		o	3 79 (18)
287.3 / 2.3	SAND, trace gravel Loose to compact brown Wet	3	SS	16			
					▽ (Elev. 286.6)		
		4	SS	9			
	A 0.6 m sand and gravel layer was encountered at 3.8 m depth.	5	SS	16		o	45 48 (7)
		6	SS	7			
283.4 / 6.3	Silty SAND, some gravel, trace clay (TILL) Dense to very dense Grey Wet	7A / 7B	SS	45		o	18 54 24 4
		8	SS	67			
		9	SS	44			
279.8 / 9.8	END OF BOREHOLE						

Elevation scale: 289, 288, 287, 286, 285, 284, 283, 282, 281, 280

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No NG-5

 METRIC

PROJECT 1661607			
W.P. 5312-14-01	LOCATION N 5482064.7; E 252222.7 MTM ZONE 12 (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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
287.8	GROUND SURFACE						
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		287	
		3	SS	18		286	18 75 (7)
		4	SS	13		285	
		5	SS	7			
		6	SS	7		284	3 94 (3)
283.3 4.5	Gravelly Silty SAND, trace clay (TILL) Dense to very dense Grey Wet	7	SS	51/0.15		283	
	Cobbles and boulders were encountered at the following depths and sizes:	-	RC	REC 42%		282	
	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%		281	
	No recovery in Samples 8 and 9 after 2 attempts.	8	SS	41		280	
		-	RC	REC 17%		279	
		9	SS	50		278	
		-	RC	REC 88%			
277.4 10.4	GRANITE (BEDROCK) Bedrock cored from 10.4 m depth to 13.8 m depth. For coring details see Record of Drillhole NG-5.	1	RC	REC 100%		277	RQD = 90%
		2	RC	REC 100%		276	RQD = 96%

Natural moisture content (%) plotted: Sample 1 ≈ 4; Sample 3 ≈ 9; Sample 6 ≈ 13.

Column headings also include: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; STRAT PLOT.

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No NG-5

 METRIC

PROJECT 1661607			
W.P. 5312-14-01	LOCATION N 5482064.7; E 252222.7 MTM ZONE 12 (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

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	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%
							275				
			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.						274				

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+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF DRILLHOLE: NG-5

PROJECT: 1661607
LOCATION: N 5482064.7; E 252222.7
MTM ZONE 12 (LAT. 49.474149; LONG. -81.725532)
INCLINATION: -90° AZIMUTH: ---

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

DATUM: GEODETIC

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	COLOUR / % RETURN	DISCONTINUITY DATA – TYPE AND SURFACE DESCRIPTION	NOTES / WATER LEVELS / INSTRUMENTATION
		REFER TO PREVIOUS PAGE	277.4				
	Boart Longyear LF-70 DD; NW / NQ CORING	GRANITE Strong Fresh Fine to medium grained Grey to black	10.4	1	GREY 100%	JNIRRo	
11–12				2	GREY 100%	JNIRRo	UCS = 94 MPa
13			274.0 / 13.8	3	GREY 100%		
14		END OF DRILLHOILE	13.8				

Column headings: RECOVERY (TOTAL CORE %, SOLID CORE %), R.Q.D. %, FRACT. INDEX METRES, DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION, Jr, Ja, Jn), HYDRAULIC CONDUCTIVITY k, cm/s, Diametral Point Load Index (MPa), RMC -Q' AVG., FLUSH, SYMBOLIC LOG

Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

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DEPTH SCALE
1 : 60

Golder Associates

LOGGED: MR
CHECKED: AB

Golder Associates

RECORD OF BOREHOLE No NG-6

 METRIC

PROJECT 1661607

W.P. 5312-14-01 LOCATION N 5482094.4; E 252252.8 MTM ZONE 12 (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

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
286.9	GROUND SURFACE					
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						

Column headings also include: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (286–276); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

Continued Next Page

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

RECORD OF BOREHOLE No NG-6

 METRIC

PROJECT 1661607

W.P. 5312-14-01 LOCATION N 5482094.4; E 252252.8 MTM ZONE 12 (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 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE Note: 1. Water level at ground surface (Elev. 286.9 m) upon completion of drilling.										

$+^{3}, \times^{3}$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF DRILLHOLE: NG-6

PROJECT: 1661607
LOCATION: N 5482094.4; E 252252.8
MTM ZONE 12 (LAT. 49.474419; LONG. -81.725121)
INCLINATION: -90° AZIMUTH: ---

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

DATUM: GEODETIC

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	COLOUR % RETURN	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE	278.5			
9–11	Boart Longyear LF-70 DD; NQ CORING; NW	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
				2	GREY 100%	
		END OF DRILLHOLE	275.4 11.5			

Depth scale: 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

Column headings also include: SYMBOLIC LOG; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %: 80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX METRES (5 10 15 20); DISCONTINUITY DATA (B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr; Ja; Jn); HYDRAULIC CONDUCTIVITY k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.

DEPTH SCALE
1 : 60

Golder Associates

LOGGED: MR
CHECKED: AB

Golder Associates

RECORD OF BOREHOLE No NG-7

 METRIC

PROJECT 1661607			
W.P. 5312-14-01	LOCATION N 5482078.4; E 252214.2 MTM ZONE 12 (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

ELEV / DEPTH (m)	DESCRIPTION
289.8	GROUND SURFACE
0.0	ASPHALT (50 mm)
	Gravelly sand (FILL)
0.3	ASPHALT (50 mm)
	Sand, trace to some gravel, trace to some silt (FILL) Loose to compact Brown Moist to wet
286.8 3.0	SAND, trace to some gravel, trace silt Loose to compact Brown Wet Trace organics noted in Sample 4.
282.6 7.2	Sandy CLAYEY SILT, trace gravel Loose to compact Grey Wet
282.0 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.
278.2 11.6	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	22	
2	SS	15	9 78 (13)
3	SS	9	
4	SS	8	14 79 (7)
5	SS	12	
6	SS	8	
7	SS	7	5 92 (3)
8A			6 20 66 8
8B	SS	43	
9	SS	30	
10	SS	49	36 51 13 0
1	RC	REC 100%	RQD = 100%

GROUND WATER CONDITIONS: water level symbol at approx. elev. 287.1 (Sample 3)

ELEVATION SCALE: 289, 288, 287, 286, 285, 284, 283, 282, 281, 280, 279, 278

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No NG-7

 METRIC

PROJECT 1661607			
W.P. 5312-14-01	LOCATION N 5482078.4; E 252214.2 MTM ZONE 12 (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

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	GRANITE (BEDROCK) Bedrock cored from 11.6 m depth to 14.6 m depth. For coring details see Record of Drillhole NG-7.		2	RC	REC 100%		277				RQD = 100%
			3	RC	REC 100%		276				RQD = 100%
275.2 14.6	END OF BOREHOLE Note: 1. Water level at a depth of 2.7 m below ground surface (Elev. 287.1 m) upon completion of drilling.										

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

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

PROJECT: 1661607
LOCATION: N 5482078.4; E 252214.2
MTM ZONE 12 (LAT. 49.474272; LONG. -81.725651)
INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: NG-7

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

DATUM: GEODETIC

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE				
	CME 55 Truck Mount; NQ CORING; NW	GRANITE Very strong Fresh Medium to coarse grained Light grey	278.2 / 11.6	1	GREY 100%	UCS = 180 MPa
12						
13				2	GREY 100%	
14			275.2	3	GREY 100%	
		END OF BOREHOLE	14.6			
15						
16						
17						
18						
19						
20						
21						
22						
23						

Other column headings: SYMBOLIC LOG; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX METRES; DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION, Jr, Ja, Jn); HYDRAULIC CONDUCTIVITY k, cm/s; Diametral Point Load Index (MPa); RMC -Q' AVG.

Golder Associates

RECORD OF BOREHOLE No NG-8

 METRIC

PROJECT 1661607

W.P. 5312-14-01 LOCATION N 5482104.2; E 252239.8 MTM ZONE 12 (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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
289.6	GROUND SURFACE								
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					289			
		1	SS	14					
						288			
		2	SS	21	▽				
		3	SS	5		287	○		0 94 (6)
		4	SS	5		286			
	Trace organics at 3.8 m depth.	5	SS	5					
285.1 4.5	ORGANIC Silty SAND, trace wood Very loose Black Wet					285			
		6	SS	2			111.7		OC = 3.7%
284.0 5.6	SILT and SAND, trace clay, trace gravel, trace organics Very loose Grey Wet					284			
		7A	SS	2			○	NP	1 60 37 2
283.1 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.	7B				283			
						282			
		8	SS	29					
						281			
		9	SS	18		280			
						279			
278.6		10	SS	32/0.25			○		41 51 8 0
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%		278			RQD = 100%

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

Continued Next Page

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

Foundation Design

Golder Associates

RECORD OF BOREHOLE No NG-8

2 OF 3 **METRIC**

PROJECT 1661607
W.P. 5312-14-01
LOCATION N 5482104.2; E 252239.8 MTM ZONE 12 (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

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	GRANITE (BEDROCK) Bedrock cored from 11.0 m depth to 14.5 m depth. For coring details see Record of Drillhole NG-8.		2	RC	REC 100%		277				RQD = 100%
			3	RC	REC 100%		276				RQD = 100%
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.										

Dynamic cone scale: 20 40 60 80 100. Shear strength: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; water content 20 40 60.

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

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT: 1661607

LOCATION: N 5482104.2; E 252239.8

MTM ZONE 12 (LAT. 49.474506; LONG. -81.725302)

INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: NG-8

DRILLING DATE: May 29, 2017

DRILL RIG: CME 55 Truck Mount

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd

DATUM: GEODETIC

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH / COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX METRES	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE																		
11	LC CME 55 Track Mount; NW / NQ Coring	GRANITE Very strong Fresh Fine to medium grained Light grey		278.6 11.0	1	GREY 100%														
12–13					2	GREY 100%														UCS = 118 MPa
14				275.1 14.5	3	GREY 100%														
		END OF DRILLHOLE																		

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

Prepared by: AD
Reviewed by: AB

Notes:
1. Sample obtained May 29, 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

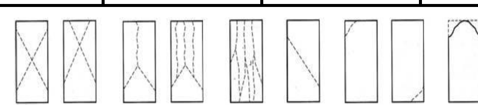
PROJECT NO.: 1661607

JOB NAME: Nagagamisis Narrows Bridge

TYPE OF UNIT: Bedrock Core

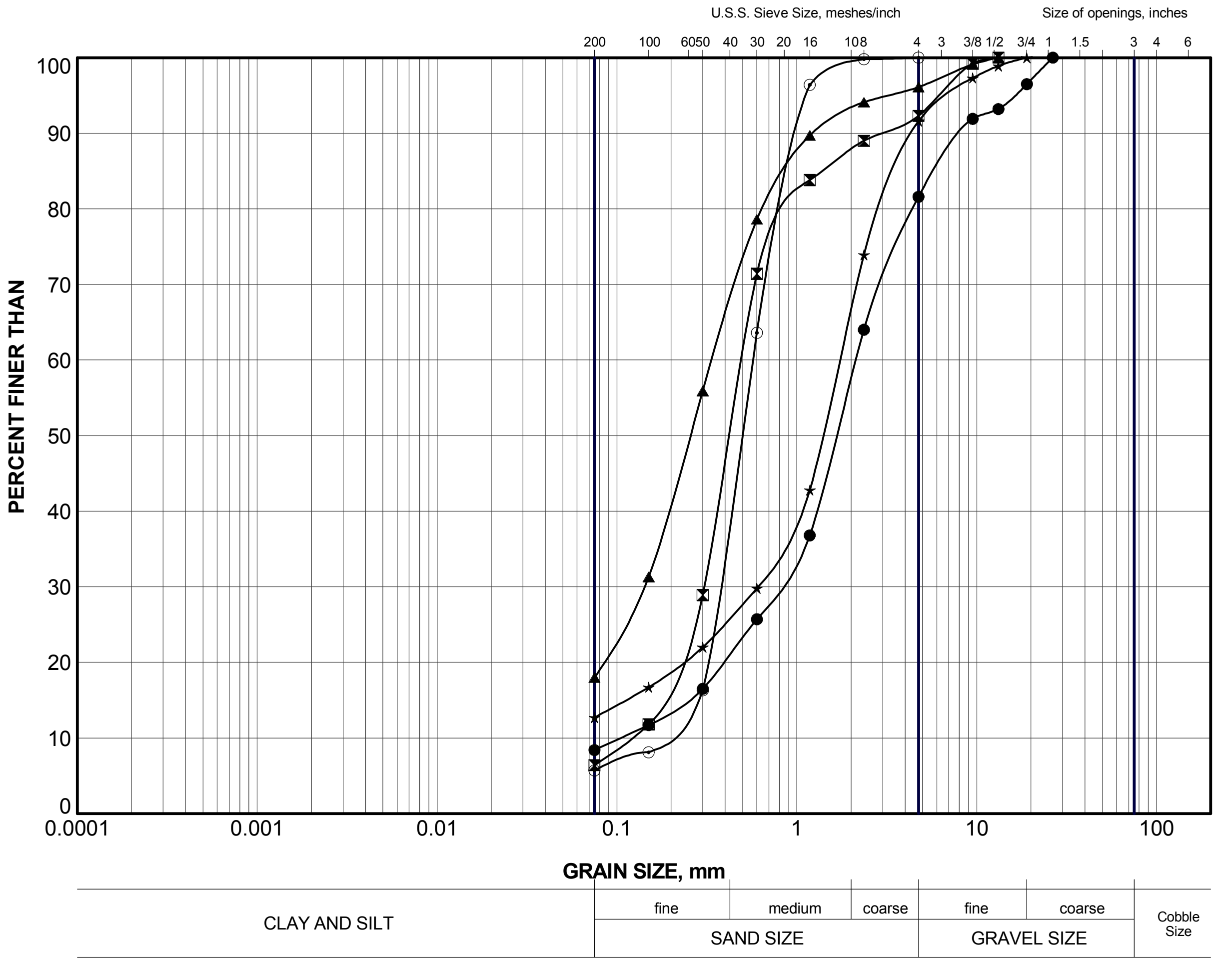
BOREHOLE	NG-2	NG-3	NG-5	NG-6	NG-7	NG-8
GOLDER LAB #	C1012	C1017	C1528	C1532	C647	C641
DATE TESTED	Jul. 25, 2017	Jul. 25, 2017	Sept. 19, 2017	Sept. 19, 2017	Jun. 9, 2017	Jun. 9, 2017
TESTED BY	JM/DM	JM/DM	JP	JP	EHS	EHS
DEPTH OF TESTED CORE (m)	13.1	11.3	11.9	9.8	11.9	12.5
LENGTH (mm)	100.9	97.5	100.3	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

Type of Fracture



1 2 3 4 5 6

Checked by : AB



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

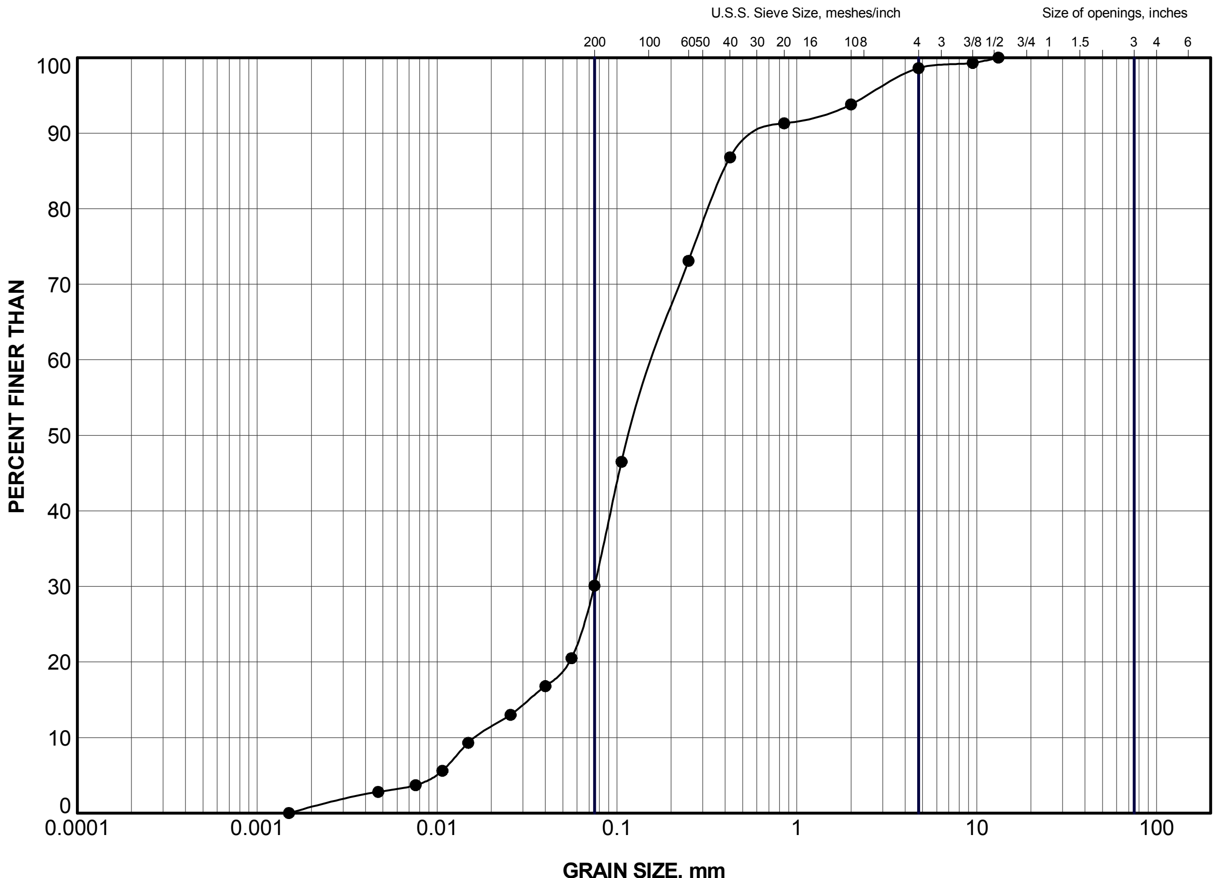
TITLE

GRAIN SIZE DISTRIBUTION

SAND (FILL)

Golder Associates
SUDBURY, ONTARIO

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



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NG-3	6	284.7

PROJECT

HIGHWAY 631
NAGAGAMISIS NARROWS BRIDGE

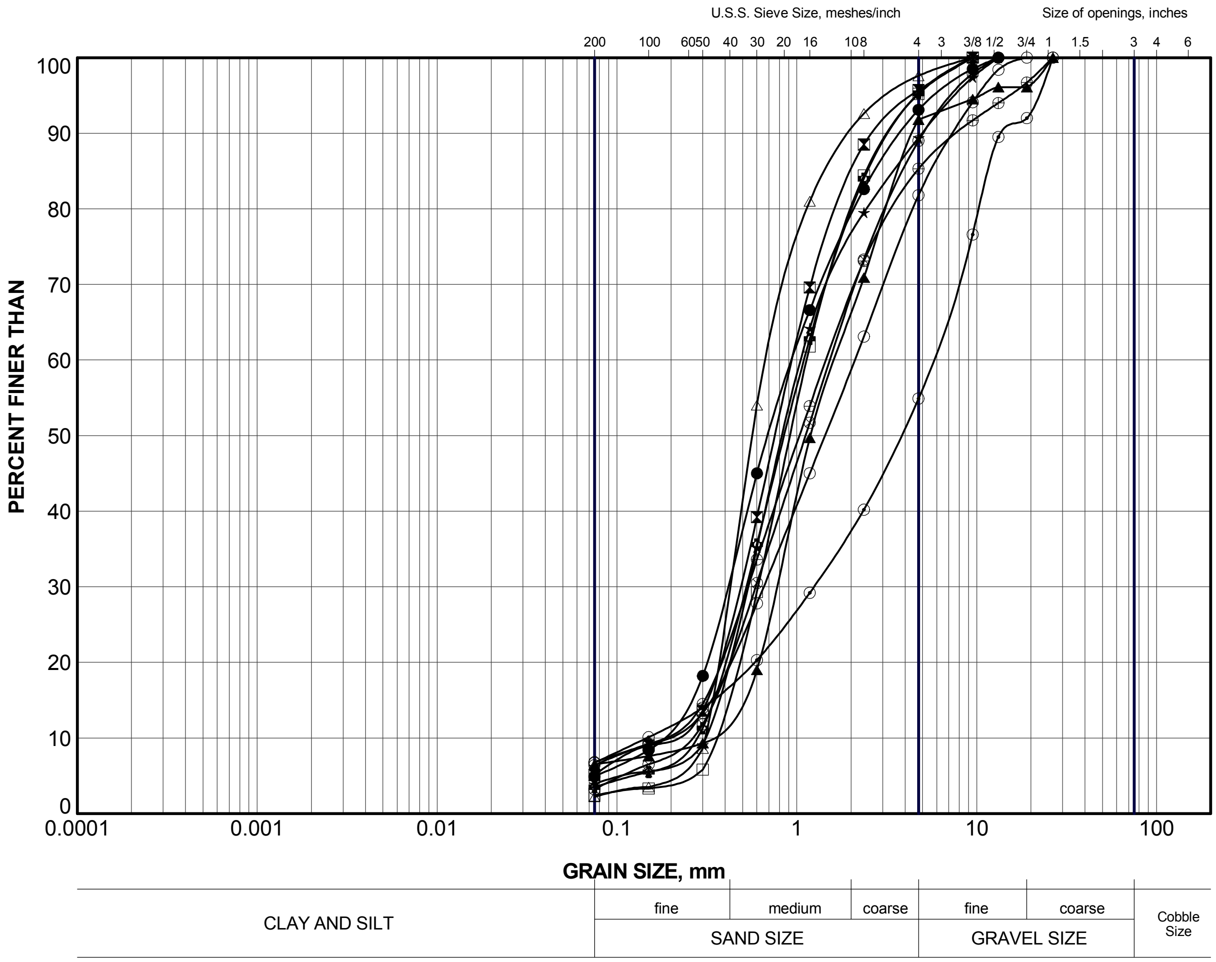
TITLE

GRAIN SIZE DISTRIBUTION
ORGANIC SILTY SAND

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1661607	FILE No.	1661607.GPJ	
DRAWN	TB	Oct 2017	SCALE	N/A	REV.
CHECK	AB	Oct 2017	FIGURE B2		
APPR		Oct 2017			

SUD-MTO GSD (2016) GLDR_LDN.GDT



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

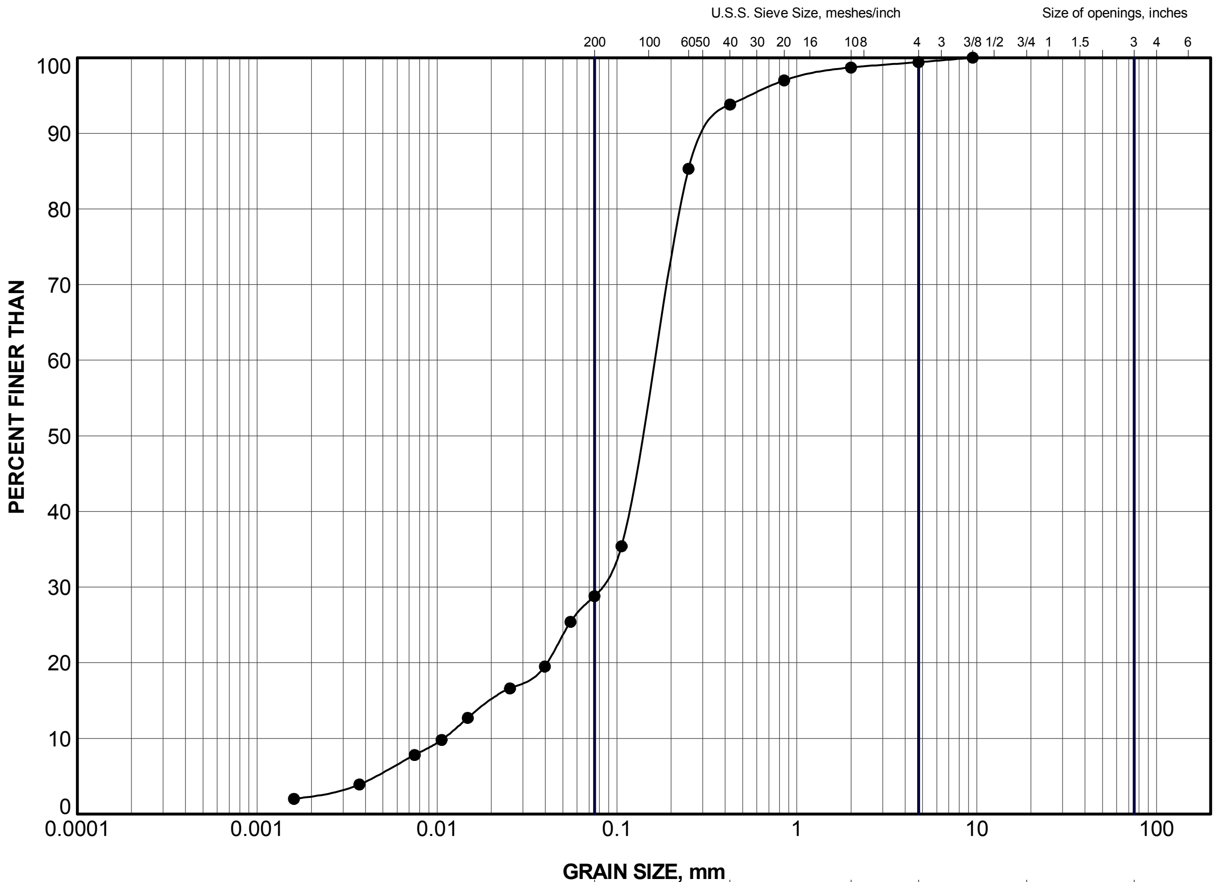
GRAIN SIZE DISTRIBUTION

SAND (and SAND and GRAVEL LAYER)

Golder Associates
SUDBURY, ONTARIO

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

SUD-MTO GSD (2016) GLDR_LDN.GDT



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

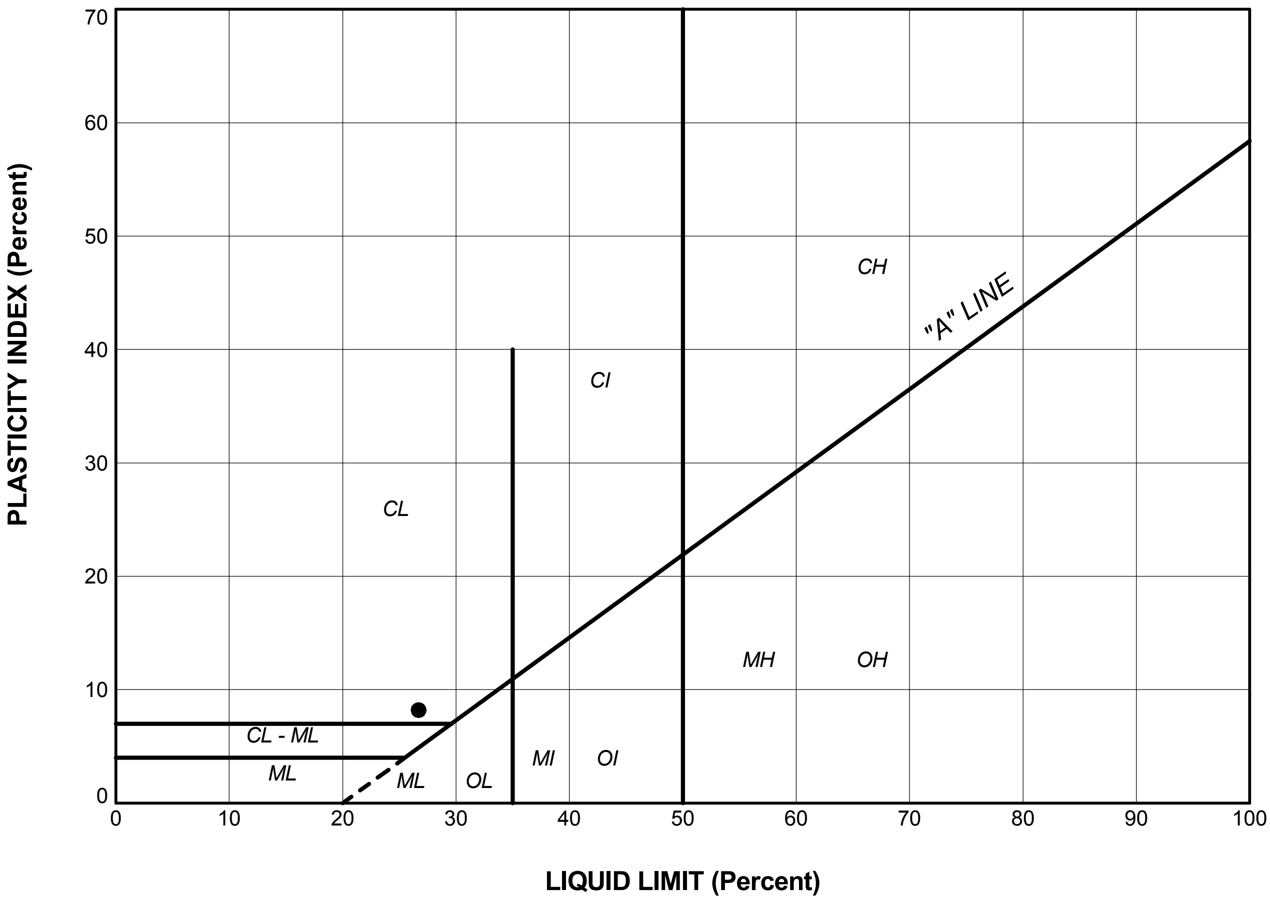
TITLE

GRAIN SIZE DISTRIBUTION
SILT and SAND

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1661607	FILE No.	1661607.GPJ
DRAWN	TB	Sep 2017	SCALE N/A	REV.
CHECK	AB	Sep 2017		
APPR		Sep 2017	FIGURE B4	

SUD-MTO GSD (2016) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

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

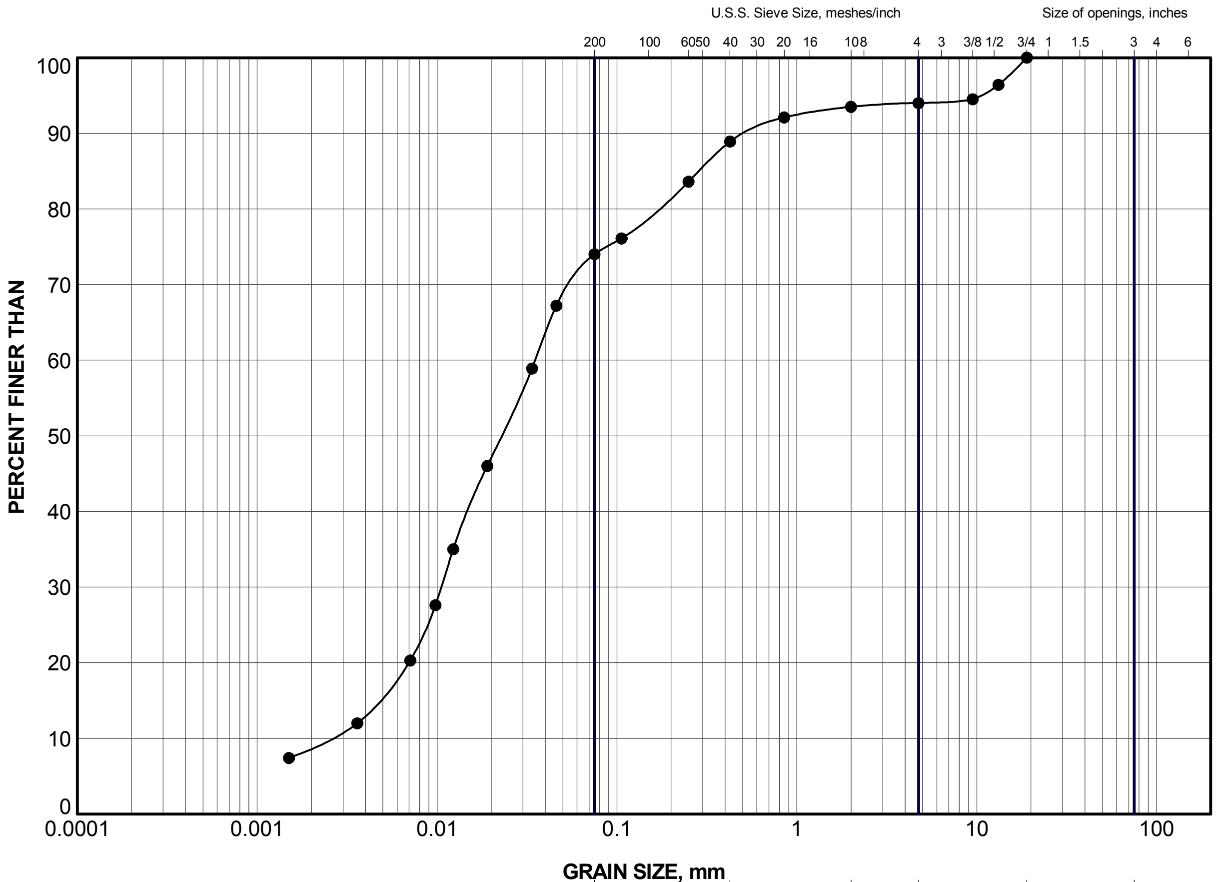
PROJECT
HIGHWAY 631
NAGAGAMISIS NARROWS BRIDGE

TITLE
PLASTICITY CHART
SANDY CLAYEY SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1661607	FILE No.	1661607.GPJ
DRAWN	TB	Oct 2017	SCALE N/A	REV.
CHECK	AB	Oct 2017	FIGURE B5	
APPR		Oct 2017		

SUD-MTO PL (2016) GLDR_LDN.GDT



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

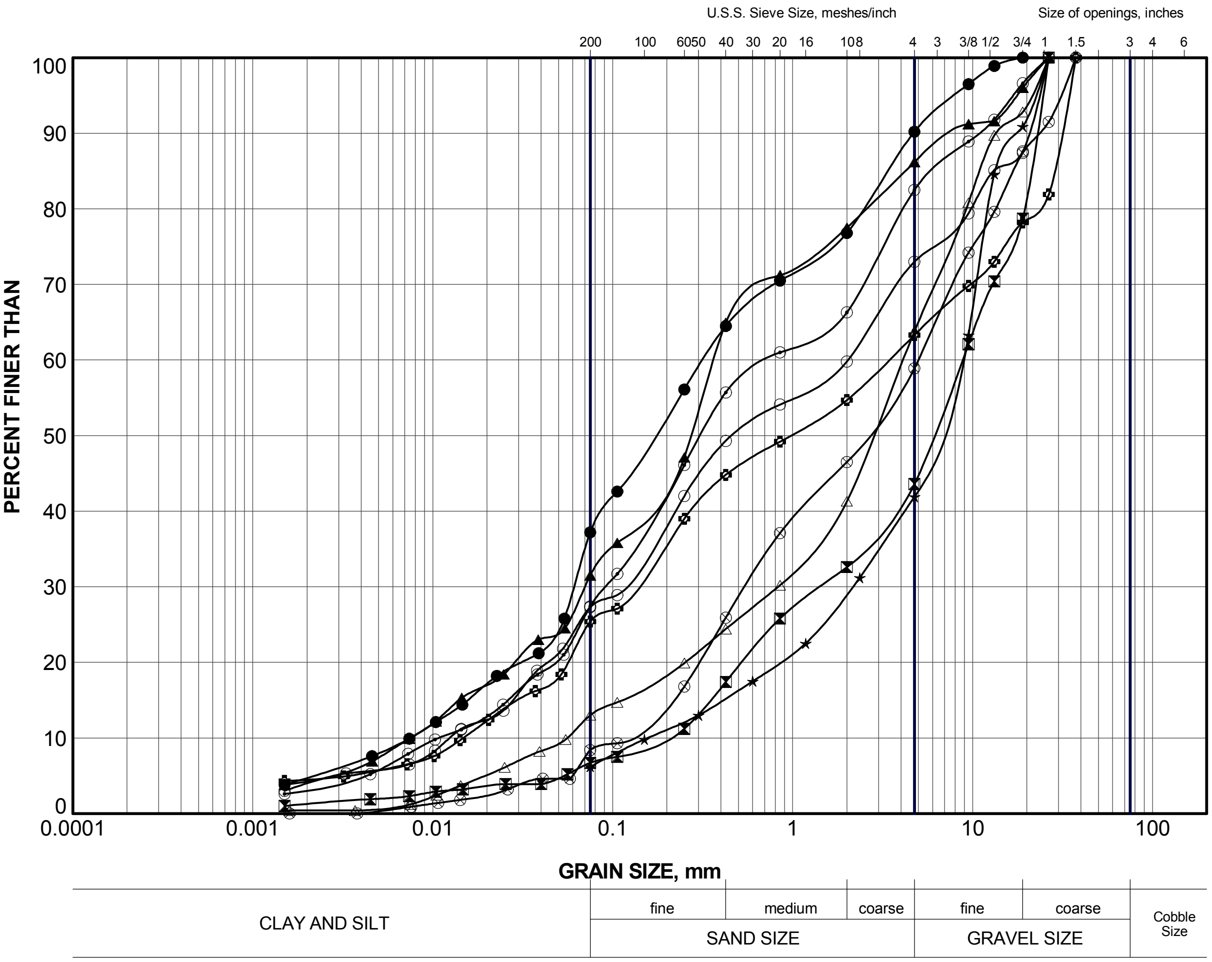
GRAIN SIZE DISTRIBUTION

SANDY CLAYEY SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1661607	FILE No.	1661607.GPJ
DRAWN	TB	Oct 2017	SCALE N/A	REV.
CHECK	AB	Oct 2017		
APPR		Oct 2017	**FIGURE B6**	

SUD-MTO GSD (2016) GLDR_LDN.GDT



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)

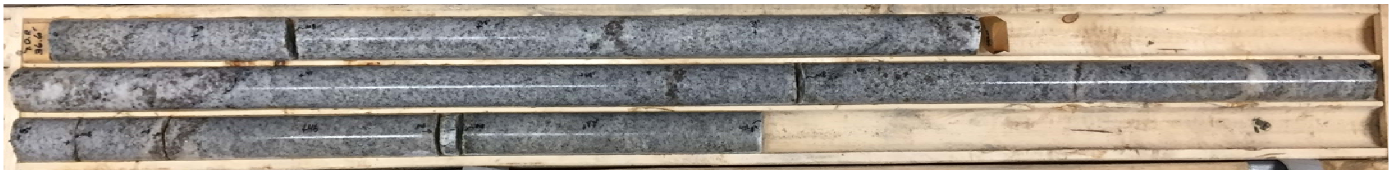
Golder Associates
SUDBURY, ONTARIO

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

PROJECT No.	1661607		FILE No.	----
DESIGN	TB	SEP 17	SCALE NTS	REV.
CADD	-- --			
CHECK	AC	SEP 17	**FIGURE B8**	
REVIEW				

Golder Associates

REVISION DATE: JuLy , 2017 BY: AB Project: 1661607

APPENDIX C

Non-Standard Special Provisions

OBSTRUCTIONS

Non-Standard Special Provision

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.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

H-PILES - Item No.

Non-Standard Special Provision

Amendment to OPSS 903

903.07.02.07 Monitoring Driven Piles

903.07.02.07.04 Wave Equation Analysis

Section 903.07.02.07.04 is deleted and replaced by the following:

The Contractor shall complete pile dynamic analyzer (PDA) testing on all piles that terminate on an inferred obstruction above the design pile tip elevation. The piles subjected to PDA testing shall be agreed by the Contractor and the Contract Administrator.

HIGH-STRAIN DYNAMIC TESTING, DEEP FOUNDATIONS – Item No.

Special Provision No. 903S06 October 2017

Amendment to OPSS 903, April 2016

903.02 REFERENCES

Section 903.02 of OPSS 903 is amended by the addition of the following under **ASTM International:**

D 4945-12 Standard Test Method for High-Strain Dynamic Testing of Deep Foundations

903.03 DEFINITIONS

Section 903.03 of OPSS 903 is amended by the addition of the following:

High Strain Dynamic Testing means a method of evaluating the quality of deep foundations and/or performance of the drive system. It is a form of load testing and involves the instrumenting and application of dynamic loads to a tested pile.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.02 Submission Requirements

Subsection 903.04.02 of OPSS 903 is amended by the addition of the following clause:

903.04.02.07 High-Strain Dynamic Testing

Prior to commencing high-strain dynamic testing, calibration certificates of all equipment used shall be submitted to the Contract Administrator. All equipment used shall be in good working condition, and shall have been calibrated within the last 2 years according to ASTM D 4945. Equipment set-up may be completed by trained Contractor personnel, however, testing shall be performed under the direction of an Engineer with at least 5 years of experience in high-strain dynamic testing and holding a proficiency rating at the Intermediate level or better for Dynamic Measurement and Analysis Proficiency Test as administered by the Pile Driving Contractors Association (PDCA). After December 31, 2020, the Engineer shall be required to hold a proficiency rating level of Advanced or better.

A preliminary report on the test results and its analysis shall be submitted to the Contract Administrator on the same day of the testing. The analysis shall be based on a closed-form solution (Case Method or approved equivalent) or signal-matching analyses (Case Pile Wave Analysis Program - CAPWAP or approved equivalent). As a minimum, the preliminary report shall include:

a) Pile ultimate resistance and integrity.

b) Calculated driving stresses.

c) Transferred energy and hammer efficiency at the time of the test.

A final report shall be submitted to the Contract Administrator within 10 Days of the field testing. The final report shall include the following:

a) Results of pile ultimate resistance and pile integrity based on signal-matching analyses (CAPWAP or approved equivalent), hammer performance and comparisons with any applicable static load test.

b) Discussion and recommendations for soil setup/relaxation, and/or revised pile installation criteria.

c) An appendix shall be included containing the following documents:
   i. Pile installation record
   ii. Reference subsurface information (borehole record)
   iii. Pile location drawing
   iv. Initial calibration check by the test computer unit
   v. Test set up geometry

The report shall be signed and sealed by two Engineers of the testing company, one of whom shall be identified as MTO's designated contact and one of whom shall have the required experience in high-strain dynamic testing and hold the required certificate of PDCA Proficiency Test.

903.07 CONSTRUCTION

903.07.02.07 Monitoring Driven Piles

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

Clause 903.07.02.07.03.01 of OPSS 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be validated using high-strain dynamic testing at end of drive (EOD). If the specified ultimate resistance is not achieved, retap/restrike should be conducted after sufficient time has passed to allow soil setup. The requirements for soil setup are as specified in the Contract Documents.

The results of the high-strain dynamic tests shall be submitted to the Contract Administrator who shall, in collaboration with the independent testing company, verify that the specified ultimate resistance has been achieved.

903.07.02.07.04 Wave Equation Analysis

Clause 903.07.02.07.04 is deleted in its entirety and replaced with the following:

903.07.02.07.04 Wave Equation Analysis and High-Strain Dynamic Testing

903.07.02.07.04.01 Wave Equation Analysis

Prior to mobilizing piling equipment to the site, a Wave Equation Analysis of Piles (WEAP) analysis shall be performed by the Contractor to demonstrate the potential for the proposed piling equipment to activate the specified ultimate resistance specified in the Contract Documents.

When requested by the Contract Administrator, all equipment, material, and personnel shall be supplied to conduct the wave equation analysis procedure.

903.07.02.07.04.02 High-Strain Dynamic Testing

An independent testing company with no corporate affiliation with the Contractor shall be employed to perform the high-strain dynamic testing. The independent testing company shall be RAQs qualified (Specialty: Geotechnical (Structures and Embankments – Medium or High Complexity)).

High-strain dynamic tests shall be performed by an Engineer employed by the independent testing company. The Engineer shall have documented evidence of training and experience in foundation engineering and wave equation analyses, and a certificate of proficiency (intermediate level or better) in the PDCA Dynamic Measurement and Analysis Proficiency Test.

High-strain dynamic testing shall be performed using the Pile Driving Analyzer, or approved equivalent, for the determination of pile ultimate resistance, establishment of pile installation criteria, assessment of pile integrity, monitoring of hammer/drive system performance and driving stresses, as specified in the Contract Documents. The method and equipment for testing and its reporting shall be according to ASTM D 4945.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing, and shall be submitted to the Contract Administrator for information purposes.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles; or as specified in the Contract Documents.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out during the retapping of piles, as specified in the Retapping Tests on Piles clause. Restrike testing shall be performed on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles; or as specified in the Contract Documents.

Restrike testing shall be carried out no sooner than 24 hours after installation of the individual pile and at a time specified in the Contract Documents. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

903.10 BASIS OF PAYMENT

Section 903.10 of OPSS 903 is amended by the addition of the following subsection:

903.10.04 High-Strain Dynamic Testing, Deep Foundations - Item

Payment for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

WARRANT: Always with this item.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Non-Standard Special Provision

Scope

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

MATERIAL

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801 and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract Drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs 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 CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP 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 sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Install piles by driving to the design tip elevation or bedrock if end-bearing piles are selected.
4. Place loose sand into the CSP.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

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

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

Criteria	**Tolerance**
Maximum deviation of CSP from pile centroid	**+/- 50 mm**
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	**+/- 10 mm**

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. 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 CSP.

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.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision

Amendment to OPSS 902, November 2010

902.01 SCOPE

Section OPSS 902.01 of OPSS 902 is amended by the addition of the following:

As part of the work under this item, the Contractor shall:

- Carry out any additional field investigation the Contractor deems necessary in order to engineer the dewatering systems;
- Design and install dewatering systems for each of the abutments to construct the substructures in the dry and to place and compact the granular backfill to the abutments in the dry;
- Carry out works necessary for the dewatering system which includes cofferdams, tremie concrete seal, and excavation for tremie concrete, etc.;
- Cut off the top of the dewatering systems 600 mm below grade or creek/river bed;
- Supply, place and compact granular backfill within the limits of the dewatering excavation beside and in front of abutment walls.

All work as shown on the Contract Drawings.

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517	Dewatering
OPSS 805	Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody 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

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

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 5 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

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

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 a distance of 500 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.

902.04.02.03 Milestone Inspections

The Quality Verification Engineer shall witness the following Interim Inspections of the work:

a) Dewatering of excavation for structure.

b) Completion of excavation for foundation.

c) Excavation for backfill and frost tapers.

d) Backfilling.

A copy of the written permission to proceed shall be submitted to the Contract Administrator prior to commencement of the successive operation.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

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 structure, 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.

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 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.

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