



September 10, 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

**REPLACEMENT OF NAGAGAMI RIVER BRIDGE - SITE NO. 39W-005
HIGHWAY 11, TOWNSHIP OF MCMILLAN, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5307-04-00**

Submitted to:
LEA Consulting Ltd.
625 Cochrane Drive, Suite 900
Markham, Ontario
L3R 9R9



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REPORT





Table of Contents

PART A –FOUNDATION INVESTIGATION REPORT

1.0	INTRODUCTION.....	1
2.0	SITE DESCRIPTION.....	1
3.0	INVESTIGATION PROCEDURES	2
4.0	SITE GEOLOGY AND SUBSURFACE CONDITIONS	5
4.1	Regional Geology	5
4.2	Subsurface Conditions.....	5
4.3	Nagamami River Bridge.....	5
4.4	Culvert Replacement – STA 20+585	10
4.5	High Fill – STA 20+650 to 20+775.....	12
4.6	Deep Cut – STA 21+050 to 21+150	14
4.7	Culvert Extension – STA 20+070.....	16
5.0	CLOSURE	17

PART B - FOUNDATION INVESTIGATION REPORT

6.0	DISCUSSION AND ENGINEERING RECOMMENDATIONS.....	19
6.1	General.....	19
6.1.1	Nagamami River Bridge	19
6.1.2	Culvert Replacement – STA 20+585.....	20
6.1.3	High Fill – STA 20+650 to 20+775	21
6.1.4	Deep Cut – STA 21+050 to 21+150.....	21
6.1.5	Culvert Extension – STA 20+070	22
6.2	Bridge Foundations.....	22
6.2.1	Deep Foundations – Steel H-Piles	23
6.2.1.1	Design Tip Elevation.....	24
6.2.1.2	Geotechnical Axial Resistance	25
6.2.1.3	Set Criteria and Pile Driving Note	25
6.2.1.4	Resistance to Lateral Loads	26
6.2.1.5	Frost Protection	28
6.2.2	Deep Foundations – Drilled Steel Casings	28
6.2.2.1	Design Tip Elevation.....	29



FOUNDATION REPORT, REPLACEMENT OF NAGAGAMI RIVER BRIDGE HIGHWAY 11, SITE NO. 39W-005, GWP 5307-04-00

6.2.2.2	Geotechnical Axial Resistance	30
6.2.2.3	Resistance to Lateral Loads	30
6.2.2.4	Frost Protection	32
6.2.3	Shallow Foundations.....	32
6.2.3.1	Geotechnical Resistance	32
6.2.3.2	Resistance to Lateral Loads	33
6.2.3.3	Frost Protection	33
6.2.4	Seismic Considerations	34
6.3	Embankment Stability	34
6.3.1	Results of Analysis.....	35
6.3.1.1	Nagagami River Bridge Approach Embankments	35
6.3.1.2	Culvert Replacement – STA 20+585	35
6.3.1.3	High Fill – STA 20+650 to 20+775.....	36
6.3.1.4	Deep Cut Slope – STA 21+050 to 21+150	36
6.4	Embankment and Culvert Settlement	36
6.4.1	Settlement of Embankment Fill	37
6.4.2	Settlement Performance Requirements	39
6.4.3	Results of Analysis.....	39
6.4.3.1	Nagagami River Bridge Approach Embankments	39
6.4.3.2	Culvert Replacement – STA 20+585	40
6.4.3.3	High Fill – STA 20+650 to 20+775.....	40
6.5	Culvert Foundations.....	40
6.5.1	Geotechnical Resistance	40
6.5.2	Resistance to Lateral Loads/Sliding Resistance	41
6.5.3	Horizontal Strain	41
6.5.3.1	Parameter Selection	41
6.5.3.2	Results of Analysis	42
6.6	Lateral Earth Pressures	42
6.7	Construction Considerations.....	43
6.7.1	Subgrade Preparation and Embankment/Cut Slope Construction	43
6.7.2	Control of Groundwater and Surface Water	45
6.7.3	Temporary Excavation Support Systems	45



6.7.4	Obstructions.....	47
6.7.5	Control of Potential Artesian Groundwater Pressure During Piling	47
6.7.6	Vibration Monitoring	47
6.7.7	Existing Structure Monitoring	47
6.7.8	Protection of Cut Slope Between STA 21+050 and 21+150	48
6.7.9	Analytical Testing for Construction Materials	48

7.0 CLOSURE	48
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REFERENCES

List of Symbols and Abbreviations

DRAWINGS

Drawing 1 Nagagami River Bridge – Index Plan

TABLES

Table 1 Evaluation of Foundation Alternatives

APPENDICES

Appendix A

Nagagami River Bridge

Drawing A1	Borehole Locations and Soil Strata
Drawing A2	Soil Strata
Drawing A3	Soil Strata
Record of Boreholes	N1 to N17, N1a and N8a
Record of Drillhole	N1a and N8a, N9 to N12
Table A1	Summary of Analytical Testing of River Water
Table A2	Summary of Rock Core Test Data
Figure A1	Grain Size Distribution – Sand to Sand and Gravel (Fill)
Figure A2	Grain Size Distribution – Sandy Silt to Silt and Sand to Clayey Silt (Fill)
Figure A3	Plasticity Chart – Sandy Silt to Clayey Silt (Fill)
Figure A4	Grain Size Distribution – Sand and Gravel (Upper Deposit)
Figure A5.1 and A5.2	Grain Size Distribution – Sandy Silt to Silt and Sand (Till), Upper Deposit
Figure A6	Plasticity Chart – Sandy Silt to Silt and Sand (Till) , Upper Deposit
Figure A7.1 and A7.2	Grain Size Distribution – Silt
Figure A8	Plasticity Chart – Silt, Some Clay
Figure A9	Grain Size Distribution – Sandy Silt to Silty Sand and Gravel (Till)
Figure A10	Plasticity Chart – Silt and Sand (Till)
Figure A11a and A11b	Rock Core Photographs
Figure A12	Stability Analysis – East Approach Front Slope

Appendix B

Culvert Replacement – STA 20+585

Drawing B1	Borehole Locations and Soil Strata
Record of Boreholes	NCU1 to NCU4
Figure B1	Grain Size Distribution – Gravelly Sand (Fill)
Figure B2	Plasticity Chart – Clayey Silt (Fill)



FOUNDATION REPORT, REPLACEMENT OF NAGAGAMI RIVER BRIDGE HIGHWAY 11, SITE NO. 39W-005, GWP 5307-04-00

Figure B3	Grain Size Distribution – Clayey Silt (Fill)
Figure B4	Grain Size Distribution – Sandy Silt to Silt and Sand (Till)
Figure B5	Plasticity Chart – Sandy Silt to Silt and Sand (Till)
Figure B6	Stability Analysis – STA 20+585

Appendix C

High Fill – STA 20+650 to 20+775

Drawing C1	Borehole Locations and Soil Strata
Drawing C2	Soil Strata
Record of Boreholes	NHF1 to NHF6
Record of Drillhole	NHF6
Figure C1	Grain Size Distribution – Silt and Sand to Sand and Gravel (Fill)
Figure C2	Grain Size Distribution – Sand and Gravel
Figure C3	Grain Size Distribution – Sandy Silt to Silt and Sand (Till)
Figure C4	Plasticity Chart – Sandy Silt to Silt and Sand (Till)
Figure C5	Grain Size Distribution – Silt to Clayey Silt
Figure C6	Plasticity Chart – Silt to Clayey Silt
Figure C7	Rock Core Photograph
Figure C8	Stability Analysis – STA 20+710

Appendix D

Deep Cut – STA 21+050 to 21+150

Drawing D1	Borehole Locations and Soil Strata
Drawing D2	Soil Strata
Record of Boreholes	NDC1 and NDC2
Figure D1	Grain Size Distribution – Clayey Silt to Silty Clay
Figure D2	Plasticity Chart – Clayey Silt to Silty Clay
Figure D3	Grain Size Distribution – Gravelly Silty Sand
Figure D4	Grain Size Distribution – Silt and Sand (Till)
Figure D5	Plasticity Chart – Silt and Sand (Till)
Figure D6	Stability Analysis – STA 21+073

Appendix E

Culvert Extension – STA 20+070

Drawing E1	Borehole Locations and Soil Strata
Record of Boreholes	1 and 2
Figure E1	Plasticity Chart – Clayey Silt
Figure E2	Grain Size Distribution – Clayey Silt
Figure E3	Grain Size Distribution – Sandy Silt to Silt and Sand (Till)
Figure E4	Plasticity Chart – Sandy Silt to Silt and Sand (Till)

Appendix F

Non-Standard Special Provisions

NSSP	CSP for Integral Abutment
NSSP	H-Piles
NSSP	Working Slab
NSSP	Groundwater Control
NSSP	Obstructions



PART A

**FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF NAGAGAMI RIVER BRIDGE - SITE NO. 39W-005
HIGHWAY 11, TOWNSHIP OF MCMILLAN, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5307-04-00**



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 Nagagami River Bridge (Site No. 39W-005), located on Highway 11 about 2 km west of Highway 631 (west of Hearst) in the Township of McMillan, Ontario. Foundation engineering services and also required for: the proposed culvert replacement and a section of high fill embankment for the highway realignment and culvert extension to the west of the bridge; and a section of a deep cut for the highway realignment to the east of the bridge.

The Terms of Reference and Scope of Work for the Foundation Investigation are outlined in MTO's Request for Proposal dated March 2011. Golder's proposal for foundation engineering services associated with replacement of the Nagagami River Bridge structure is contained in Section 6.8 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 August, 2011.

This report addresses the results of the detail level subsurface investigation carried out at the following sites as shown on Drawing 1, Index Plan:

- the proposed Nagagami River Bridge replacement, including the west and east abutments and pier, the associated west and east approach embankments and the roadway protection;
- the proposed Culvert replacement at about STA 20+585, approximately 320 m west of the bridge on the relocated alignment;
- the proposed High Fill embankment between about STA 20+650 and STA 20+775, approximately 200 m west of the bridge on the relocated alignment;
- the proposed Deep Cut between about STA 21+050 and STA 21+150 approximately 50 m east of the bridge on the relocated alignment; and
- the proposed Culvert extension at about STA 20+070, approximately 830 m west of the bridge.

2.0 SITE DESCRIPTION

The Nagagami River Bridge is situated in the Township of McMillan on Highway 11 approximately 2 km west of Highway 631 and approximately 60 km west of Hearst, Ontario. The surrounding land is generally flat but slopes down towards the riverbanks, and is of rural development with moderate tree-covered terrain beyond the highway right-of-way limits. The riverbanks adjacent to the existing bridge area are vegetated with grass and small shrubs. The river flows in a northerly direction and is about 60 m wide at the existing bridge location.

The existing structure consists of an approximately 12 m wide, 83 m long, five-span two-lane bridge constructed in 1958. The steel girder bridge structure is supported at the abutments and piers on shallow foundations consisting of spread footings, inferred from the original design drawings, founded on the native cohesionless soils.



The river water level shown on the General Arrangement (GA) drawing is Elevation 204.6 m (September, 2011). The water level of Nagagami River measured at the bridge site during the field investigation, which took place in July and August 2012, varied between Elevation 205.4 m and 205.1 m. The high water level is reported to be Elevation 206.4 m. The existing highway embankment grade is between about Elevation 213 m at the west approach and Elevation 216 m at the east approach, corresponding to about 8 m and 11 m above the water level measured during the field investigation, respectively.

3.0 INVESTIGATION PROCEDURES

The fieldwork for this subsurface investigation was carried out between July 26 and August 1, 2012, at the Preliminary Foundation Investigation phase of the project, and between July 18 and August 27, 2013, and March 21 and April 17, 2014, at the Detail Foundation Investigation phase, during which time thirty-three (33) boreholes (Boreholes N1 to N17, N1a, N8a, NCU1 to NCU4, NHF1 to NHF6, NDC1 and NDC2, 1 and 2) were advanced at the site. As Boreholes N2 and N4 were drilled to the south of the existing structure and are no longer pertinent to the new bridge alignment to the north of the existing structure, these two boreholes are not referred to any further. The investigation was carried out using the following drilling equipment:

- Boreholes N5, N6, N13 to N15, N17, NCU1 to NCU3 and NHF1 to NHF3 were advanced using a CME-55 track-mounted drill rig supplied and operated by Landcore Drilling Inc. of Sudbury, Ontario; and
- Boreholes N1, N1a, N3, N12, N16, NCU4, NDC1, NDC2, 1 and 2 located on land and Boreholes N7 to N11, N8a and NHF4 to NHF6 located in the river were advanced using a D-25 semi-portable drill rig, mounted on a truck, skid or a track machine where drilled on land and placed on a raft or on the ice where drilled in the river. The D-25 was supplied and operated by Walker Drilling Ltd. of Barrie, Ontario. Boreholes N2 and N4 were drilled to the south of the existing structure at the Preliminary Phase of the project and are not relevant to the Detail Phase of the project.

The borehole locations are shown on Drawings A1 to E1 in Appendices A to E.

The boreholes were advanced using 108 mm inner diameter hollow-stem augers and/or HQ/NW casing and wash boring. Where coring through boulders/bedrock was required, a NQ size core barrel was used. Soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter split-spoon sampler operated by an automatic hammer on the CME-55 drilling and a manual hammer (Cathead) on the D-25 drill rig, in accordance with 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 N17 at the bridge site to permit monitoring of the groundwater level. The piezometer consists of a 30 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 to the ground surface. The water level readings are indicated on the Record of Borehole sheets contained in Appendices A to E. The piezometer installation details are presented on the Record of Borehole sheet in Appendix A. The boreholes were backfilled with bentonite as per Ontario Regulation 903 (as amended) upon completion of drilling or the day after drilling and the piezometer was decommissioned in accordance with Ontario Regulation 903 (as amended) on August 27, 2013.



The fieldwork was supervised on a full-time basis by a member of Golder's staff, who located the boreholes in the field, cleared the site for buried 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. 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 select bedrock core samples.

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

A sample of the river water was obtained during the field investigation using appropriate sampling protocols and submitted to a specialist analytical laboratory under chain of custody procedures for testing for a suite of inorganic parameters.

The borehole locations and elevations were measured in the field by Golder personnel, relative to existing site features and surveyed to stakes placed in the field by JD Barnes Ltd., except for Boreholes NDC1 and NDC2, which were referenced to cross-section drawings provided by LEA as they are located along the proposed realigned roadway on relatively high ground about 8 m above the existing roadway embankment. The borehole locations (referenced to the MTM NAD83 co-ordinate system), ground surface elevations (referenced to Geodetic datum) and borehole depths are presented on the Record of Borehole sheets in Appendices A to D, and are summarized below.

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.



**FOUNDATION REPORT, REPLACEMENT OF NAGAGAMI RIVER BRIDGE
HIGHWAY 11, SITE NO. 39W-005, GWP 5307-04-00**

Location (Reference Appendix)	Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground/Water or Ice Surface Elevation (m)	Borehole Depth (m) (* includes water)
Bridge and Approach Embankments (Appendix A)	N1	5 515 137.5	266 116.3	211.9	23.2
	N1a	5515139.8	266 113.0	211.9	43.3
	N3	5 515 079.8	266 178.6	206.8	7.8
	N5	5 515 145.6	266 096.9	211.8	5.2
	N6	5 515 127.5	266 106.9	211.2	24.8
	N7	5 515 111.6	266 144.5	205.2	14.1*
	N8	5 515 108.3	266 149.6	205.2	9.9*
	N8a	5 515 107.5	266 150.3	204.8	14.7*
	N9	5 515 103.6	266 142.9	205.4	12.7*
	N10	5 515 100.1	266 136.7	205.1	11.0*
	N11	5 515 094.0	266 140.7	205.1	8.0*
	N12	5 515 068.9	266 170.4	209.3	11.6
	N13	5 515 061.1	266 189.1	213.2	8.2
	N14	5 515 138.8	266 091.0	212.1	9.6
	N15	5 515 129.6	266 101.5	211.3	15.8
	N16	5 515 068.8	266 166.0	208.1	8.1
	N17	5 515 055.8	266 184.5	213.9	14.3
Culvert Replacement STA 20+585 (Appendix B)	NCU1	5 515 358.2	265 872.6	206.4	14.3 (DCPT: 14.3 - 17.4)
	NCU2	5 515 348.0	265 856.2	206.2	15.8 (DCPT: 15.8 - 16.3)
	NCU3	5 515 334.1	265 850.9	211.8	18.9
	NCU4	5 515 329.1	265 859.9	211.9	15.8
High Fill STA 20+650 to 20+775 (Appendix C)	NHF1	5 515 281.3	265 915.5	211.2	18.6
	NHF2	5 515 255.0	265 945.6	211.0	17.4
	NHF3	5 515 228.7	265 975.7	211.0	13.0
	NHF4	5 515 301.3	265 932.6	204.0	12.8*
	NHF5	5 515 274.0	265 963.4	203.6	12.3*
	NHF6	5 515 247.9	265 992.7	203.2	5.9*
Deep Cut STA 21+050 to 21+150 (Appendix D)	NDC1	5 515 039.2	266 237.4	225.1	13.9
	NDC2	5 515 000.3	266 274.5	224.8	12.6
Culvert Extension STA 20+070 (Appendix E)	1	5 515 505.1	265 379.6	233.2	6.7
	2	5 515 513.4	265 379.2	230.7	4.8



4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on NOEGTS³ Mapping, the subsoils in the vicinity of the Nagagami River Bridge site generally consist of clayey till deposited as a ground moraine.

Published literature indicates that the site is located in the Quetico Subprovince of the Superior Province (OGS, 1991)⁴. The bedrock of this domain consists of muscovite-bearing granitic rocks (peraluminous), and may include biotite granite. Beyond the muscovite-bearing granitic boundary, bedrock consists of meta-sedimentary rocks.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced in the vicinity of Nagagami River Bridge, Culvert Replacement and Extension, High Fill and Deep Cut areas, together with the results of the laboratory tests carried out on selected soil samples, are presented on the Record of Borehole sheets and the laboratory test sheets in Appendices A to E. The results of the analytical testing on a sample of the river water are summarized in Table A1 in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets, and on the interpreted stratigraphic profile and cross-sections shown on the drawings presented in Appendices A to E are inferred from non-continuous sampling, observations of drilling progress and soil cuttings returns and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

Detailed descriptions of the subsurface conditions at each investigated area are provided in the subsequent sections of this report.

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

4.3 Nagamami River Bridge

The plan and profile along the Nagagami River Bridge centreline showing the borehole locations and interpreted stratigraphy, as well as cross-sections and profiles along the abutments and roadway protection centreline are shown on Drawings A1 and A2 in Appendix A. A total of fifteen boreholes were advanced at the bridge site:

- Boreholes N1, N1A, N5, N6, N14 and N15 were advanced at the west side of the bridge for the new abutment, approach embankment and roadway protection;
- Boreholes N7 to N11 and N8a were advanced for the centre pier; and

³ Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Map Reference Number 42GNW.

⁴ Ontario Geological Survey, 1991, Geology of Ontario.. ,Special Volume 4, Part 1. Eds P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott, Ministry of Northern Development and Mines, Ontario.



- Boreholes N3, N12, N13, N16 and N17 were advanced at the east side of the bridge for the new abutment, approach embankment and roadway protection.

Water/Ice

Boreholes N7 to N11 were advanced from the Nagagami River water surface, ranging from Elevation 205.4 m to Elevation 205.1 m and the water column was between 0.7 m and 1.3 m deep (measured between August 8 and 20, 2013). Borehole N8a was drilled over about 0.5 m of ice with surface at Elevation 204.8 m.

Fill

A 0.1 m to 0.7 m thick layer of peat/topsoil fill was encountered from ground surface in Boreholes N1, N3, N5, N6 and N14. Below the peat/topsoil fill in Boreholes N1, N3, N6 and N14 and from ground surface in Boreholes N12 to N17, a variety of fill material was encountered, consisting of sand and gravel, silty sand, sandy silt, silt and/or clayey silt, with peat pockets/seams in several samples as noted on the borehole logs recovered. The overall thickness of the fill deposit, including the surface layer of peat/topsoil, ranges from 0.6 m to 6.7 m with the ground surface encountered from Elevation 213.9 m to 206.8 m.

Two SPT 'N'-values measured within the peat/topsoil fill are 3 blows and 4 blows per 0.3 m of penetration, suggesting a soft to firm consistency. SPT 'N'-values measured within cohesionless fill deposit (sand and gravel, sand, silt) range from 0 blows (weight of hammer) to 78 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. In the cohesive soil fill strata (silt to clayey silt), the SPT 'N'-values range from 1 blow to 22 blows per 0.3 m of penetration, suggesting a soft to very stiff consistency.

The natural water content measured on one sample of the peat fill is about 32 per cent. The natural water content measured on samples of the sand and gravel, sand and silt fill strata range from 3 per cent to 18 per cent. The natural water content measured on samples of the silt to clayey silt fill strata range from 12 per cent to 37 per cent.

The grain size distribution of three samples of the sand to sand and gravel fill strata are presented on Figure A1 in Appendix A. The grain size distributions of four samples of the sandy silt to silt and sand clayey silt fill strata are presented on Figure A2.

Atterberg limits tests were carried out on four samples of the sandy silt to clayey silt fill strata and measured liquid limits ranging from about 17 per cent to 30 per cent, plastic limits ranging from about 14 per cent to 17 per cent and plasticity indices ranging from about 3 per cent to 16 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A3 and indicate the fill materials in places consist of sandy silt of slight plasticity to clayey silt of low plasticity.

Cobbles and Boulders

Cobbles and boulders were observed in the river bed in the vicinity of Boreholes N7 to N11. In Borehole N16, a 0.3 m size boulder was recovered from below the fill material at a depth of 1.2 m below ground surface (Elevation 206.9 m).



Sand and Gravel (Upper Deposit)

A 0.1 m to 0.7 m deposit of sand and gravel was encountered underlying the fill in Borehole N12 at a depth of 3.0 m below ground surface (Elevation 206.3 m), below the boulder in Borehole N16 at a depth of 1.5 m below ground surface (Elevation 206.6 m) and from the riverbed in Boreholes N8 to N11 (Elevation 204.4 m to 204.0 m).

The SPT 'N'-values measured within the sand and gravel deposit range from 15 blows to 107 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

The natural water content measured on a sample of the sand and gravel deposit is about 10 per cent.

The grain size distribution of one sample of the sand and gravel deposit is presented on Figure A4 in Appendix A.

Sandy Silt to Silt and Sand (Till)

A till deposit comprised of sandy silt to silt and sand was encountered below the fill below the upper deposit of sand and gravel and below the silt deposit (described in the next section) in Boreholes N1/N1a, N3, N5, N6 and N12 to N17, which were advanced on land, with the surface of the deposit encountered at a depth between 0.2 m and 6.7 m below ground surface, corresponding to between Elevations 211.2 m and 204.6 m. The thickness of the till deposit ranges from 6.5 m to 8.7 m where fully penetrated in Boreholes N1, N6, N12 and N15. In the remaining boreholes (i.e., N3, N5, N13, N14, N16 and N17), the deposit was not fully penetrated after exploring the deposit between 2.6 m and 7.6 m. Cobbles were encountered or inferred by auger grindings to be present within the till deposit as follows:

- Borehole N3 below a depth of 6.3 m (Elevation 200.5 m);
- Borehole N5, augers were grinding from 1.5 m to 2.1 m and at 3.0 m depth (Elevations 210.3 m to 209.7 m and 208.8 m);
- Borehole N6 at a depth of 2.6 m (Elevation 208.6 m); and
- Borehole N15 at a depth of 4.2 m (Elevation 207.1 m).

Difficult auger and casing advancement was noted throughout this deposit and coring techniques were required to advance some boreholes at various depths throughout this deposit.

The SPT 'N'-values measured within the till deposit range from 15 blows to greater than 100 blows per 0.3 m of penetration indicating a compact to very dense relative density, however the majority of SPT 'N'-values indicate very dense relative density.

The natural water content measured on samples of this deposit ranges from about 8 per cent to 19 per cent.

The grain size distributions of eighteen samples of the sandy silt to silt and sand till are presented on Figures A5.1 and A5.2.

Atterberg limits testing was carried out on fifteen samples of the till deposit and yielded liquid limits ranging from about 17 per cent to 24 per cent, plastic limits ranging from about 11 per cent to 17 per cent, and plasticity



indices ranging from about 4 per cent to 9 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A6, and indicate the material to be sand/silt of slight plasticity.

Silt

A silt deposit was encountered in most boreholes immediately below the water column, underlying and within the till deposit, or as a pocket underlying the fill deposit, as follows:

- In Borehole N1 and N6, a 10.7 m and 12.9 m thick deposit of silt was encountered at Elevations 201.7 m and 204.0 m, respectively. In Borehole N6, a cobble was encountered at a depth of 8.7 m below ground surface (Elevation 202.5 m);
- In Boreholes N7 to N11, a 3.2 m to 8.4 m thick deposit of silt was encountered from the riverbed or below the sand and gravel deposit between Elevation 204.1 m and 203.8 m. Below depths between 3.0 m and 5.3 m below ground surface, clay seams/layers were noted within the deposit in each of the boreholes;
- In Borehole N14, a 2.3 m thick deposit of silt was encountered interlayered within the silt and sand till deposit at a depth of 3.8 m below ground surface (Elevation 208.3 m);
- In Borehole N15, the silt deposit was encountered at Elevation 204.0 m and not fully penetrated after exploring the deposit for 8.5 m;
- In Borehole N16, a 0.4 m thick layer of silt was encountered below the upper deposit of sand and gravel at a depth of 2.2 m below ground surface (Elevation 205.9 m).
- In Boreholes N7, N8 and N9, clay seams/layers were noted within the silt samples obtained.

The SPT 'N'-values measured within the silt deposit range from 15 blows to greater than 100 blows per 0.3 m of penetration, indicating a compact to very dense relative density and generally dense to very dense.

The natural water content measured on samples of this deposit ranges from about 15 per cent to 26 per cent.

The grain size distribution of twenty-one samples of the silt deposit are presented on Figure A7.1 and A7.2.

Atterberg limits tests were carried out on nineteen samples of the silt deposit and the clay seams and measured liquid limits ranging from about 22 per cent to 37 per cent, plastic limits ranging from about 16 per cent to 22 per cent, and plasticity indices ranging from about 5 per cent to 19 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A8 in Appendix A, and indicate the material to be silt of slight plasticity and the seams to consist of clayey silt of low plasticity. One Atterberg limits test indicate a non-plastic result.

Silt and Sand to Silty Sand and Gravel (Till), Lower Deposit

A till deposit comprised of silt and sand to silty sand and gravel intermixed with cobbles and boulders in places was encountered below the silt or sandy silt deposits in Boreholes N1/N1a and N6, at depths of 20.9 m and 20.1 m below ground surface, respectively and in Boreholes N7, N8/N8a, N9, N10 and N11, at depths between 4.5 m and 9.0 m below the water surface. The surface of the deposit was encountered between Elevations 200.6 m



and 191.0 m and the thickness of the deposit is 19.3 m in Borehole N1a and ranges from 0.3 m to 2.6 m in Boreholes N8a, N9, N10 and N11 where fully penetrated and up to 5.4 m where not fully penetrated in Borehole N7.

Difficult casing advancement was noted throughout this deposit and coring techniques were required to advance the boreholes through the cobbles and/or boulders present at various depths/locations within this deposit.

The SPT was generally terminated before penetrating 0.3 m due to the hammer bouncing or measuring 100 blows or greater, indicating a very dense relative density.

The natural water content measured on samples of this deposit is between about 7 per cent and 10 per cent.

The grain size distribution of six samples of this deposit are presented on Figure A9.

Atterberg limits tests were carried out on four samples of this deposit and measured liquid limits ranging from about 16 per cent to 20 per cent, plastic limits ranging from about 10 per cent to 12 per cent, and plasticity indices ranging from about 5 per cent to 9 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A10, and indicate the material to be sand/silt of slight plasticity.

Bedrock/Refusal

In Borehole N1a, bedrock was encountered at a depth of 40.2 m below ground surface, corresponding to Elevation 171.7 m and the bedrock was cored for a length of 3.1 m.

In Boreholes N8a and N9 to N11, bedrock was encountered at depths ranging from 6.0 m to 11.6 m below water/ice surface, corresponding to between Elevations 199.1 m and 193.2 m and the bedrock was cored for lengths ranging from 2.0 m to 3.2 m.

In Borehole N12, bedrock was encountered at a depth of 10.4 m below ground surface (Elevation 198.9 m) and was cored for a length of 1.2 m.

The retrieved bedrock is described as fine to very coarse grained, fresh, grey and pink (where applicable) gneiss in Boreholes N1a, N8a and N9 to N11 and very coarse grained, slightly weathered, pink pegmatite in Borehole N12. Photographs of the retrieved bedrock cores are shown on Figure A11.

The Total Core Recovery of the bedrock core is 100 per cent. The RQD measured ranges from 81 per cent to 100 per cent, indicating a rock mass of good to excellent quality.

Laboratory UCS testing was carried out on four samples of the bedrock core. The UCS values are presented on the Record of Drillhole sheets in Appendix A and summarized below and indicate that the bedrock is considered medium strong ($R3, 25 \text{ MPa} < \text{UCS} < 50 \text{ MPa}$) to very strong ($R5, 100 \text{ MPa} < \text{UCS} < 250 \text{ MPa}$). The laboratory test sheet is presented as Table A2 in Appendix A.



Borehole/ Core Sample	Elevation (m)	UCS (MPa)
N8a/Run 1	193.0	70
N9/Run 1	195.4	102
N10/Run 2	196.0	76
N11/Run 2	198.0	110
N12/Run 1	198.3	41

Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling and a piezometer was installed in Borehole N17, sealed within the fill deposit, to monitor the groundwater level. The groundwater level in the open boreholes ranges from Elevation 210.0 m to 201.7 m measured at the following depths:

- In the boreholes advanced on land (Boreholes N1, N3, N5, N6 and N12 to N17), the depth to the groundwater level was measured between 0.3 m and 10.2 m below ground surface; and
- In the boreholes advanced in the river (Boreholes N7 to N11 and N8a), the groundwater level inside the drill casing was measured between 0.3 m below and 1.0 m above the river water surface.

In the piezometer at Borehole N17, the depth to the groundwater level was measure at 3.1 m below ground surface, corresponding to Elevation 210.8 m on August 27, 2013.

The river water surface was measured at between about Elevations 205.4 m and 205.1 m, between August 8 and 20, 2013. The ice surface was measured at about Elevation 204.8 m on March 21, 2014.

4.4 Culvert Replacement – STA 20+585

The plan and profile along the centreline of the culvert at STA 20+585 showing the borehole locations and interpreted stratigraphy are shown on Drawing B1 in Appendix B. The existing embankment at this location is approximately 6 m high and the existing timber box culvert is 1.8 m wide by 1.3 m high and 31 m long. A total of four boreholes (NCU1 to NCU4) were advanced at the culvert location.

Embankment Fill

In Borehole NCU3, a 300 mm thick layer of asphalt was encountered at the ground surface (Elevation 211.8 m). A 2.6 m to 2.7 m thick layer of gravelly sand to sand and gravel fill was encountered in Borehole NCU3 underlying the asphalt and in Borehole NCU4 from ground surface. Underlying the granular fill in both boreholes, a 2.0 m to 2.3 m thick layer of clayer silt fill was encountered.

The SPT 'N'-values measured within the gravelly sand fill range from 13 blows to 40 blows per 0.3 m of penetration, indicating a compact to dense relative density. SPT 'N'-values up to 75 blows per 0.3 m of penetration were recorded in Borehole NCU4 but are likely due to the frozen nature of the fill. The



SPT 'N'-values measured within the clayey silt fill range from 6 blows to 16 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency.

The natural water content measured on two samples of the gravelly sand fill is about 3 per cent and 7 per cent and of one sample of the clayey silt fill is about 18 per cent.

The grain size distributions of two samples of the gravelly sand fill are presented on Figure B1 in Appendix B.

An Atterberg limits test was carried out on a sample of the clayey silt portion of the fill deposit and measured a liquid limit of about 27 per cent, aplastic limit of about 17 per cent, and plasticity index of about 10 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B2 and indicate that the material is clayey silt of low plasticity.

The grain size distribution of one sample of the clayey silt portion of the fill is presented in Figure B3.

Peat

A 0.3 m and 0.6 m thick deposit of fibrous peat was encountered from ground surface in Boreholes NCU1 and NCU2, respectively and an approximately 0.6 m and 0.1 m thick layer of peat was encountered below the clayey silt fill in Boreholes NCU3 and NCU4. The surface of the peat was encountered between Elevation 206.8 m and 206.2 m.

One SPT 'N'-value measured within the peat deposit is 3 blows per 0.3 m of penetration, suggesting a soft consistency.

Sandy Silt to Silt and Sand (Till)

Underlying the peat in the four boreholes, a deposit of sandy silt to silt and sand till was encountered at depths ranging from 0.3 m to 5.6 m below ground corresponding to between Elevation 206.9 m and 205.6 m. All boreholes were terminated within this deposit, penetrating between 10.8 m and 15.2 m into the deposit.

The SPT 'N'-values measured within the sandy silt to silt and sand till deposit range between 4 blows and 92 blows per 0.3 m of penetration, indicating a loose to very dense relative density, but generally a compact to dense relative density.

The natural water content measured on ten samples of this deposit ranges from about 8 per cent to 11 per cent.

The grain size distribution of twelve samples of the sandy silt to silt and sand till deposit are shown on Figure B4.

Atterberg limits tests were carried out on twelve samples of this deposit and measured liquid limits ranging from about 15 per cent to 19 per cent, plastic limits ranging from about 12 per cent to 13 per cent, and plasticity indices ranging from about 3 per cent to 6 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B5 and indicate that the material is silt of slight plasticity.



Groundwater Conditions

Boreholes NCU1, NCU3 and NCU4 were dry upon completion of drilling. In Borehole NCU2, the depth to the the water level measured upon completion of drilling is 11.0 m below ground surface, corresponding to Elevation 195.2 m.

4.5 High Fill – STA 20+650 to 20+775

The plan and profile along the high fill embankment on the relocated alignment between STA 20+650 and STA 20+775 showing the borehole locations and the interpreted stratigraphy are shown on Drawing C1 in Appendix C. The existing embankment at this location is approximately 8 m high above the adjacent river water level. A total of six boreholes (NHF1 to NHF6) were advanced to investigate the subsurface conditions under the proposed high fill embankment.

Water

Boreholes NHF4 to NHF6 were advanced in the Nagagami River and encountered a water column between 0.8 m and 1.1 m deep. The surface of the water at the time of drilling ranges between Elevations 204.0 m and 203.2 m.

Embankment Fill

In Boreholes NHF1 to NHF3, a 400 mm thick layer of asphalt was encountered at the ground surface (Elevations 211.2 m to 211.0 m) underlain by 2.6 m to 5.2 m of embankment fill consisting of sand to gravelly sand to sand and gravel, silt and/or silt and sand.

The SPT 'N'-values measured within the embankment fill deposit range between 4 blows and 42 blows per 0.3 m of penetration, indicating a loose to dense relative density, but generally a compact relative density.

The natural water content measured on five samples of the fill deposit ranges from about 3 per cent to 10 per cent.

The grain size distributions of three samples of the embankment fill are presented on Figure C1.

Sand and Gravel

In Boreholes NHF3, an approximately 1.6 m thick deposit of sand and gravel was encountered below the embankment fill at a depth of 5.6 m below ground surface (Elevation 205.4 m). In Borehole NHF4 to NHF6, an approximately 0.1 m to 2.5 m thick deposit of sand and gravel was encountered from the river bed, with the surface of the deposit between Elevations 202.9 m and 202.4 m.

The SPT 'N'-values measured within this deposit range from 13 blows to 61 blows per 0.3 m of penetration, indicating a compact to very dense relative density. In Borehole NHF6, split spoon refusal (hammer bouncing)



was encountered at a depth at 1.2 m below the river water surface and NQ coring techniques were required to recover the remaining 0.4 m thick portion of the deposit.

The natural water content measured on one sample of the sand and gravel is about 10 per cent.

The grain size distribution of a sample of the sand and gravel deposit is presented on Figure C2.

Silt and Sand (Till)

A deposit of sandy silt to silt and sand till was encountered either below the embankment fill or below the sand and gravel deposit with the surface of the deposit at depths ranging from 3.0 m to 7.2 m below the ground surface in Boreholes NHF1 to NHF3 (Elevations 208.2 m to 203.8 m) and at depths of 1.2 m and 3.6 m below the water surface in Boreholes NHF4 and NHF5, respectively (Elevations 202.8 m and 200.0 m). The thickness of the deposit in Boreholes NHF2, NHF3 and NHF5 is between approximately 0.9 m and 4.2 m and Boreholes NHF1 and NHF4 were terminated within the till deposit, penetrating 15.6 m and 11.6 m into the deposit, respectively, upon auger refusal in Borehole NHF1,.

The SPT 'N'-values measured within this deposit range from 14 blows to 81 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

The natural water content measured on samples of the silt and sand till ranges from about 8 per cent and 11 per cent.

The grain size distributions of nine samples of the deposit are shown on Figure C3.

Atterberg limits tests were carried out on eight samples of this deposit and measured liquid limits ranging between about 16 per cent and 17 per cent, plastic limits ranging between about 12 per cent and 13 per cent and plasticity indices ranging from about 3 per cent to 5 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C4 in Appendix C, and indicate the material to be a silt of slight plasticity.

Silt to Clayey Silt

Underlying the silt and sand till deposit in Boreholes NHF2, NHF3, NHF5 and NHF6, a 1.4 m to 7.5 m thick deposit of silt to clayey silt was encountered at depths between 1.5 m and 8.7 m below ground surface, between Elevations 203.8 m and 199.1 m. The bottom of the deposit is inferred from auger refusal in Borehole NHF3.

The SPT 'N'-values measured within the silt to clayey silt deposit range from 26 blows to 62 blows per 0.3 m of penetration, suggesting a compact to very dense relative density/very stiff to hard consistency.

The natural water content measured of six samples of the silt to clayey silt ranges from about 15 per cent and 22 per cent.

The grain size distribution of six samples of the silt to clayey silt deposit is shown on Figure C5 in Appendix C.

Atterberg limits tests were carried out on six samples of the silt to clayey silt deposit and measured liquid limits ranging from about 19 per cent to 29 per cent, plastic limits ranging from about 6 per cent to 19 per cent and plasticity indices ranging from about 5 per cent to 15 per cent. The results of the Atterberg limits tests are shown



on the plasticity chart on Figure C6, and indicate the composition of the deposit ranges from silt of slight plasticity to clayey silt of low plasticity.

Bedrock

In Boreholes NHF5 and NHF6, bedrock was encountered at depths of 11.9 m and 2.9 m below the river water surface, corresponding to Elevations 191.7 m and 200.3 m, respectively, and the bedrock was cored for lengths of 0.3 m and 3.0 m, respectively.

In Borehole NHF5, the bedrock is described as gneiss, and in Borehole NHF6 the bedrock is described as very coarse grained, granitic pegmatite. A photograph of the retrieved bedrock core from Borehole NHF6 is shown on Figure C7.

The Total Core Recovery of the bedrock is 100 per cent. The Rock Quality Designation in Borehole NHF6 is 100 per cent, indicating a rock mass of excellent quality.

Groundwater Conditions

The depth to the groundwater level was measured in Borehole NHF3 at 9.8 m below ground surface (Elevation 201.2 m) upon completion of drilling.

The river water surface at Boreholes NHF4 to NHF6 was measured between Elevations 204.0 and 203.2 m between July 25 and 29, 2013.

4.6 Deep Cut – STA 21+050 to 21+150

The plan and profile along the deep cut of the highway realignment between STA 21+050 and 21+150 showing the borehole locations and the interpreted stratigraphy are shown on Drawing D1 in Appendix D. The depth of the cut is up to approximately 8 m relative to the proposed highway centreline. A total of two boreholes (NDC1 and NDC2) were advanced to investigate the subsurface conditions within the deep cut area through which the section of realigned highway will extend.

Topsoil

A 0.2 m and 0.3 m thick deposit of topsoil was encountered from ground surface in Boreholes NDC1 and NDC2, at approximately Elevations 225.1 m and 224.8 m, respectively.

Clayey Silt

A 2.1 m and 2.0 m thick deposit of clayey silt to silty clay was encountered underlying the topsoil at Elevations 224.8m and 224.6 m in Boreholes NDC1 and NDC2, respectively.



The SPT 'N'-values measured within the cohesive deposit range from about 3 blows to 8 blows per 0.3 m of penetration, suggesting a firm consistency.

The natural water content measured on two samples of the clayey silt is about 27 per cent and 28 per cent.

The grain size distributions of two samples of the clayey silt to silty clay deposit are shown on Figure D1 in Appendix D.

Atterberg limits tests were carried out on two samples of the cohesive deposit and the measured liquid limits are about 33 per cent and 36 per cent, the plastic limits are about 18 per cent and 19 per cent and the plasticity indices are about 14 per cent and 18 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure D2, and indicate the material tested to be clayey silt of low plasticity.

Gravelly Silty Sand

A 3.2 m and 0.8 m thick deposit of gravelly silty sand was encountered underlying the clayey silt to silty clay deposit in Boreholes NDC1 and NDC2, respectively, at a depth of 2.4 m and 2.2 m below ground surface corresponding to Elevations 222.7 m and 222.6 m.

The natural water content measured on a sample of the gravelly silty sand is about 13 per cent.

The SPT 'N'-values measured within the gravelly silty sand deposit range from about 12 blows to 59 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

The grain size distribution of one sample of the gravelly silty sand is shown on Figure D3.

Silt and Sand (Till)

Underlying the gravelly silty sand in Boreholes NDC1 and NDC2, a deposit of silt and sand till was encountered at a depth of 5.6 m and 3.0 m below ground surface, corresponding to Elevations 219.5 m and 221.8 m, respectively. Both boreholes were terminated within this deposit after penetrating 8.3 m and 9.6 m into the deposit.

The SPT 'N'-values measured within the silt and sand till deposit range from 73 blows per 0.3 m of penetration to 60 blows per 0.1 m of penetration, indicating a very dense relative density.

The natural water content measured on five samples of the sand and silt deposit is about 9 per cent.

The grain size distributions of five samples of the till deposit are shown on Figure D4 in Appendix D.

Atterberg limits tests were carried out on five samples of the till deposit and measured liquid limits ranging from about 16 per cent to 18 per cent, plastic limits ranging from about 12 per cent to 13 per cent and plasticity indices ranging from about 4 per cent to 6 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure D5 in Appendix D, and classify the fines material of the till deposit as silt of slight plasticity.



Groundwater Conditions

The water level in Borehole NDC1 and NDC2 upon completion of drilling was measured at is 1.4 m and 2.0 m below ground surface, corresponding to Elevations 223.7 m and 222.8 m.

4.7 Culvert Extension – STA 20+070

The plan and profile along the centreline of the culvert at STA 20+070 showing the borehole locations and interpreted stratigraphy are shown on Drawing E1 in Appendix E. The existing embankment at this location is approximately 2.5 m high and the existing timber box culvert is 1.3 m wide by 900 mm high and 20 m long. A total of two boreholes (Boreholes 1 and 2) were advanced at the culvert location.

Embankment Fill

In Borehole 1, a 300 mm thick layer of asphalt was encountered from the pavement surface (Elevation 233.2 m) underlain by a 2.3 m thick layer of sand and gravel to sand fill with a 75 mm thick layer of Styrofoam at a depth of 0.8 m. In Borehole 2, a 0.2 m thick layer of sand fill was encountered from ground surface at Elevation 230.7 m.

Two SPT 'N'-values measured within the fill are 23 blows and 24 blows per 0.3 m of penetration, indicating a compact relative density.

Clayey Silt

An approximately 1 m thick stratum of clayey silt was encountered in Borehole 1 at Elevation 230.8 m.

One SPT 'N'-value measured within the clayey silt is 7 blows per 0.3 m of penetration. An in situ field vane test carried out at the interface with the underlying silt and sand deposit measuring an undrained shear strength test of about 75 kPa, suggesting a stiff consistency. The sensitivity of the vane shear test is 40 suggesting a high influence of the less cohesive underlying deposit at the test depth.

The natural water content measured on a sample of the clayey silt stratum is about 22 per cent.

An Atterberg limits test was carried out on a sample of this deposit and measured a liquid limit of about 30 per cent, a plastic limit of about 17 per cent, and a plasticity index of about 13 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure E1 and indicate that the material is a clayey silt of low plasticity.

The grain size distribution of a sample of the clayey silt stratum is shown on Figure E2.



Sandy Silt to Silt and Sand (Till)

Underlying the clayey silt in Borehole 1 and the fill in Borehole 2, a deposit of silt and sand till was encountered at Elevation 229.8 m and 230.5 m, respectively. The boreholes were terminated after penetrating 3.3 m and 4.6 m into the deposit.

The SPT 'N'-values measured within the deposit range between 45 blows and greater than 100 blows per 0.3 m of penetration, indicating a dense to very dense relative density, with one near surface N-value of 6 blows per 0.3 m of penetration indicating a loose relative density.

The natural water content measured on samples of this deposit ranges from about 8 per cent to 12 per cent.

The grain size distributions of three samples of the deposit are shown on Figure E3.

Atterberg limits tests were carried out on three samples of this deposit and measured liquid limits ranging from about 17 per cent to 20 per cent, plastic limits ranging from about 12 per cent to 15 per cent, and plasticity indices of about 5 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure E4 and indicate that the material is silt of slight plasticity.

Groundwater Conditions

Boreholes 1 and 2 were dry upon completion of drilling.

5.0 CLOSURE

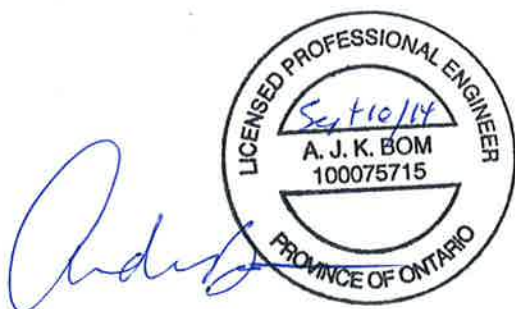
The field drilling program was supervised by Messers Ed Savard, Indulis Dumpis and Shane Albert. This report was prepared by Adam Core E.I.T., and the technical aspects were reviewed by Mr. André Bom, P.Eng. Mr. Jorge Costa, P.Eng., Principal and Golder's Designated MTO Foundations Contact for this project, conducted an independent review of this report.



Report Signature Page

GOLDER ASSOCIATES LTD.


Adam Core, E.I.T.
Geotechnical Engineering Intern



André Bom, P.Eng.
Geotechnical Engineer



Jorge M. A. Costa, P.Eng.
Designated MTO Contact, Principal

AB/JMAC/kp

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PART B

FOUNDATION DESIGN REPORT
REPLACEMENT OF NAGAGAMI RIVER BRIDGE – SITE NO. 39W-005
HIGHWAY 11, TOWNSHIP OF MCMILLAN, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5307-04-00



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the Nagagami River Bridge (Site No. 39W-005) located on Highway 11, west of Hearst, Ontario. Foundation design recommendations are also provided for the following structures/works:

- Proposed replacement of the existing Culvert located at about Station 20+585, approximately 300 m west of the existing bridge;
- Proposed High Fill embankment on the relocated northerly alignment between about Stations 20+650 and 20+775, approximately 200 m west of the existing bridge;
- Proposed Deep Cut on the relocated northerly alignment between about Stations 21+050 and 21+150, approximately 50 m east of the existing bridge; and
- Proposed northerly extension of the existing Culvert located at about STA 20+070, approximately 830 m west of the existing bridge.

The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at these sites.

The interpretation of the subsurface information and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed bridge structure foundations and approach embankments, and associated works as noted above. As such, where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on construction aspects should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

A summary of the existing conditions and proposed works are presented in the following sections.

6.1.1 Nagagami River Bridge

The existing five-span Nagagami River Bridge structure was constructed in 1958 and the abutments and piers are supported on shallow foundations as shown on the GA drawing for the existing bridge. We understand that based on the inspection/review of the existing bridge by others, the bridge is in poor condition structurally. Based on the soil profile shown on the GA drawing for the existing structure and the results of the current foundation investigation, the existing abutments are likely founded on native cohesionless soils.

We understand that the replacement bridge will be a two-span structure, on a new alignment located 18 m north of the existing centreline. The current GA drawing indicates that deep foundations are the preferred support system from a structural perspective. The finished grade for the realigned Highway 11 will be approximately Elevation 213.3 m at the west abutment and Elevation 215.3 m at the east abutment, requiring approach embankments up to about 1.4 m and 8.5 m high, respectively, relative to the existing natural ground surface on the north side of the proposed abutments.



The subsurface conditions encountered at the boreholes drilled at the proposed bridge alignment generally consist of the following:

- At the west abutment (Boreholes N1/1a and N6), a deposit of peat/topsoil and/or fill up to 2.2 m thick was encountered from ground surface, underlain by an about 6.5 m to 8.7 m thick deposit of dense to very dense sandy silt to silt and sand till. A silt interlayer in places and cobbles were encountered within the till. Underlying the till, a dense to very dense silt deposit between about 11 m and 13 m thick was encountered. In Borehole N1a, the silt deposit is underlain by a 19.3 m thick deposit of very dense silt and sand intermixed with cobbles in places, which was encountered to bedrock.
- At the east abutment (Boreholes N3, N12 and N16), an up to 6.7 m thick deposit of fill generally consisting of sand, silt and/or clayey silt was encountered from ground surface. Below the fill, a 5.5 m to 6.7 m thick deposit of sandy silt to silt and sand till was encountered. A relatively thin layer of sand and gravel and/or silt was encountered between the fill and till in places. The till deposit was penetrated at one borehole location and encountered pegmatite bedrock at a depth of 10.4 m below existing ground surface.
- At the pier (Boreholes N7 to N11), cobbles and boulders were observed on the river bed at the borehole locations and the depth of water ranged between 0.7 m and 1.3 m. A relatively thin layer of sand and gravel was encountered from the river bed, underlain by a deposit of compact to dense silt between about 3 m and 8 m thick, increasing in thickness from south to north. In addition:
 - Below the silt deposit at the northwest side of the pier, a 5.4 m thick deposit of sandy silt till was encountered, of which the approximately upper 1.8 m thick portion of the deposit contains gravel, cobbles and boulders up to 0.5 m sizes, but the deposit was not fully penetrated to the bottom of the borehole at a depth of 14.1 m below the water surface. At the northeast side of the pier, the silt deposit extends to a depth of 9.6 m below the water surface and is underlain by an up to about 2.6 m deposit of silt and sand till, with a 400 mm boulder, underlain by gneiss bedrock at a depth of 11.6 m below the water surface, at Elevation 193.2 m.
 - Below the silt deposit at the centre and south end of the pier, the boreholes encountered a deposit of till between about 0.3 m and 1.5 m thick, underlain by gneiss bedrock at depths ranging from 6.0 m to 9.5 m below the water surface, between Elevation 199.1 m and 195.9 m.

6.1.2 Culvert Replacement – STA 20+585

At the existing culvert located at STA 20+585, approximately 300 m west of the bridge, the new Highway 11 centreline will be relocated approximately 18 m to the north of the existing centreline. We understand that the existing timber box culvert will be replaced with a 1.8 m wide by 1.2 m high and 45 m long section of precast concrete box culvert, which is of similar dimensions to the existing culvert. A temporary roadway protection system will be required to allow for traffic staging during culvert replacement.

The finished grade of the realigned Highway 11 will be approximately Elevation 211.6 m, requiring an embankment up to about 6 m high, relative to the existing ground surface on the north end of the existing culvert.

The subsurface soils encountered at the boreholes drilled in the area of the proposed culvert replacement generally consist of:



- Peat from ground surface at the boreholes advanced beyond the toe of the existing embankment;
- Fill comprised of gravelly sand, underlain by a relatively thin layer of peat, at the boreholes advanced through the existing embankment;
- A deposit of compact to very dense sandy silt to silt and sand till underlying the peat, to the full extent of the boreholes, penetrating up to about 15 m into the deposit.

6.1.3 High Fill – STA 20+650 to 20+775

Between STA 20+650 and 20+775, approximately 200 m west of the bridge, the new Highway 11 centreline will be relocated approximately 24 m to the north of the existing centreline and the majority of the realigned embankment footprint will be located within the Nagagami River. The finished embankment grade in this area is Elevation 210.5 m requiring an embankment up to about 8 m high relative to the measured river bottom at the borehole locations.

The subsurface soils encountered along the high fill section (Boreholes NHF1 to NHF6) consist of:

- Within the river: An up to 3.6 m thick deposit of compact sand and gravel encountered from the riverbed, underlain a deposit of compact to very dense silt and sand till up to 11.6 m thick but not fully penetrated, or/in turn underlain by a deposit of very stiff to hard clayey silt to silt up to 7.4 m thick, decreasing in thickness easterly. Bedrock was encountered underlying the clayey silt to silt deposit at depths of 11.9 m and 2.9 m below the water surface.
- Existing embankment: A 3 m to 5.6 m thick deposit of fill comprised of up to 0.4 m of asphalt surface course and silt to silt and sand to sand to gravelly sand, underlain by a deposit of very dense silt and sand till to a depth of 8.6 m below the existing roadway surface near the western end of the section, underlain by a deposit of hard silt to clayey silt up to 10.2 m thick for the eastern portion of the section.

6.1.4 Deep Cut – STA 21+050 to 21+150

Between STA 21+050 and 21+150, approximately 50 m east of the bridge, the new Highway 11 centreline will be located approximately 18 m to the north of the existing centreline. The finished embankment final grade along the cut increases from Elevations 216.5 m to 221.0 m from west to east requiring cuts up to about 8 m deep at STA 21+050, up to about 9 m deep at STA 21+075 and decreasing to 3 m deep at STA 21+150.

The subsurface soils encountered at the boreholes advanced within the proposed deep cut area (Boreholes NDC1 and NDC2) generally consist of a thin deposit of topsoil underlain by a 2 m thick deposit of firm clayey silt to silty clay, underlain by a 1 m to 3 m thick deposit of compact to very dense gravelly silty sand, underlain by very dense silt and sand till. The till was not penetrated after exploring the deposit for up to 9.6 m.

The groundwater level in Boreholes NDC1 and NDC2 is about 3 m and 7 m above the proposed roadway surface of the realigned Highway 11 centreline at STA 21+075 and 21+125, respectively, at the time of investigation. The existing approximately 2 Horizontal to 1 Vertical (2H:1V) slope along the present roadway is covered with a layer of gravel/cobble sheeting material and we understand that a pond is located to the north of this area beyond the MTO right-of-way. We further understand that in the past water has emanated from the slope resulting in surficial sloughing of the slope.



6.1.5 Culvert Extension – STA 20+070

The new Highway 11 centreline will be relocated about 1 m to the north of the existing centreline at the culvert at STA 20+070. We understand that the existing timber culvert (1.3 m wide by 900 mm high and 20 m long) will be extended to the north and to the south with a 6 m and 3 m long section, respectively, with a precast concrete box culvert of similar dimensions to the existing culvert. The embankment at the culvert site is up to about 2.5 m high, relative to the existing ground surface on the north end of the existing culvert. The subsurface soils encountered at the borehole drilled through the embankment crest encountered fill, underlain by a thin layer of stiff clayey silt, underlain by silt and sand till to the bottom of the borehole. Near the north end of the proposed culvert extension, the borehole encountered a thin layer of fill underlain by silt and sand till to the bottom of the borehole.

6.2 Bridge Foundations

Based on the proposed bridge geometry and location relative to the existing structure as shown on the General Arrangement drawing, the relative proximity to and condition of the existing bridge, as well as the subsurface conditions at this site, steel H-pile foundations are recommended to support the abutments and drilled steel casing foundations to support the pier. A comparison of the alternative foundations options based on advantages, disadvantages, risks and relative costs, for the abutments and the pier is provided in Table 1 following the text of this report.

At the west abutment, the steel H-piles will have to be driven from the bottom of pre-augered holes to penetrate through the upper very dense silt and sand till deposit, depending on the elevation of the underside of the pile cap, eliminating the potential for the piles to hang-up and not achieving the minimum pile length for integral abutment design. At the east abutment, the H-piles would be driven into the very dense silt and sand till. At the pier, the H-piles, if selected as the foundation system, would require pre-drilling/socketing into bedrock in consideration of the sloping bedrock surface, and due to the shallow depth to bedrock on the south side of the pier. In order to construct the pile caps, excavations will be required into the very dense silt and sand till at the west abutment, into the very dense silt at the pier and to remove the fill adjacent to the river at the east abutment.

Drilled steel casings of 610 mm diameter socketted into bedrock are recommended for the pier foundations and would also be a suitable foundation system for the abutments as the casings would be able to penetrate through the very dense till and be founded in the till at the west abutment and socketted into bedrock at the east abutment. We understand that MTO Structural Section is considering the installation of three (3) 1.2 m or 1.5 m diameter caissons socketted into bedrock extending to the underside of the bridge deck at the pier as an alternative to 610 mm diameter drilled steel casings which would require a pile cap. In August 2014, Golder contacted deep foundation installation contractors and although advancing 1.2 m or 1.5 m diameter caissons into the bedrock would be technically feasible, it would be costly for this site due to the difficulty in seating/socketing the casings through the sloping bedrock surface, drilled into the very strong bedrock, and for equipment charges given the minimal number of caissons required. The cost for installing the three caissons is estimated to be \$0.8M to over \$1M by one contractor, excluding any additional costs associated with the high risk of construction of such a system, and possibly up to \$2.5M by another contractor. While the construction of such a foundation system may be technically feasible, given the disadvantages and high risk associated with socketing such a



large diameter casing into sloping bedrock, as presented in Table 1, this alternative is not considered a suitable option to be pursued at this site and was not considered further.

At the pier, a 1.2 m diameter casing installed to the bedrock surface with micropiles socketted into bedrock could also be considered at this site. For this foundation, the micropiles could potentially consist of a 0.27 m diameter casing and Number 18 Dywidag bars socketted up to about 3 m into bedrock to obtain a resistance of about 1,500 kN per pile. For this foundation alternative, a detailed micropile design would be required, as well as revisions to the existing micropile Non-Standard Specification (NSSP).

Shallow foundations have been also considered for support of the bridge at the abutments founded on the very dense silt and sand till deposit, however, at the east abutment, sub-excavation of the fill and into native material below the river water level is required to a larger footprint than that for pile cap construction. Sub-excavation of the existing fill for the construction of a granular pad to raise the footing elevation would also impact the footprint of the existing east abutment/approach embankment. The footing founding elevation at the west abutment should be as high as possible to minimize the depth of excavation and take advantage of the higher axial resistances provided by the very dense silt and sand till deposit. At the pier, the compact silt deposit does not provide suitable geotechnical axial resistance to support a shallow foundation and a different foundation system is required.

The following sections provide recommendations for steel H-pile and drilled steel casing foundations at the abutments and piers and for shallow foundations as an alternative at the abutments only.

6.2.1 Deep Foundations – Steel H-Piles

The abutments could be supported on steel HP310X110 piles which allow for an integral abutment design and the piers could also be supported on steel HP310X110 piles. However, due to the presence of cobbles and boulders within the till, which could cause the piles to “hang up” or be deflected from their intended vertical alignment, consideration should be given to using a heavier H-pile section, such as HP310x132, to reduce the potential for damage to the piles during driving to the required tip elevation.

At the abutments, the piles would be driven into the till deposit. At the pier, the piles should be installed in pre-drilled holes into the bedrock due to the steeply sloping bedrock surface across the pier.

At the east abutment, a granular pad could be incorporated into the embankment fill section through which the piles would be driven and upon which the pile cap would be constructed to facilitate abutment construction, as discussed in Section 6.7.1.

As discussed further in Sections 6.7.3 and 6.7.4, excavation into the very dense till deposit to the underside of the west abutment and the dense silt deposit for the pier cap will be difficult. Driving sheet piles for cofferdam construction will likely not be practical, or even possible, at this site due to the dense to very dense near surface soil deposits and presence of the cobbles and boulders, as well as the shallow depth to bedrock at the south side of the pier, to achieve a suitable depth of penetration to mitigate for base instability and minimize water inflow. An alternative cofferdam design, such as a temporary box, will be required to allow for excavation of the subsoils in the wet and a tremie concrete plug will be required to cut off water inflow for construction of the foundations in dry conditions (see Section 6.7.2).



Artesian conditions were encountered inside the casing upon completion of borehole drilling at several boreholes advanced at the pier (Boreholes N7, N8, N10 and N11). The unstabilized water level inside the casing in the four boreholes at the pier was measured between Elevations 206.2 m and 206.0 m, corresponding to between 0.9 m and 1.0 m above the River water level. In Borehole N9, the unstabilized water level inside the casing upon completion of drilling was at Elevation 205.1 m, corresponding to 0.3 m below the river level. A filter sand blanket (see Section 6.7.5) should be constructed immediately below the pile cap at the pier to dissipate potentially artesian groundwater and filter soil fines that may be carried upwards to the surface of the native soils in the event that the predrilled holes penetrate through an artesian deposit.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design, the CSPs should be backfilled with a loose, uniform, fine to medium sand. An NSSP detailing the CSP installation method and gradation of the sand should be included in the Contract Documents; an example is provided in Appendix F.

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

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

6.2.1.1 Design Tip Elevation

The piles should be advanced to the tip elevations as follows:

Foundation Element (Relevant Boreholes)	Proposed Underside of Pile Cap (m)	Estimated Tip Elevation (m)	Estimated Design Length (m)
West Abutment (N1 and N6)	206.8	189.0 (terminated 2 m into very dense silt and sand till deposit)	17.8
East Abutment (N3 and N12)	210.0	200.0 (terminated 2 m into very dense silt and sand till deposit)	10.0
North Half of Pier (N7 and N8a)	202.4 (tremie plug to 200.4)	192.7 m to Below 190.6 (pre-drilled at least 0.5 m into bedrock)	9.8 to 11.9
Centre (N9)		195.4 m (pre-drilled at least 0.5 m into bedrock)	7.1
South Half of Pier (N10 and N11)		198.6 m to 196.5 m (pre-drilled at least 0.5 m into bedrock)	3.9 to 6.0 *

Note: * Actual depth of pre-drilling at south side of pier will need to be determined by structural engineer based on required minimum pile lengths for foundation design.

There should be a provision made in the Contract for dealing with varying pile lengths due to the variability of the depth to bedrock and depth of penetration into the very dense subsoils. The lengths given above should be considered minimum lengths.



6.2.1.2 Geotechnical Axial Resistance

For steel H-piles end-bearing in the very dense foundation soils at the abutments, the geotechnical axial resistance at ULS is achieved by a combination of shaft resistance and toe resistance, and the factored ULS is estimated by applying a factor of 0.5 on the ultimate resistance in accordance with current MTO Foundations practice. The geotechnical reaction at SLS (for 25 mm of settlement) assumes that the pile will settle approximately 10 mm to 15 mm to mobilize shaft friction. The factored ULS and SLS values for two different pile types driven to the elevations given in Section 6.2.1.1 are as follows:

Foundation Element	Pile Section	Factored Geotechnical Axial Resistance at ULS (kN)	Geotechnical Axial Reaction at SLS (for 25 mm settlement) (kN)
West and East Abutments	HP310X110	1,600	1,100
	HP310X132	1,800	1,200
Pier *	HP310X110	2,000	N/A
	HP310X132	2,300	N/A

Note: * At the pier, the piles will be installed in pre-drilled holes (socketed) into bedrock and SLS does not apply.

We understand that at the pier, the structural designer may want to account for (include) the tensile resistance of the piles socketed into bedrock to reduce the thickness of the tremie plug below the pile cap. In this case, all of the piles would need to be socketed into bedrock a minimum of 1.1 m with the top 0.5 m of the less quality bedrock socket neglected in the tensile capacity of the pile. Based on a 610 mm diameter steel casing and a minimum 0.6 m long uncased socket below the top 0.5 m of the bedrock, a factored tensile axial resistance of 900 kN/m may be used for design (for concrete strength of 40 MPa).

6.2.1.3 Set Criteria and Pile Driving Note

All pile installation/driving operations should be in accordance with OPSS 903 (Deep Foundations). The piles at the abutments to be driven to a tip elevation within the very dense till deposit should be fitted with driving shoes or flange plates (reinforced tips) in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe) to minimize damage to the pile tip during driving. Given the presence of cobbles and boulders and potential for damage to the pile tip during driving, we recommend that consideration be given to using the heavier pile section (HP310X132) for the foundation at the abutments.

For end-bearing piles, the pile termination or set criteria will be dependent on the pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods including empirical correlations, such as the use of the Hiley Formula, and wave equation analyses, at the time of construction once the hammer and pile types are known. The criteria need to be set to allow for founding of the piles into the very dense till and to also avoid overdriving and possibly damaging the piles.

For end-bearing piles not founded on bedrock, the pile capacity must be verified in the field by the use of the Hiley Formula in accordance with Standard Drawing SS103-11 (MTO, 2008) "Pile Driving Control" during the



final stages of driving, starting at about 2 m to 3 m higher than the tip elevations provided in Section 6.2.1.1. The ultimate geotechnical axial resistance predicted from the Hiley Formula should then be multiplied by a geotechnical resistance factor equal to 0.5 as per current MTO practice to verify that the factored ULS design value has been achieved.

An NSSP, which outlines the above set criteria for end-bearing piles not founded on bedrock, should be included in the Contract; an example is included in Appendix F.

The pile driving note that should be added to the drawings at the abutments is Note 2 in Clause 3.3.3 of the Structural Manual (MTO, 2008). For HP310X110 (or HP310X132) piles, the note should read:

- Piles to be driven in accordance with Standard Drawing SS103-11 using an ultimate geotechnical resistance of 3,200 kN (or 3,600 kN) per pile but must be driven below EL 191 m at the west abutment and EL 202 m at the east abutment.

For piles to be installed into a bedrock socket at the pier, the piles should be tapped to confirm they are seated on the bedrock. For the pier foundation, the standard pile driving notes do not apply and the piles are to be installed/founded on bedrock at the bottom of the predrilled holes into bedrock.

6.2.1.4 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 Nagagami River Bridge structure. Where the integral design includes 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 F), the upper portion of the H-piles will generally 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. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

At the pier, the annulus between the predrilled holes and the piles should also be backfilled with uniformly graded, loose sand, as noted above.

The following equation (CFEM, 1992, as referenced in the CHBDC Commentary 2006) may be used to calculate values of the coefficient of horizontal subgrade reaction, k_h (in kPa/m), for non-cohesive soils. The value should be reduced by a factor of 0.75 to account for sloping ground surrounding the casing, where applicable.

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

where: n_h = constant of subgrade reaction (kPa/m)



**FOUNDATION REPORT, REPLACEMENT OF NAGAGAMI RIVER BRIDGE
HIGHWAY 11, SITE NO. 39W-005, GWP 5307-04-00**

z = depth (m)
 B = pile diameter or width (m)

The lateral resistance of the steel piles should be developed primarily from the passive resistance of the soil. The values of n_h (Terzaghi, 1955) to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the subsoils to be utilized for the structural lateral analysis of the piles at this site are summarized below.

Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)
West Abutment (N1 and N6)	CSP Liners (3 m)	u/s of pile cap to 203.8	1,300
	Silt, very dense Silt and Sand Till, very dense	203.8 to 189.0	11,000
Pier (N7 to N11)	Uniformly graded loose sand (in the predrilled hole)	Varies across pier, generally increasing in thickness from southeast to northwest, from the bottom of the tremie concrete plug (200.4 m) to the top of the bedrock at north side and to the top of the concrete within the bedrock at south side	1,300
East Abutment (N3 and N12)	CSP Liners (3 m)	u/s of pile cap to 207.0	2,200
	Compacted Granular Pad	207.0 to 204.3	18,000
	Silt and Sand (Till), very dense	204.3 to 198.9	11,000

For a single vertical HP310X110 (or HP310X132) pile advanced to the design tip elevations provided in Section 6.2.1.1, the estimated factored lateral resistance at ULS and the lateral reaction at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analysis carried out using the commercially available program LPile Plus (Version 5.0 for the abutments and 2013 for the pier), produced by Ensoft Inc.

Foundation Element	Lateral Resistance/Reaction (kN)	
	ULS (Factored)	SLS (10 mm of deflection)
West Abutment	90	40
Pier (north side; 1.5 m into bedrock)	See Note	115
East Abutment	90	40

Note: For the steel H-piles socketed into bedrock at the pier, the lateral resistance will be developed primarily from the fixity (in concrete) within the drilled sockets. In this case, the structural resistance of the steel H-pile will govern the ultimate lateral resistance.



The lateral resistances given above are based on an assumed fixed-head condition of 1,200 kN unfactored axial load applied at the top of the pile for HP310X110 and HP310X132 piles. The lateral resistance should be reviewed if greater vertical loads are anticipated.

6.2.1.5 Frost Protection

The pile caps 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.

6.2.2 Deep Foundations – Drilled Steel Casings

The abutments and pier could be supported on 610 mm diameter drilled steel casings installed by rotary percussive hammer and diamond drilling equipment, similar to that used to pre-drill into bedrock for the steel H-piles. The drilled steel casings have the following advantages over other deep foundation types:

- At the west abutment, the drilled steel casing can more readily be installed through/into the very dense silt and sand till deposit as well as penetrate through the cobbles and boulders that are present in the till deposit; and
- At the east abutment and pier, the drilled steel casings will be drilled/socketted into the bedrock to achieve the required capacity.

Appropriate drilling methods will be required during casing installation to balance the hydrostatic heads to minimize and control base disturbance due to hydrostatic pressures within the water-bearing granular deposits. Such methods may include the use of drilling mud or an adequate head of water to counter-balance the water pressure throughout all stages of the casing installation and during the placement of concrete by tremie methods.

The performance of the socket below the tip of the steel casing in compression will depend to a large degree on the condition of the subgrade soil or bedrock at the base of the drill hole/shaft. For steel casings acting in compression, the base of each casing/shaft must be cleaned to remove all loose cuttings to ensure that the tremie concrete is in intimate contact with the founding stratum or the foundation resistance will have to be developed through friction along the shaft. The inspection of the base of the shafts can be accomplished, after flushing and cleaning of the base by means of a Shaft Inspection Device (SID) such as a video camera, provided the water is not murky along the shaft and at the base of the hole. Should the camera inspection indicate that loosened/unacceptable soil is present at the base, the shaft/base would need to be re-cleaned and re-inspected. To account for the potential presence of sediment/loose material at the bottom of the shaft or socket (in the event that the hole cannot be fully cleaned) the design length of the shaft and socket recommended below has been increased such that the resistance of the drilled steel casing foundations do not rely on any contribution of base resistance. However, a contribution for base resistance has also been provided below for the condition that the bottom of the socket is properly flushed/cleaned of drill cuttings and sediment and the socket is confirmed by measurement to be clear to the bottom.



The drilling and construction of the casing foundations should be observed throughout by the Quality Verification Engineer to confirm that the conditions encountered are consistent with the information obtained from the boreholes and that the required tip elevation and base cleanliness has been achieved. The centre-to-centre spacing between the proposed casings, founded within the very dense foundation soils at the west abutment should be greater than 3 times the casing diameter to limit interaction between the casings. For casings socketed into the bedrock at the east abutment and the pier, the centre-to-centre spacing between the casings should be greater than 2.5 times the casing diameter to limit interaction between casings.

A granular pad incorporated into the embankment fill at the east abutment would facilitate the installation of the drilled steel casings, as discussed further in Section 6.7.1.

6.2.2.1 Design Tip Elevation

The drilled steel casings should be advanced to the tip elevations as provided below. At the west abutment the casing must be withdrawn/retracted as tremie concrete is placed to provide for a full length of concrete shaft in intimate contact with the surrounding soil to be able to achieve the required axial resistance. At the east abutment and pier, the drilled steel casing should be advanced a minimum depth of 0.5 m into good quality rock (RQD > 75%), the uncased socket should be 2.5 m long below the bottom of the casing and the casings will be left in the hole and form part of the shaft. The casings and uncased socket into bedrock, where applicable, should be advanced to the following elevations or lower:

Foundation Element (Relevant Boreholes)	Proposed Underside of Pile Cap (m)	Estimated Bottom of Casing Elevation	Estimated Design Length (m)
West Abutment (N1 and N6)	206.8	186.0 m – a minimum of 5 m into very dense till; casing to be removed.	17.8
East Abutment (N3 and N12)	210.0	198.0 m – bottom of rock socket at Elevation 195.5 m	14.5

North End of Pier (BH N7 and N8a)		Centre of Pier (BH N9)		South End of Pier (BH N10 and N11)	
Bottom of Casing Elevation (m)	Bottom of Uncased Socket Elevation (m)	Bottom of Casing Elevation (m)	Bottom of Uncased Socket Elevation (m)	Bottom of Casing Elevation (m)	Bottom of Uncased Socket Elevation (m)
Below 190.6 (West side) 192.7 m (East side)	Below 188.1 (West side) 190.2 m (East side)	195.4	192.9	196.2 *	193.7 *

Note * Relative to the lowest bedrock surface as encountered at Borehole N10.



There should be a provision made in the Contract for dealing with varying drilled steel casing lengths due to the variability in the thickness of the subsoils strata or surface elevation of the bedrock at the site. The lengths given above should be considered minimum lengths.

6.2.2.2 *Geotechnical Axial Resistance*

For a 610 mm diameter drilled steel casing founded at the elevations given in Section 6.2.2.1, the following factored geotechnical axial resistances at ULS may be used for design:

- West Abutment: 1,600 kN per unit for minimum 5 m penetration into very dense till; casing removed as tremie concrete is placed;
- East Abutment and Pier: 10,000 kN per unit comprised of 3,200 kN for shaft resistance for a minimum 2.5 m long uncased rock socket and 6,800 kN for base resistance. The geotechnical resistance is based on the full shaft resistance between the uncased bedrock side walls and the concrete, and a contribution for base resistance for the condition that the bottom of the socket is properly flushed/cleaned of drill cuttings and sediment and the socket is confirmed by measurement to be clear to the bottom.

The SLS reactions do not apply to the drilled steel casings advanced into the till deposit at the west abutment and socketed into the bedrock at the east abutment and pier, as the SLS reaction is greater than the factored geotechnical resistance at ULS.

We understand that at the pier, the structural designer may want to account for (include) the tensile resistance of the drilled steel casing socketed into bedrock to reduce the thickness of the tremie plug below the pile cap. In this case, assuming that the uncased bedrock socket extends a minimum of 2.5 m below the top 0.5 m length cased section (top 0.5 m length is neglected), a factored tensile axial resistance of 900 kN/m may be used for design based on a 610 mm diameter drilled steel casing and assuming a concrete strength of 40 MPa.

6.2.2.3 *Resistance to Lateral Loads*

The design of drilled steel casings subjected to lateral loads should take into account such factors as the batter of the casing (if any), the relative rigidity of the casing to the surrounding soil, the fixity condition at the head of the casing (pile cap level), the structural capacity of the casing to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the casing and casing group effects. For a longer, more flexible casing, the maximum yield moment of the casing 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 casings.

The resistance to lateral loading in front of a single casing may be calculated using subgrade reaction theory using the coefficient of horizontal subgrade reaction, k_h (kPa/m) (CFEM, 1992, as referenced in CHBDC, 2006). However, the response of a drilled steel casing to lateral loads is highly nonlinear and methods that assume linear behaviour (such as subgrade reaction theory) are only appropriate where the maximum casing deflections are less than 1 percent of the casing diameter, where the loading is static (not cyclical) and where the pile material is linear (CFEM, 2006). If one or more of these conditions are not satisfied, then it is recommended that the lateral pile analysis be carried out using p-y curves.



The following equation (CFEM, 1992, as referenced in the CHBDC Commentary 2006) may be used to calculate values of coefficient of horizontal subgrade reaction k_h (in kPa/m), for non-cohesive soils. The value should be reduced by a factor of 0.75 to account for sloping ground surrounding the casing, where applicable.

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

where:

n_h	=	constant of subgrade reaction (kPa/m)
z	=	depth (m)
B	=	casing diameter (m)

The lateral resistance of the drilled steel casings should be developed primarily from the passive resistance of the soil. The values of n_h (Terzaghi, 1955) to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the native subsoils to be utilized for the structural lateral analysis of the piles at this site are summarized in Section 6.2.1.4.

For a single vertical 610 mm diameter drilled steel casing advanced at the north side of pier, to the design tip elevations provided in Section 6.2.2.1, the estimated factored lateral resistance at ULS and the lateral reaction at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analysis carried out using the commercially available program LPILE Plus (Version 2013), produced by Ensoft Inc.

Foundation Element	Lateral Resistance/Reaction (kN)	
	ULS (Factored)	SLS (10 mm of deflection)
Pier (north side)	See note	250 kN

Note: For drilled steel casings socketed into bedrock, the lateral resistance will be developed primarily from the fixity (in concrete) within the drilled sockets. In this case, the structural resistance of the drilled steel casing will govern the ultimate lateral resistance.

The lateral resistances given above are based on an assumed fixed-head condition of 1,500 kN unfactored axial load applied at the top of the casing. The lateral resistance should be reviewed if greater vertical loads are anticipated.

Group action for lateral loading should also be considered when the casing spacing in the direction of the loading is less than eight (8) casing diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1986) in the direction of loading by a reduction factor, R, as follows:

Casing Spacing in Direction of Loading d = casing 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 casing spacings in between those listed above.



6.2.2.4 Frost Protection

The pile cap at the abutments if the pile cap is constructed in ground, should be provided with a minimum of 2.6 m of soil cover for frost protection as per OPSD 3090.100, or a combination of soil cover and rigid polystyrene foam insulation. Alternatively, the pile cap may be constructed at the underside of the bridge substructure.

6.2.3 Shallow Foundations

The new abutments could be supported on spread footings founded on the very dense silt and sand till. The recommended founding elevation for the footing at each foundation element is summarized below.

Foundation Element	Reference Borehole	Ground Surface (m)	Recommended Founding Elevation (m)
West Abutment	N1 and N6	211.9 to 211.2	206.8 ¹
East Abutment	N3 and N12	206.8 to 209.3	204.3 to 205.6 ² (following sub-excavation of existing fill)

Note: 1. Assumes 2.6 m of soil cover for protection from frost penetration, relative to the ground surface and embankment away from the abutment front slope, however, the founding elevation should be as high as possible to minimize the depth of excavation.
2. Shoring would be required to support the existing east approach embankment prior to sub-excavation of the existing fill, however, the presence of the cobbles and boulders at this location must be considered in the selection of the shoring system. If a granular pad is considered to support the east abutment, the footprint of the granular pad will encroach into the footprint of the existing east abutment/approach embankment.

As discussed further in Section 6.7.2, unwatering will be required at the east abutment to construct the footing(s) in-the-dry. Depending on the final founding elevation, unwatering is likely not required at the west abutment.

As discussed further in Sections 6.7.3 and 6.7.4, excavation into the very dense subsoils to the underside of the west abutment will be difficult. Driving sheet piles for cofferdam construction will likely not be practical, or even possible, at this site due to the presence of the cobbles and boulders, to achieve a suitable depth of penetration to mitigate for base instability and minimize water inflow. The founding elevation of the west abutment footing should be as high as possible to reduce the depth of excavation required and therefore consideration could be given to a combination of soil cover and rigid insulation for protection of the footing from frost penetration.

6.2.3.1 Geotechnical Resistance

Spread footings founded at the elevations given in Section 6.2.3, should be designed based on the factored geotechnical axial resistances at Ultimate Limit States (ULS) and geotechnical reactions at Serviceability Limit States (SLS) given below.



Founding Stratum	Footing Width (m)	Factored Geotechnical Axial Resistance at ULS (kPa)	Geotechnical Reaction at SLS (for 25 mm settlement) (kPa)
Very dense silt and sand till	2	700	300
	3	700	300

The ULS resistance 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 or founding elevation differs from those given above. In addition, the geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the Canadian Highway Bridge Design Code (CHBDC 2006) and its commentary.

All loose, softened or disturbed subgrade soil should be removed immediately prior to placement of concrete. Construction and inspection of footings should be carried out in accordance with OPSS 902 (Excavating and Backfilling – Structures).

The founding soils will be susceptible to disturbance from construction traffic and/or ponded water. To limit the effects of this disturbance, it is recommended that a concrete working slab be placed on the subgrade, as further discussed in Section 6.7.1.

Provided the spread footings are designed and constructed as recommended above, it is estimated that less than 25 mm differential settlement will occur between the spread footings founded on the very dense silt and sand till deposit at the abutments and the deep foundations at the pier.

6.2.3.2 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footing or concrete working slab and the compact to very dense native cohesionless soils should be calculated in accordance with Section 6.7.5 of the CHBDC. The following summarizes the coefficient of friction, $\tan \delta$, for the interface materials.

Interface Materials	Coefficient of Friction ($\tan \delta$)
Concrete footing or working slab on properly prepared very dense silt and sand till	0.55

This value is an unfactored value.

6.2.3.3 Frost Protection

The spread footings should be provided with a minimum of 2.6 m of soil cover for frost protection as per OPSS 3090.100 or a combination of soil cover and rigid insulation.



6.2.4 Seismic Considerations

Based on the information obtained from the NRCAN (2013) website for the site located at latitude 49.7721° N and longitude 84.5370° W, the peak horizontal acceleration (PHA) is equal to 0.029g at the bedrock level, for a probability of exceedance of 10 per cent in 50 years. According to Table 4.1 of the CHBDC, this site is located in Seismic Performance Zone 1 and the corresponding site-specific zonal acceleration ratio, A_s , is 0. Given this assessment, and in accordance with Section 4.4.5.1 of the CHBDC, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.3 Embankment Stability

Slope stability analyses were carried out for the following critical sections:

- Bridge east approach embankment front slope (Figure A12);
- Culvert Replacement embankment north slope adjacent to STA 20 + 585 (Figure B6);
- High Fill embankment north slope at STA 20+710 (Figure C8);
- Deep Cut slope at STA 21+075 (Figure D6); and
- Culvert Extension at STA 20+070 – based on an embankment height up to about 2.5 m in the immediate culvert location and the foundation soils consisting of stiff clayey silt and/or dense to very dense silt and sand, embankment stability is not a concern for the proposed culvert extension and embankment widening.

The analyses assume that fill, peat and topsoil, where encountered, will be removed from below the footprint of the new embankment sections and that the new embankments will be constructed of granular fill at the bridge approaches (alternatively rock fill could be used at the east approach), and of rock fill in the culvert replacement area, high fill section and culvert extension area.

The geometry of the proposed embankments, existing ground surface and existing river bed included in the analyses are based on the information provided by LEA. The piezometric conditions required in the stability analyses are based on the groundwater level as encountered during the subsurface investigation.

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 FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions at the end of construction. This FoS is considered adequate for the embankments at this site considering the design requirements and the field data available. 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 and rock fill, the existing fill and the cohesionless 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	Applicable Site	Bulk Unit Weight (kN/m ³)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
New Granular B Type I or Type II Fill (Engineered Fill)	Bridge East Approach (as an alternative to rock fill)	21	35°	-
Rock Fill	Bridge East Approach, Culvert and High Fill Areas	19	40°	-
Silt, Compact to Dense	Bridge only	19	33°	-
Sand and Gravel, Compact	High Fill only	21	35°	-
Silt to Clayey Silt, Hard	High Fill only	19	-	200
Clayey Silt to Silty Clay, Firm	Deep Cut only	18	-	40
Gravelly Silty Sand, Compact to Very Dense	Deep Cut only	19	33°	-
Silt and Sand Till, Compact to Very Dense	All 4 sites	21	35°	-

6.3.1 Results of Analysis

6.3.1.1 Nagagami River Bridge Approach Embankments

The stability analysis is focused at the east approach embankment front slope (i.e. towards the river) where the proposed embankment will be up to 11 m high relative to the river bottom. The analysis indicates that the new east approach embankment constructed of Granular B Type I or Type II material with the front slope graded to match the existing ground profile, but no steeper than 2H:1V, will have a FoS greater than 1.3 against global instability, as shown on Figure A12, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials will be carried out. Embankment side slopes inclined no steeper than 2H:1V will also have a FoS greater than 1.3 and are considered less critical than the front slope. The east approach front slope constructed of rock fill with a slope inclination of 1.25H:1V also has a FoS greater than 1.3.

6.3.1.2 Culvert Replacement – STA 20+585

The stability analysis of the embankment north slope in the immediate vicinity of the culvert replacement was carried out where the proposed embankment will be up to about 6 m high relative to the existing ground surface. The analysis indicates that the embankment constructed with rock fill with the slope graded to 1.25H:1V, placed and compacted on a properly prepared subgrade will have a FoS greater than 1.3 against global instability as shown on Figure B6. A granular fill embankment section at this location constructed with side slopes inclined at 2H:1V will also have a FoS greater than 1.3.



6.3.1.3 High Fill Embankment – STA 20+650 to 20+775

The stability analysis for the approximately 125 m long embankment section in this High Fill section was carried out for the embankment cross-section at approximately STA 20+710 where the proposed embankment will be up to about 8 m high relative to the existing river bottom at the north toe of the embankment. The analysis indicates that the north slope of the embankment constructed with rock fill material with the slope inclined at 1.25H:1V, will have a FoS greater than 1.3 against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials will be carried out, as shown on Figure C8.

6.3.1.4 Deep Cut Slope – STA 21+050 to 21+150

The stability analysis for the cut slope along the north side of the existing roadway was carried out for the cut at approximately STA 21+073 where the proposed cut will be about 9 m deep relative to the proposed realigned Highway 11 centreline, and indicates a global FoS greater than 1.3 as shown on Figure D6. The existing slope is inclined at about 2H:1V and is covered with granular sheeting (cobble size material) and appears to have no stability issues, however, we understand that sloughing was observed on the slope prior to the placement of the granular sheeting. Due to MTO property restrictions along the north limit of this area, the slope cross-sections provided by LEA indicate that a 2H:1V slope is proposed. We recommend that the new “cut” slope be excavated at 2H:1V final inclination and incorporate a 2 m wide bench mid-slope so that the height of uninterrupted slope does not exceed 6 m as adopted by MTO. Further, the slope should be covered with a 0.6 m thick layer of granular sheeting as further discussed in Section 6.7.8. If extending the property limits are not feasible, other alternatives to reduce the width of the cut slope are a concrete gravity retaining wall or RSS wall along the roadway cut thus reducing the height of the section of slope above the top of the wall.

6.4 Embankment and Culvert Settlement

Based on the relative density of the native cohesionless soils at the immediate bridge approach embankments, settlement of the foundation soils for the approach embankments are anticipated to be relatively minor (i.e., 10 mm or less) provided the existing fill and organic materials are removed prior to embankment construction.

At the west approach embankment, given the low embankment height and the minimal sub-excavation required at this location (i.e., total of about 1.6 m), we recommend that the new west approach embankment be constructed using granular fill (i.e., Granular ‘B’ Type I or Type II) up to the top of roadway subgrade level. Settlement of granular fill that is properly placed and compacted, is considered nominal and would occur during construction.

At the east approach embankment, sub-excavation of up to 3 m of existing fill will be required, followed by the construction of an up to about 10 m high rock fill embankment (relative to original ground surface), the height of which decreases away from the river as the ground surface elevation rises away from the river. If a granular pad is incorporated into the embankment fill below the east abutment, the remainder of the east approach embankment could also be constructed with granular fill. Alternatively, rock fill could be used to construct the east approach embankment and settlement of the rock fill itself is expected to occur. The methodology for calculating the magnitude of rock fill settlement is discussed in Section 6.4.1 and the result of settlement analysis for the east approach embankment is discussed in Section 6.4.3.1.



For the realigned Highway 11 embankment at the culvert replacement at STA 20+585 and the high fill embankment between STA 20+650 and 20+775, settlement of the foundation soils are expected to occur. Settlement of new granular embankment fill, that is properly placed and compacted, is considered nominal and would occur during construction. Where rock fill is used for embankment construction, settlement of the rock fill itself is expected to occur. The results of the settlement analysis for the culvert area and the high fill embankment are discussed in Sections 6.4.3.2 and 6.4.3.3, respectively.

To estimate the magnitude of the expected immediate settlements of the native cohesionless soils below the culvert at STA 20+585, analyses were carried out using the commercially available program Settle 3D (Version 2.0) produced by Rocscience Inc. Settlement analysis for native cohesionless soils below the high fill embankment between STA 20+650 and STA 20+775 were carried out using hand calculations. 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 employed for the different foundation soil types at the culvert location and high fill section are summarized below. Where the cohesionless foundation soil deposits noted is dense to very dense and where the consistency of the cohesive foundation soil deposit is very stiff to hard, settlement of these deposits is not expected to occur and these stratigraphic units and selected analytical parameters have not been included below.

Soil Type	Location (Representative Borehole)	Approximate Thickness (m)	γ (kN/m³)	E (MPa)
Sand and Gravel, compact	High Fill (NHF5)	2.5	21	20
Silt and Sand (Till), compact	Culvert (NCU2)	14.0	21	15

For the culvert extension at STA 20+070, minor settlement (i.e. less than 25 mm) is expected to occur within the foundation soils below the extended culvert and the minor embankment widening, given the relatively low embankment heights of about 2.5 m and that the foundation soils consist of stiff clayey silt and/or dense to very dense till.

6.4.1 Settlement of Embankment Fill

Where rock fill is used for the construction of the proposed embankments, there will be settlement due to compression of the rock fill itself under self-weight, in addition to the settlement of the underlying foundation soils. The magnitude of settlement of the rock fill depends on the following factors:

- type of rock/strength of particles;
- size and shape of rock particles;
- gradation of rock fill;
- total height/thickness of rock fill (stress level); and
- method of construction and sequence of placement (including lift thickness, compactive effort and state of packing).



The settlement of rock fill occurs as a result of re-arrangement of rock particles under load and wetting and as a result of localized crushing of rock particles at point contacts. The magnitude of both the short-term and long-term post-construction settlement of the rock fill is a function of the height of fill as well as the method of fill placement (i.e., compacted versus dumped rock fill) as outlined in “*MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates*” (MTO, September 2010).

Rock fill should be placed, whenever possible, in a controlled manner (i.e., not end-dumped) in accordance with OPSS.PROV 206 (Grading). Blading, dozing and ‘chinking’ the rock fill to form a dense, compact mass is required to minimize voids and bridging and, reduce settlements and should be used to construct rock fill embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e., below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.

Short-Term Rock Fill Settlement

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (September 2010), as follows:

Height of Rock Fill, H	Short-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 5 m	0.5% H	1.0% H
>5 m to 10 m	0.75% H	1.5% H
>10 m to 15 m	1.0% H	2.0% H

Approximately 90 per cent of the short-term settlement may be expected to occur within the first six (6) months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one (1) year following the completion of embankment construction to full height.

Long-Term Rock Fill Settlement

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (September 2010), as follows:

Total Height of Rock Fill, H	Long-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 15 m	0.1% H	0.2% H

The long-term rock fill settlement is expected to occur from one (1) year following the completion of construction over the life of the embankment.



6.4.2 Settlement Performance Requirements

The settlement performance criterion for design of high fill embankments is 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
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.

In general, new embankments not approaching a structural element are to be designed as follows:

Type	Maximum Limits During Pavement Design Life	
	Total Settlement	Differential Settlement Rate
Non-Freeways (Highway 11)	200 mm	100:1

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 the embankment in the vicinity of the culvert replacement and the high fill area.

6.4.3 Results of Analysis

6.4.3.1 Nagagami River Bridge Approach Embankments

At the east approach embankment, rock fill is not to be used for backfill to the abutments, but could be used beyond the abutment backfill zone. Assuming a 10 m high rock fill embankment is required beyond the abutment after the sub-excavation of the existing fill, approximately 85 mm of post-construction settlement will occur of which 65 mm is expected to occur within the first 6 months after construction. Since the total post construction settlement is greater than 25 mm within 20 m of the bridge east abutment, a six-month preload period will be required to allow for the elimination of the short term rock fill settlement., or the embankment will have to be constructed of granular fill.



6.4.3.2 Culvert Replacement – STA 20+585

The settlement of the foundation soils at the centerline of the proposed Highway 11 realignment near the midpoint of the new concrete culvert is estimated to be about 110 mm and will occur after culvert/embankment construction. The settlement at the north end of the new culvert (i.e., outlet) will be about 20 mm and will also occur after culvert/embankment construction. Settlement is not anticipated at the south (i.e. inlet) end of the culvert based on the current grading design.

The structural designer should verify that the proposed concrete culvert can tolerate the above noted magnitude of settlement and provide any required camber to accommodate any tolerable settlement applicable to both structural and hydraulic designs.

If rock fill is placed around/above the culvert for embankment construction, approximately 60 mm of post-construction settlement of the rock fill will occur, of which 50 mm is expected to occur within the first 6 months after construction.

Since the total post construction settlement is estimated to be less than the 200 mm settlement criterion for non-freeway roads, settlement mitigation measures are not required at the culvert site, provided the existing/proposed culverts and connection can tolerate the estimated magnitude of settlement.

6.4.3.3 High Fill Embankment – STA 20+650 to 20+775

Due to the compact relative density of the surficial 2.5 m thick deposit of sand and gravel encountered at Borehole NHF5 at about STA 20+710, the total settlement of the foundation soils at this location is estimated to be 20 mm and will occur during embankment construction.

Based on the relative density of the cohesionless soils and consistency of the cohesive foundation strata below the proposed realigned Highway 11 centreline at Boreholes NHF4 and NHF6 at about STA 20+670 and 20+750, the immediate settlement will be negligible.

If rock fill is used to construct the 8 m high embankment, approximately 70 mm of post-construction settlement will occur of which 60 mm is expected to occur within the first 6 months after embankment construction.

Since the total post construction settlement is estimated to be less than the 200 mm settlement criterion for non-freeway roads, settlement mitigation measures are not required.

6.5 Culvert Foundations

The following foundation design recommendations are provided for a box culvert replacement at STA 20+585. Due to the relatively small width of the culvert (i.e., 1.8 m), an open footing culvert is not practical.

6.5.1 Geotechnical Resistance

For the box culvert replacement at STA 20+585, it is recommended that any organic materials encountered below the culvert footprint be sub-excavated and replaced with Granular 'B' Type II material. The factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS (for 25 mm settlement) for a 1.8 m



wide box culvert founded on a properly prepared subgrade/granular bedding overlying the native compact silt and sand till may be taken as 350 kPa and 100 kPa, respectively.

The loading on the foundation soils below the culvert and the associated total settlement at the culvert location will be governed by the design height of the overlying and adjacent embankment fill. As such, it is recommended that the structural engineer exercise caution when utilizing the values of the geotechnical axial resistance at SLS in the design of the culverts and that consideration be given to the sequence and staging of construction.

The geotechnical resistances provided are for loads applied perpendicular to the base of the culvert. Where loads are not applied perpendicular to the base of the culvert, inclination of the loads should be taken into account in accordance with Section 6.7.4 and Section C6.7.4 of the Canadian Highway Bridge Design Code (CHBDC, 2006) and its Commentary.

6.5.2 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of a box culvert and the granular fill/bedding placed following sub-excavation of organic deposits or existing fill and levelling of the subgrade surface, should be calculated in accordance with Section 6.7.5 of the CHBDC. The following summarizes the coefficient of friction for the interface materials.

Interface Materials	Coefficient of Friction
Precast Concrete Box Culvert on Compacted Granular 'A' or Granular 'B' Type II Bedding	$\tan \delta = 0.45$
Cast-in-Place Concrete Box Culvert on Compacted Granular 'A' or Granular 'B' Type II Bedding	$\tan \delta = 0.55$

These values represent unfactored values.

6.5.3 Horizontal Strain

The following sections outline the methods used to estimate the horizontal strain along the culvert and the results of the analysis.

6.5.3.1 Parameter Selection

As a result of the two-dimensional nature of the proposed embankment geometry, shear stresses will be mobilized in the foundation soils upon completion of embankment construction causing lateral spreading of the foundation soils and new embankment fill. This, in conjunction with the non-uniform vertical settlement of the foundation soils along the proposed culvert alignment, will generate horizontal straining along newly constructed culvert. In order to maintain the structural integrity of the culvert, the design must incorporate a suitable allowance for extension at the joints/couplings of the culvert segments, as well as the connection/joint between the new concrete culvert and the existing timber culvert, to prevent the new culvert from cracking and/or failing in tension and the new joints opening excessively.



The research work by Rutledge and Gould (1973) on the movements on articulated conduits under earth dams on compressible foundations can be used to estimate the magnitude of the horizontal strain likely to occur as a result of the proposed embankment construction at culvert sites. The following equations have been used to obtain a relationship between vertical settlement, vertical strain, horizontal strain and maximum joint opening as a result of settlement of the foundation soils:

$$\varepsilon_v = \frac{\delta_v}{d}$$

$$\varepsilon_h = \varepsilon_v \frac{\varepsilon_h}{\varepsilon_v}$$

$$\Delta L = \varepsilon_h L$$

where :

ΔL = maximum joint opening (m)

ε_v = maximum vertical strain

ε_h = maximum horizontal strain

$\frac{\varepsilon_h}{\varepsilon_v}$ = estimated ratio of maximum horizontal strain to maximum vertical strain from Figure 2 in Rutledge and Gould (1973)

L = length of culvert (m)

δ_v = maximum vertical settlement of culvert as a result of immediate and post-construction settlement of foundation soils and granular fill/bedding material (m)

d = thickness of compressible foundation deposits at culvert location (m)

6.5.3.2 Results of Analysis

As discussed in Section 6.4.3.2, the settlement analysis indicates that the total post-construction settlement of the foundation soils along the new culvert at STA 20+585 will be between about 20 mm at the north end of the new culvert and 110 mm near the culvert midpoint. Therefore, the maximum post-construction horizontal strain along the north half of the new 45 m long culvert is estimated to be about 0.35 per cent of the culvert length (or about 70 mm).

6.6 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls, as well as culvert 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.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II, but with less than 5 per cent passing the No. 200 sieve, 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 501 and SP 105S21 (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) or OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), as applicable.

- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northern Region Directive (2002) for backfill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill, Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. 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). The pressures are based on the proposed embankment fill material and 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

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. If the wall support and superstructure does not allow lateral yielding, at-rest earth pressures should be assumed for foundation design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHBDC.

6.7 Construction Considerations

6.7.1 Subgrade Preparation and Embankment/Cut Slope Construction

For the new Highway 11 realigned embankment sections, removal of the fill and organic soils from below the footprint of the embankment is recommended prior to construction of the new embankments, including up to about 3.0 m at the east abutment (Borehole N12) and about 5.6 m at the east approach (Borehole N13 located 20 m east of the east abutment) of the Nagagami River Bridge. Sub-excavation of the fill from within a shored area or a temporary box (see Section 6.7.3) will be required within the footprint of the new east abutment, to avoid impacting the existing east abutment/embankment. Also, all softened/loosened soils should be stripped from below the approach embankment footprint, prior to placement of new fill.



Fill for construction of the new embankment sections should consist of a Granular 'B' Type I or Type II meeting the specifications of OPSS.PROV 1010 (Aggregates). Alternatively, the embankment sections at the east approach, at the culvert at STA 20+585, and the high fill embankment between STA 20+650 and 20+775 may be constructed of rock fill. The embankment fill for the realigned Highway 11 should be placed and compacted in accordance with OPSS 501 (Compacting) and OPSS.PROV 206 (Grading). Rock fill should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging should be minimized by blading, dozing and 'chinking' the rock to form a dense, compacted mass. Where new fill is to tie into existing fill along and beyond the approaches and the realigned highway, the new fill should be "keyed-in" or benched into the existing fills, in accordance with OPSD 208.010 (Benching of Earth Slopes). Side slopes for rock fill embankments should be no steeper than 1.25H:1V. Side slopes for granular fill should be no steeper than 2H:1V.

At the east abutment, if a compacted granular pad is incorporated into the embankment through which the piles or drilled steel casings would be installed, the pad should be constructed such that it extends to the base of the fill sub-excavation and is integrated into the east approach embankment (rock or granular fill). We recommend the pad be constructed using Granular 'B' Type II material with maximum particle size of 75 mm. The granular pad should extend at least 1 m beyond the plan limits of the pile cap and be sloped no steeper than 1H:1V. The granular pad should be constructed in accordance with OPSS.PROV 206. Following sub-excavation of the fill, the granular pad will be up to about 6 m thick at the east abutment based on the underside of the abutment at Elevation 210.0 m, though sloping ground and the variable depth of subexcavation required could locally require the pad to be thicker. The granular pad should be constructed concurrently with embankment construction to reduce the potential for differential settlement occurring and for overall simplicity of construction.

The precast box culvert at STA 20+585 should be constructed in accordance with OPSS 422 and SP 422S01 (Precast Reinforced Concrete Box Culvert). The box culvert should be constructed on a minimum 300 mm thick layer of SP 110S13 (Aggregates) Granular 'A' or Granular 'B' Type II material for bedding purposes. All granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 per cent of the standard Proctor maximum dry density.

Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

For embankments constructed using rock fill, the incorporation of 2 m wide benches (or successive berms) into the uniform side slope profile is required wherever the embankment will exceed a height of 10 m such that the uninterrupted rock fill slope does not exceed a height of 10 m as per OPSD 202.010 (Slope Flattening). For embankments constructed using granular fill, 2 m wide benches (or successive berms) are required for embankment heights of 8 m or greater. For the east approach embankment and the east abutment front slope, the geometry of the embankment should incorporate a 2 m wide branch.

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 modifications 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 scour below the pile caps in the design of the bridge foundations.



To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments (unless rock fill is used). 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) to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

The cohesionless soils that will be exposed within the excavation at the abutments and pier (if applicable) will be susceptible to disturbance from construction traffic and/or ponded water. To limit the effects of this disturbance, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP; an example of which is included in Appendix F. If piles are installed, it is anticipated that the piles would be driven through the working slab in this case.

6.7.2 Control of Groundwater and Surface Water

The pile cap or spread footing subgrade elevation at the abutments should be set as high as possible to avoid the need for unwatering for construction of the footings in-the-dry. However, depending on the final founding elevation and the water level at the west abutment at the time of construction, unwatering may be required and could be carried out using a temporary box that is lowered as the excavation progresses deeper, followed by a tremie concrete plug placed at the bottom of the excavation within the temporary box.

At the pier, the excavation for the pile cap should extend a minimum of 2.6 m below the riverbed to provide for protection from frost penetration and will extend into the water bearing silt deposit. Therefore, appropriate unwatering of this deposit within a temporary cofferdam will be required to maintain the water level below the founding level during excavation and for construction of the pile cap in-the-dry. It is expected that a temporary box with adequate lateral support, that is lowered as the excavation progresses deeper in-the-wet, followed by a tremie plug at the base of the excavation, will be required. Unwatering would then be carried out from within the cofferdam once the required excavation depth is achieved and the tremie concrete plug has set.

An NSSP should be included in the Contract to alert the contractor to the potential issues associated with unwatering of the soils at the abutments and pier and that the excavation must be unwatered and kept stable during footing or pile cap construction; an example NSSP is included in Appendix F.

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.

6.7.3 Temporary Excavation Support Systems

At the abutments, temporary shoring will be required to remove existing fill below the new abutments (specifically at the east abutment), construct the footings or pile cap, as applicable, and to allow for construction of the new approach embankments adjacent to the existing highway embankment, which will need to remain in operation during construction of the new bridge. However, due to the dense to very dense relative density of the non-



cohesive deposits at this site and the presence of cobbles and boulders, driving of sheet piles is likely not practical. The temporary support system could consist of soldier piles and lagging (temporary roadway protection) where the H-piles would be driven or installed in pre-drilled holes to a suitable depth, followed by horizontal lagging installed as the excavation proceeds. If soldier piles and lagging is selected, pile installation should be in accordance with OPSS 903 (Deep Foundations). Support to the cofferdam could be in the form of struts and walers; bracing is likely not required for the temporary roadway protection, depending on the unsupported height of the excavation required behind the cofferdam.

Temporary excavation support systems should be designed and constructed in accordance with OPSS 539 (Temporary Protection Systems). The lateral movement of the temporary shoring systems should meet Performance Level 2 as specified in OPSS 539. The contractor is responsible for the complete detailed design of the temporary shoring/protection systems.

The design of braced soldier pile and lagging walls should be based on a rectangular earth pressure distribution using the design parameters given below.

For a braced excavation in granular fill and native non-cohesive soils, the unfactored rectangular earth pressure distribution (P in kN/m^2 ; constant with depth), can be calculated as follows (CFEM 2006):

$$\begin{array}{llll}
 P & = & K_a (0.65 \gamma H + q) \\
 \text{where } K_a & = & \text{active coefficient of earth pressure} \\
 H & = & \text{the total depth of the excavation (m)} \\
 \gamma & = & \text{soil unit weight (kN/m}^3\text{)} \\
 q & = & \text{surcharge for traffic and other loadings (kN/m}^2\text{)}
 \end{array}$$

The support systems may be designed using the following parameters:

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction (ϕ , degrees)	Unit Weight (γ , kN/m^3)
	Active, K_a	At Rest, K_o	Passive, K_p		
Existing sandy, silty fill	0.36	0.53	2.8	28	18
Existing clayey fill (east abutment/approach only)	0.37	0.55	2.7	27	17
New Granular 'B' Type I or II Fill	0.27	0.43	3.7	35	21
Silt, compact to dense	0.29	0.46	3.4	33	19
Silt and Sand (Till), compact (culvert only)	0.29	0.46	3.4	33	19
Silt and Sand (Till), very dense	0.27	0.43	3.7	35	21

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficient of earth pressure should be adjusted accordingly.



6.7.4 Obstructions

Cobbles and boulders were observed on the surface of the river bed. Further, the fill at the site was noted to contain cobbles/boulders and the native soils at this site are glacially derived and as such are very dense and contain coarse gravel, cobbles and boulders as noted in the Record of Borehole sheets, which could affect the installation of deep foundations, excavations for foundations and installation of cofferdams/temporary roadway protection systems. An NSSP should be included in the Contract Documents to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils; an example of which is included in Appendix F.

6.7.5 Control of Potential Artesian Groundwater Pressure During Piling

If steel H-piles are driven at the pier, given that groundwater levels were measured in the casing above the river water level during the subsurface investigation in Boreholes N7, N8, N10 and N11, we recommend that a drainage/filter blanket consisting of a 0.5 m thick layer of concrete fine aggregate (OPSS.PROV 1002, Aggregates – Concrete) be placed below the underside of the pile cap. The concrete fine aggregate layer should extend a minimum of 0.5 m horizontally beyond each of the pile caps or be contained within the cofferdam.

6.7.6 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 Nagagami River 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 pile driving operations and for the piles driven closest to the existing bridge. Based on vibration monitoring experience, it is considered unlikely that the vibrations induced by conventional construction activities (such as pile driving) will affect the performance of the existing structures, 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.

6.7.7 Existing Structure Monitoring

We recommend that the abutments and piers of the existing structure be monitored for settlement and lateral movement during the new construction, especially during construction works adjacent to the existing structure, such as excavation operations, installation of temporary shoring or roadway protection and installation of deep foundations for the following reasons:

- the existing bridge is likely supported on spread footings founded on the native soil strata;
- the age and poor condition of the existing structure; and
- the requirement for the existing structure to carry traffic during construction of the new structure.

The foundation monitoring should be carried out by a qualified foundations consultant reporting to the Contract Administrator.



6.7.8 Protection of Cut Slope Between STA 21+050 and 21+150

As discussed in Section 6.3.1.4, for the 9 m deep cut in the proximity of STA 21+075, decreasing westerly to about 8 m deep at STA 21+050 and easterly to 5 m deep at STA 21+150, the cut slope should be excavated to an inclination of 2H:1V and should include a 2 m wide bench for the extent of the cut where the slope height/depth is greater than 6 m, as adopted by MTO.

Once overall drainage is provided to the cut slope, long term drainage and slope protection will be required to maintain stability and to minimize surficial sloughing. A granular blanket/sheeting should be provided on the cut slope as per OPSS 511 (Rip-Rap, Rock Protection and Granular Sheeting). The granular sheeting material should be as per OPSS.PROV.1004 (Aggregates – Miscellaneous). The granular sheeting should be connected to toe drains/interceptor ditches that are adequately sloped to drain away from the cut area. The granular sheeting should be a minimum of 400 mm thick. We recommend that a non-woven geotextile (i.e., Terrafix 270R or equivalent) be placed on the native slope prior to the granular sheeting being placed to prevent the migration of fines into the gravel sheeting and emanating onto the face of the slope and into the drainage ditch.

6.7.9 Analytical Testing for Construction Materials

The analytical test results on a sample of river water are presented in Table A1. The suite of parameters tested is intended to allow the structural engineer to assess the requirements for the appropriate type of cement to be used in construction and the need for corrosion protection.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Adam Core, E.I.T. and Mr. André Bom, P.Eng. Mr. Jorge Costa, P.Eng., Golder's Designated MTO Foundations Contact for this project and a Principal with Golder, carried out a technical and independent quality control review of this report.



Report Signature Page

GOLDER ASSOCIATES LTD.

Adam Core, E.I.T.
Geotechnical Engineering Intern



André Bom, P.Eng.
Geotechnical Engineer



Jorge M. A. Costa, P.Eng.
Designated MTO Contact, Principal

AC/AB/JMAC/kp

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Structural Manual. Provincial Highways Management Division, Highway Standards Branch, Bridge Office, April, 2008.

Pile Driving Control, Standard Drawing SS103-11, April 2008

Ministry of Transportation Ontario Special Provisions

SP 422S01 Precast Concrete Box Culvert

Ontario Provincial Standard Drawings

OPSD 202.010	Slope Flatening
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3000.201	Foundation, Piles, Steel H-Pile 310 Oslo Point
OPSD 3001.100	Foundation, Piles, Steel Tube Pile Driving Shoe
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3101.200	Walls Abutment, Backfill, Rock
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement

Ontario Provincial Standard Specifications

OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
OPSS 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 206	Construction Specifications for Grading.
OPSS.PROV 1002	Material Specification for Aggregates – Concrete
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS 1359	Material Specifications for Unshrinkable Fill

Ontario Water Resources Act

Ontario Regulation 903/90 Wells: O. Reg. 468/10 Amendment to Ontario Regulation 903



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
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
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 = σ'_p / σ'_{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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



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; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	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	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 With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

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° 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	

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

CONT No.
GWP No. 5307-04-00

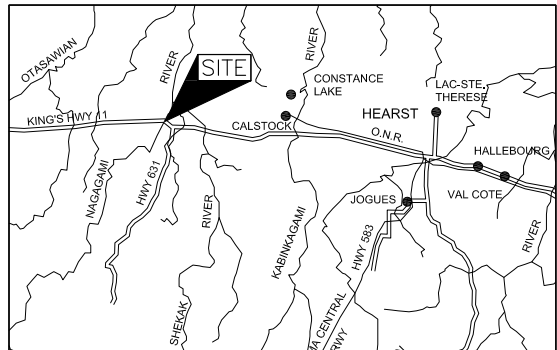


HIGHWAY 11
NAGAGAMI RIVER BRIDGE
INDEX PLAN

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN

SCALE

20 0 20 40 km

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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by LEA, drawing file nos. 8960-Nag-S01.dwg received on May 28, 2013 and x8960 Nagagami Base.dwg, received Sept 25, 2013.

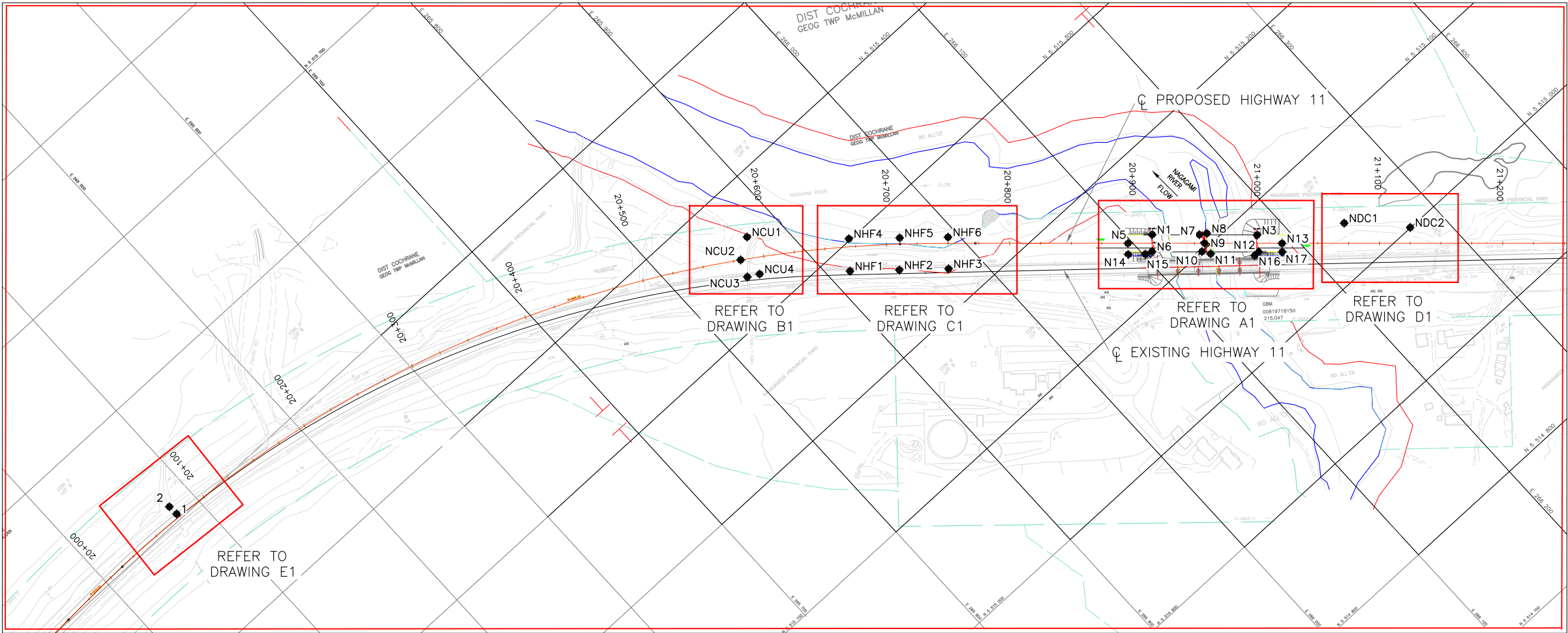




Table 1: Evaluation of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven Steel H-piles (Driven into till at abutments and installed in pre-drilled holes into bedrock at pier)	1 (Abut) 2 (Pier)	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Higher axial resistance compared to spread footings. ■ Allows for integral abutment design. ■ Similar foundation support systems at abutments and pier. 	<ul style="list-style-type: none"> ■ Potential for “hanging up” on cobbles and boulders within cohesionless deposits or piles terminating within the dense stratum at shallow depth, but likely easier to advance than pipe piles. ■ Requires socketing into bedrock at pier due to sloping bedrock and to achieve required minimum pile length at south side of pier. ■ Requires shoring system to excavate through the fill (below groundwater level) adjacent to existing east abutment for pile cap construction. ■ Requires cofferdam and unwatering (after tremie plug construction) at pier to allow for pile cap construction in dry conditions. 	<ul style="list-style-type: none"> ■ Relative costs higher than shallow foundations. ■ Mobilization of specialized drilling equipment relatively expensive. ■ Lower relative cost than drilled steel casing at abutments. ■ Unwatering system at pier can be expensive. 	<ul style="list-style-type: none"> ■ Potential for not achieving design resistance at design pile tip elevation and therefore have to drive piles to a lower tip elevation. ■ Much reduced, or elimination of, potential for discharge of sediments into the river. ■ At pier, risk of difficulties predrilling into sloping and strong to very strong bedrock which could raise costs and potentially affect schedule. ■ Difficulties in constructing unwatering system at pier and potential for claims.
Drilled Steel Casings (610 mm diameter) socketted into bedrock using DTH drilling at pier and east abutment and founded at least 5 m in the till at the west abutment	1 (Pier) 2 (Abut)	<ul style="list-style-type: none"> ■ Highly suited to drilling/excavating through the very dense subsoils at west abutment. ■ Higher axial resistance compared to spread footings at abutments and steel H-piles at pier ■ Reduced vibrations on existing bridge compared with pile driving ■ Allows for similar construction at abutments and pier. ■ Better suited for installation on sloping bedrock surface as is the case at the pier. 	<ul style="list-style-type: none"> ■ Requires specialized drilling equipment. ■ Likely not suitable for integral abutment design as lateral resistance on casings will be too great due to the dense/very dense relative density of the soil strata. ■ Would require smaller diameter casings to provide the desired lateral resistances but MTO does not normally accept drilled casing/pipe piles for integral abutment design. ■ Would require more onerous management of cuttings/drilling fluid to prevent discharge of these materials into the river. 	<ul style="list-style-type: none"> ■ Mobilization of specialized equipment relatively expensive; more efficient from cost perspective to install at both the abutments and the pier. ■ Higher cost than steel H-piles due to requirement for casings to remain 	<ul style="list-style-type: none"> ■ Potential impact on river water quality due to cuttings/drilling fluid release. ■ Requires off-site disposal area for disposal of drilling fluid and cutting. ■ At pier and east abutment, risk of difficulties drilling rock sockets on sloping and strong to very strong bedrock which could raise costs and



Table 1: Evaluation of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
		<ul style="list-style-type: none"> At pier, higher lateral capacity than H-piles. 	<ul style="list-style-type: none"> Casing is a permanent liner where drilled into bedrock such as at the pier and therefore must have an allowance for greater wall thickness due to potential corrosion at waterline. Requires cofferdam and unwatering (after tremie plug construction) at pier to allow for pile cap construction in dry conditions. 	<ul style="list-style-type: none"> in place as permanent liners at pier and east abutment. Higher cost due to need to dispose of drilling fluid/cuttings off site. Unwatering system at pier can be expensive. 	<ul style="list-style-type: none"> potentially affect schedule. Difficulties in constructing unwatering system at pier and potential for claims.
Micropiles at pier (0.273 m diameter) socketted into bedrock using DTH drilling; arranged in sets of 3 or 4; installed within 1.2 m diameter outer steel casing	3	<ul style="list-style-type: none"> Relatively straightforward construction. Pile cap/shoring not required as micropiles can be connected to steel reinforcement of concrete columns extending to underside of bridge. Fewer units required compared to smaller diameter drilled steel casings. Small diameter micropiles drilled using DTH offers best chance of seating and socketing piles in sloping, strong to very strong bedrock. Installed in groups of 3 or 4 within 1.2 m diameter steel casings, backfilled with grout/concrete, structurally advantageous to provide large, stiff cross-section for higher lateral resistance. Avoids the full socketing of the 1.2 m caisson into bedrock. 	<ul style="list-style-type: none"> Requires specialty contractor to install micropiles. Requires more structural design consideration at interface between micropiles and large diameter outer casing. Must have seal for dewatering inside casing; otherwise would require concrete placement by tremie method. Micropile design required. 	<ul style="list-style-type: none"> Costs for pile cap eliminated. Lower cost per pile than larger diameter drilled steel casings, but more piles will be required as a result of lower individual axial resistance. Additional cost for installing 1.2 m diameter outer steel casing. Additional costs for micropile design. 	<ul style="list-style-type: none"> Construction difficulties when installing and socketing 3 or 4 micropiles in close proximity in a single group. May require input/review from micropile contractor at design stage and a template to arrange micropiles within the large diameter casing.



Table 1: Evaluation of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven steel tube piles (Driven into till at abutments and installed in pre-drilled holes into bedrock at pier)	4	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Higher axial resistance compared to spread footings. 	<ul style="list-style-type: none"> ■ Displacement piles - potential for “hanging up” or deflecting on cobbles and boulders within cohesionless deposits or piles founding within dense stratum at shallow depth. ■ Not suitable for integral abutment design. ■ Requires pre-drilling through the very dense soils (upper 3.5 m). ■ Requires shoring system to excavate through wet fill adjacent to existing east abutment for pile cap construction. ■ Likely requires socketing into bedrock at pier due to sloping bedrock surface and to achieve required minimum pile length at south side of pier. ■ Requires cofferdam and unwatering (after tremie concrete plug construction) at pier to allow for pile cap construction in dry conditions. 	<ul style="list-style-type: none"> ■ Relative costs higher than shallow foundations. ■ Mobilization of specialized equipment relatively expensive; more efficient from cost perspective to install at abutments and pier. ■ Unwatering system at pier can be expensive. 	<ul style="list-style-type: none"> ■ Greater potential than the steel H-Piles for not achieving design resistance at design pile tip elevation and therefore have to drive piles to a lower tip elevation (west abutment). ■ Potential risk (compared to H-piles) for difficulty in penetrating into the very dense stratum or through or displacing potential cobbles and boulders. ■ Potential for difficulties seating piles on sloping bedrock surface at the pier. ■ Difficulties in constructing unwatering system at pier and potential for claims.



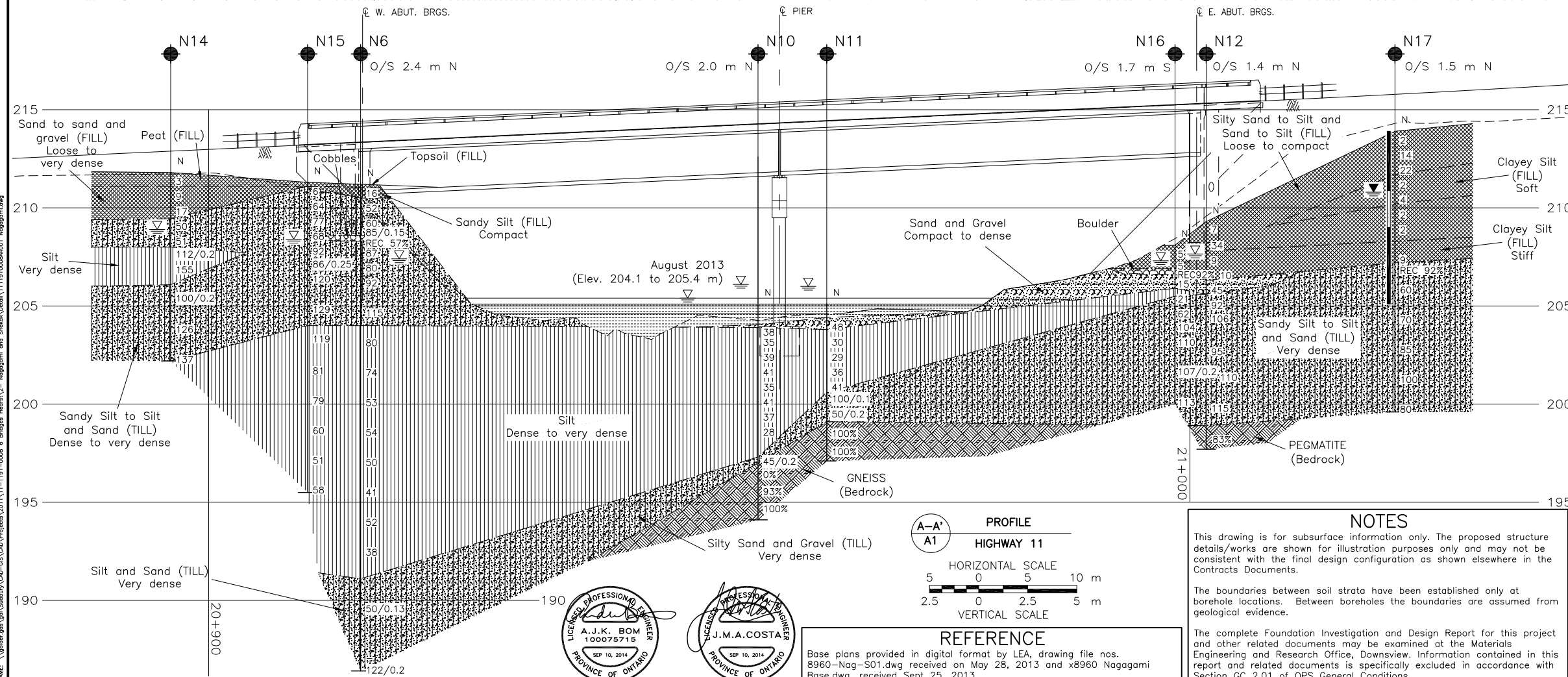
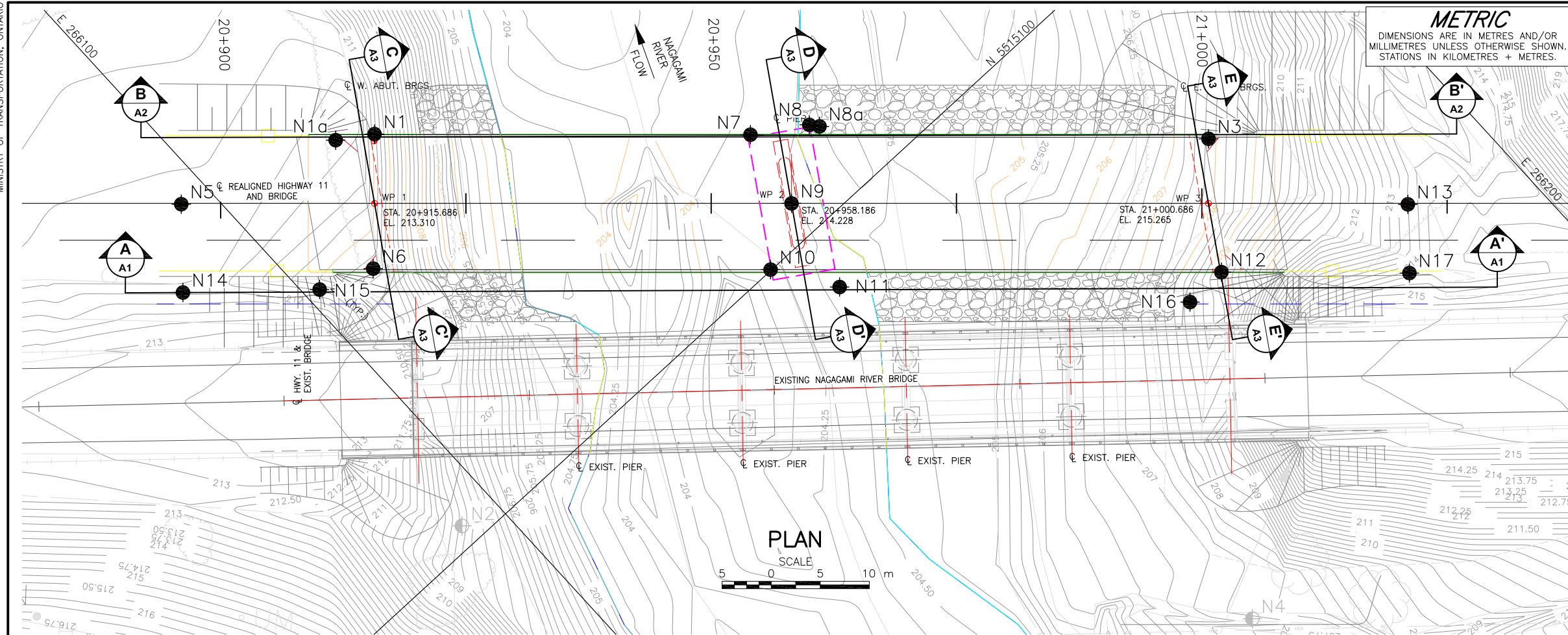
Table 1: Evaluation of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Shallow Foundations (As High a Founding Elevation as Practical); abutments only, not feasible at the pier.	5	<ul style="list-style-type: none"> Conventional construction. Could place footing at west abutment at higher elevation eliminating the need to excavate through very dense soils. 	<ul style="list-style-type: none"> Not suitable for integral abutment design. Lower geotechnical axial resistances than for deep foundations. Variable soil conditions at abutments compared to deep foundations at pier potentially results in differential settlement. Large and difficult excavations through the very dense cohesionless soils at west abutment dependant on founding elevation required. Sub-excavation of the fill with shoring will be required within the footprint of the new east abutment without impacting the existing east abutment/embankment. Would still require a deep foundation system (H piles, pipe piles, micropiles) at the pier and would encounter same issues as with other combination systems, such as unwatering for cap construction at pier. 	<ul style="list-style-type: none"> Typically lower relative cost than deep foundations. Cost could rise substantially due to difficulties associated with shoring and unwatering system installation/operation. 	<ul style="list-style-type: none"> Potential difficulties installing shoring and unwatering system; and increased costs for unwatering, potential for claims. Potential for differential settlement between foundation units. Potential for claims for dewatering system at pier for pile cap construction of deep foundation systems.
Caissons at pier (1.2 m diameter) socketted into bedrock	6	<ul style="list-style-type: none"> Pile cap/shoring not required. Fewer units required compared to smaller diameter drilled steel casings. Shorter socket lengths compared to smaller diameter drilled steel casings. 	<ul style="list-style-type: none"> Equipment not readily available and long lead time may be required. Few contractors have capabilities to attempt such installation. Slow rate of advance. Must have seal for dewatering inside casing; otherwise would require concrete placement by tremie method. Likely need to manufacture a specialized down-the-hole hammer mechanism. Cobbles and boulders within till may cause slow advance. 	<ul style="list-style-type: none"> Foundation costs: \$0.8M to over \$1M excluding any additional costs associated with the high risk; \$500,000 initial set up costs plus caisson installation cost. 	<ul style="list-style-type: none"> Difficulty of seating steel casing in sloping bedrock of high strength and high risk of claims similar to another site with very strong sloping bedrock in Northern Ontario. Potential difficulties with high groundwater pressures. Potential for contractor "Changed Proposal" submission requiring assessment of proposed design.



APPENDIX A

Nagagami River Bridge



PROFILE
HIGHWAY 11
HORIZONTAL SCALE
2.5 0 2.5 5 m
VERTICAL SCALE

REFERENCE

Base plans provided in digital format by LEA, drawing file nos.
8960-Nag-S01.dwg received on May 28, 2013 and x8960 Nagagami
Base.dwg, received Sept 25, 2013.

NOTES

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METRIC
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MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5307-04-00

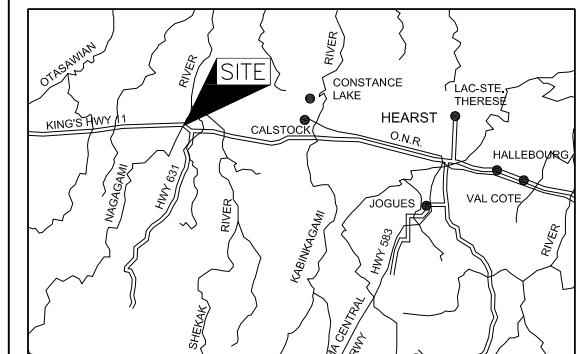
HIGHWAY 11
NAGAGAMI RIVER BRIDGE
BOREHOLE LOCATIONS AND
SOIL STRATA



SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN
SCALE
20 0 20 40 km

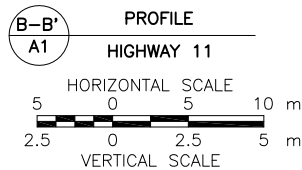
LEGEND

- Borehole
- Borehole - Preliminary 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 Aug 27, 2013
- ▽ WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
N1	211.9	5515137.5	266116.3
N1a	211.9	5515139.8	266113.0
N2	208.8	5515102.1	266095.9
N3	206.8	5515079.8	266178.6
N4	207.2	5515040.7	266148.9
N5	211.8	5515145.6	266096.9
N6	211.2	5515127.5	266106.9
N7	205.2	5515111.6	266144.5
N8	205.2	5515108.3	266149.6
N8a	204.8	5515107.5	266150.3
N9	205.4	5515103.6	266142.9
N10	205.1	5515100.1	266136.7
N11	205.1	5515094.0	266140.7
N12	209.3	5515068.9	266170.4
N13	213.2	5515061.1	266189.1
N14	212.1	5515138.8	266091.0
N15	211.3	5515129.6	266101.5
N16	208.1	5515068.8	266166.0
N17	213.9	5515055.8	266184.5

NO.	DATE	BY	REVISION
1	SEP 10, 2014	J.M.A.COSTA	1
2	SEP 25, 2013	J.M.A.COSTA	2
3	SEP 25, 2013	J.M.A.COSTA	3
4	SEP 25, 2013	J.M.A.COSTA	4
5	SEP 25, 2013	J.M.A.COSTA	5
6	SEP 25, 2013	J.M.A.COSTA	6
7	SEP 25, 2013	J.M.A.COSTA	7
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Base.dwg, received Sept 25, 2013.

NO.	DATE	BY	REVISION	
Geocres No. 42F-28				
HWY. 11		PROJECT NO. 11-1191-0008		DIST.
SUBM'D. AC	CHKD.	DATE: SEP 2014	SITE: 42F-28	
DRAWN: TB	CHKD. AB	APPD. JMAC	DWG. A2	

[illegible]



Table A1 - Summary of Analytical Testing of River Water

Parameter	Units	Result
Resistivity	ohm-cm	5,700
Conductivity	µmho/cm	180
pH	pH	7.92
Sulphate	mg/L	Not Detected
Chloride	mg/L	3

Notes:

1. Sample obtained July 6, 2013
2. Analytical testing carried out by Maxxam Analytics Inc.

Prepared by: AC
Reviewed by: AB

Golder Associates Ltd.
 1010 Lorne Street
 Sudbury, Ontario, Canada P3C 4R9
 Telephone: (705) 524-6861
 Fax: (705) 524-1984



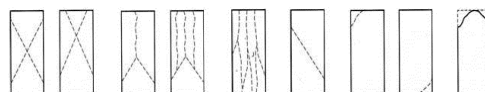
TABLE A2 - SUMMARY OF ROCK CORE TEST DATA

PROJECT NO.: 11-1191-0008
JOB NAME: Nagagami River Bridge
TYPE OF UNIT: Bedrock Core

BOREHOLE	N8	N9	N10	N11	N12
GOLDER LAB #	G0252	GA 1049	GA 1050	GA 1051	GA 1052
DATE TESTED	Apr. 10, 2014	Oct. 22, 2013	Oct. 22, 2013	Oct. 22, 2013	Oct. 22, 2013
TESTED BY	JM	TDM	TDM	TDM	TDM
DEPTH OF TESTED CORE (m)	11.8	10.0	9.1	7.1	11.0
LENGTH (mm)	99.1	99.1	101.2	100.5	100.6
DIAMETER (mm)	47.1	47.3	47.2	47.1	47.3
DENSITY (kg/m3)	2747	2727	2712	2739	2631
COMPRESSIVE STRENGTH (MPa)	69.5	102.2	76.2	109.7	41.1
TYPE OF FRACTURE	3	3	3	4	4

Reviewed by : TG

Type of Fracture



1 2 3 4 5 6

PROJECT		11-1191-0008		RECORD OF BOREHOLE No N1		1 OF 2		METRIC				
G.W.P.		5307-04-00		LOCATION		N 5515137.5; E 266116.3		ORIGINATED BY ID				
DIST		HWY 11		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring		COMPILED BY AC				
DATUM		GEODETIC		DATE		July 26 and 27, 2012		CHECKED BY AB				
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
211.9	GROUND SURFACE											
0.0	Topsoil (FILL)		1a									
0.2	Brown Moist		1b	SS	27							39 46 (15)
	Sand and gravel, some silt (FILL)											
	Compact to dense											
	Brown		2	SS	32		211					
	Moist to wet											
210.4												
1.5	Sandy SILT to SILT and SAND, some clay, trace gravel (TILL)		3	SS	130		210					
	Very dense											
	Brown, grey below 3 m depth		4	SS	105/0.2							
	Moist to wet						209					2 33 50 15
			5	SS	162							
							208					
			6	SS	104							
							207					
			7	SS	175							
							206					
			8	SS	138		205					
			9	SS	135		204					2 24 57 17
							203					
			10	SS	91							
							202					
201.7												
10.2	SILT, some clay, trace sand											
	Very dense		11	SS	89		201					
	Brown						200					
	Moist											
			12	SS	73		199					0 2 82 16
							198					
			13	SS	60							
							197					

PROJECT <u>11-1191-0008</u>		RECORD OF BOREHOLE No N1				2 OF 2 METRIC											
G.W.P. <u>5307-04-00</u>		LOCATION <u>N 5515137.5; E 266116.3</u>				ORIGINATED BY <u>ID</u>											
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring</u>				COMPILED BY <u>AC</u>											
DATUM <u>GEODETIC</u>		DATE <u>July 26 and 27, 2012</u>				CHECKED BY <u>AB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>										
	SILT, some clay, trace sand Very dense Brown Moist		14	SS	54												
							196										
			15	SS	50		195										
							194										
			16	SS	69		193										
							192										
			17	SS	103		191										
191.0							190										
20.9	SILT and SAND, some gravel, trace to some clay (TILL) Very dense Grey Moist Cobbles from 21.2 m to 23.2 m depth.		18	SS	100/0.2		189										
			-	RC	-												
188.7			19	SS	150/0.2												
23.2	END OF BOREHOLE - SEE BOREHOLE N1a (EXTENSION) Note: 1. Water level at a depth of 10.2 m below ground surface (Elev. 201.7 m) on July 28, 2012 after leaving borehole open overnight.																

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 30/07/14 DATA INPUT:



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

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

PROJECT 11-1191-0008		RECORD OF BOREHOLE No N1a				3 OF 3 METRIC											
G.W.P. 5307-04-00		LOCATION N 5515139.8; E 266113.0				ORIGINATED BY EHS											
DIST _____ HWY 11		BOREHOLE TYPE NW Casing, NQ Coring				COMPILED BY AC											
DATUM GEODETIC		DATE March 24 to 26 and April 1, 2014				CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m ³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	20 40 60	20 40 60	20 40 60	20 40 60					
--- CONTINUED FROM PREVIOUS PAGE ---																	
	SILT and SAND, trace to some gravel, trace to some clay (TILL) Very dense Grey Moist		5	SS	93		181										
	A 150 mm cobble encountered at 31.7 m depth.						180										
							179										
			6	SS	92		178										
							177										
							176										
			7	SS	157		175										3 28 46 23
							174										
							173										
	A 150 mm cobble encountered at 39.7 m depth.		8	SS	100/0.2		172										
171.7	GNEISS (BEDROCK)						171										RQD = 100%
40.2	Bedrock cored from 40.2 m depth to 43.3 m depth. For coring details see Record of Drillhole N1a.		1	RC	REC 100%		170										RQD = 95%
			2	RC	REC 100%		169										RQD = 100%
168.6	END OF BOREHOLE																
43.3	Note: 1. Borehole N1a located 4 m west of Borehole N1.																

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 3007/14 DATA INPUT:

PROJECT: 11-1191-0008

RECORD OF DRILLHOLE: N1a

SHEET 1 OF 1

LOCATION: N 5515139.8 ;E 266113.0

DRILLING DATE: March 24 to 26 and April 1, 2014

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25

DRILLING CONTRACTOR: Walker

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.																NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
								RECOVERY			R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY k, cm/s			Diametral Point Load Index (MPa)	RMC -Q AVG																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
								FLUSH	TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10 ⁰	10 ¹	10 ²																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
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DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 11-1191-0008 DETAIL GPJ GAL-MISS.GDT 30/07/14 DATA INPUT:

1 OF 2 **METRIC**

ORIGINATED BY ID



COMPILED BY AC

CHECKED BY _____ AD _____

Continued Next Page

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




USUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 30/07/14 DATA INPUT:

PROJECT <u>11-1191-0008</u>			RECORD OF BOREHOLE No N2			2 OF 2 METRIC													
G.W.P. <u>5307-04-00</u>			LOCATION <u>N 5515102.1; E 266095.9</u>			ORIGINATED BY <u>ID</u>													
DIST <u> </u> HWY <u>11</u>			BOREHOLE TYPE <u>108 mm Hollow Stem Augers, NW Casing, Wash Boring</u>			COMPILED BY <u>AC</u>													
DATUM <u>GEODETIC</u>			DATE <u>July 28 and 29, 2012</u>			CHECKED BY <u>AB</u>													
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa											
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>												
	Sandy SILT to SILT, some clay Dense to very dense Grey Moist to wet Clayey silt seams in sample 14 (15.2 m to 15.8 m).		14	SS	31		193												
							192												
			15	SS	30		191												
190.3			16	SS	100/0.1														
189.9	GRAVEL and COBBLES Dense Grey Wet		-	RC	-		190												
18.9	END OF BOREHOLE Note: 1. Water level at a depth of 3.3 m below ground surface (Elev. 205.5 m) upon completion of drilling.																		

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 30/07/14 DATA INPUT:

[illegible]

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

PROJECT		RECORD OF BOREHOLE No N5				1 OF 1 METRIC											
G.W.P. 5307-04-00		LOCATION N 5515145.6; E 266096.9				ORIGINATED BY SA											
DIST _____ HWY 11		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY AC											
DATUM GEODETIC		DATE August 7, 2013				CHECKED BY AB											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
211.8	GROUND SURFACE							20	40	60	80	100					
0.0	Peat (FILL) Soft to firm Brown Moist		1	SS	4												
211.2																	
0.6	Sandy SILT, trace to some gravel, trace clay (TILL) Compact to very dense Brown, grey below 2 m depth. Moist to wet		2	SS	15												
	Augers grinding between 1.5 m and 2.1 m depth and at 3.0 m depth.		3	SS	45												
			4	SS	85/0.25												
			5	SS	74												
			6	SS	88												
			7	SS	77												
206.6	END OF BOREHOLE																
5.2	Note: 1. Water level in open borehole at a depth of 4.5 m below ground surface (Elev. 207.3 m) upon completion of drilling.																

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 3007/14 DATA INPUT:

PROJECT 11-1191-0008			RECORD OF BOREHOLE No N6			1 OF 2 METRIC											
G.W.P. 5307-04-00			LOCATION N 5515127.5; E 266106.9			ORIGINATED BY SA											
DIST _____ HWY 11			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring			COMPILED BY AC											
DATUM GEODETIC			DATE August 8 and 9, 2013			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	γ	GR SA SI CL			
211.2	GROUND SURFACE																
0.0	Topsoil (FILL)		1	SS	16		211										
0.2	Sandy silt, some gravel (FILL)																
210.5	Compact Brown to grey Moist		2	SS	52		210										
0.7	Sandy SILT, some clay (TILL) Very dense Grey Moist		3	SS	60		209							0 24 58 18			
	Auger grinding at 1.4 m depth.		4A	SS	85/0.15												
208.6	COBBLE		4B	RC	REC 57%												
208.3	Sandy SILT, some clay, trace gravel (TILL) Dense to very dense Brown to grey Moist to wet		5	SS	87		208										
2.9			6	SS	80		207							8 30 47 15			
			7	SS	92		206										
			8	SS	115		205										
204.0	SILT, some clay, trace sand, trace gravel Dense to very dense Brown Moist to wet		9	SS	80		204										
7.2			10	SS	74		203										
	125 mm cobble encountered at 8.7 m depth.		11	SS	53		202							0 2 79 19			
			12	SS	54		201										
			13	SS	50		200										
							199										
							198										
							197							3 3 78 16			

Continued Next Page

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

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 3007/14 DATA INPUT:

PROJECT <u>11-1191-0008</u>		RECORD OF BOREHOLE No N6		2 OF 2 METRIC								
G.W.P. <u>5307-04-00</u>		LOCATION <u>N 5515127.5; E 266106.9</u>		ORIGINATED BY <u>SA</u>								
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring</u>		COMPILED BY <u>AC</u>								
DATUM <u>GEODETIC</u>		DATE <u>August 8 and 9, 2013</u>		CHECKED BY <u>AB</u>								
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES					
	--- CONTINUED FROM PREVIOUS PAGE ---											
	SILT, some clay, trace sand, trace gravel Dense to very dense Brown Moist to wet		14	SS	41							
			15	SS	52							
			16	SS	38							
191.1												
20.1	SILT and SAND, some clay, trace to some gravel (TILL) Very dense Grey Moist		17	SS	50/0.13							
			18	CS	-							
186.4			19	SS	122/0.2							
24.8	END OF BOREHOLE Note: 1. Water level at a depth of 3.8 m below ground surface (Elev. 207.4 m) on August 10, 2013 after leaving borehole open overnight.											

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 30/07/14 DATA INPUT:

PROJECT 11-1191-0008				RECORD OF BOREHOLE No N7				1 OF 2 METRIC									
G.W.P. 5307-04-00				LOCATION N 551511.6; E 266144.5				ORIGINATED BY EHS									
DIST _____ HWY 11				BOREHOLE TYPE HW Casing, NW Casing, NQ Coring				COMPILED BY AC									
DATUM GEODETIC				DATE August 10, 2013				CHECKED BY AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
205.2	WATER SURFACE							20	40	60	80	100					
0.0	WATER																
203.9	Cobbles and boulders observed on river bed.																
1.3	SILT, some clay, trace sand Compact to dense Brown to grey Wet		1	SS	38												
			2	SS	38												
			3	SS	35												
			4	SS	32												
			5	SS	30												
	Clay layers (25 mm to 50 mm thick) below 5.3 m depth.		6	SS	23												
			7	SS	33												
			8	SS	17												
196.5	Sandy SILT, some clay (TILL) Very dense Grey Wet		-	RC	REC 100%												
8.7	GRAVEL and COBBLES and BOULDERS (including two 0.5 m boulders) to 10.5 m depth.		9	SS	35/0.15												
			-	RC	REC 100%												
			10	SS	130												
			11	SS	94												
			12	SS	82/0.2												
191.1																	
14.1																	

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 3007/14 DATA INPUT:

Continued Next Page

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

PROJECT <u>11-1191-0008</u>		RECORD OF BOREHOLE No N7		2 OF 2 METRIC	
G.W.P. <u>5307-04-00</u>		LOCATION <u>N 5515111.6; E 266144.5</u>		ORIGINATED BY <u>EHS</u>	
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>HW Casing, NW Casing, NQ Coring</u>		COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 10, 2013</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20	40	60	80	100	WATER CONTENT (%)							
	<div>--- CONTINUED FROM PREVIOUS PAGE ---</div> <div>END OF BOREHOLE</div> <div>Note: 1. Water level at 1.0 m above river surface (Elev. 206.2 m) inside casing upon completion of drilling.</div>																				

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 30/07/14 DATA INPUT:

PROJECT 11-1191-0008			RECORD OF BOREHOLE No N8			1 OF 1 METRIC															
G.W.P. 5307-04-00			LOCATION N 5515108.3; E 266149.6			ORIGINATED BY EHS															
DIST _____ HWY 11			BOREHOLE TYPE HQ Casing, NW Casing, NQ Coring			COMPILED BY AC															
DATUM GEODETIC			DATE August 11 and 12, 2013			CHECKED BY AB															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
205.2	WATER SURFACE							20 40 60 80 100	20 40 60 80 100	20 40 60											
0.0	WATER						205														
204.3	Cobbles and boulders observed on river bed.						204														
1.1	SAND and GRAVEL Brown Wet		1	SS	23		204														
	SILT, some clay, trace sand Compact to very dense Brown to grey Wet		2	SS	50		203														
			3	SS	33		202														
			4	SS	34		201														
	Clay layers (25 mm to 50 mm thick) below 3.7 m depth.		5	SS	24		200														
			6	SS	15		199														
			7	SS	17		198														
			8	SS	15		197														
	A 150 mm cobble was encountered at 7.9 m depth.		9	SS	34		196														
196.2	SILT and SAND, some clay, trace to some gravel (TILL) Very dense Grey Moist		10	SS	118		196														
195.3			-	RC	REC 100%																
195.3	Boulder (400mm) at 9.5 m depth. END OF BOREHOLE - SEE BOREHOLE N8a (EXTENSION) Note: 1. Water level inside casing at 1.0 m above river surface (Elev. 206.2 m) upon completion of drilling.																				

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 3007/14 DATA INPUT:

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

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

USUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 30/07/14 DATA INPUT:

DATUM: GEODETIC

DRILL RIG: D 25

DRILLING CONTRACTOR: Walker

[illegible]

DEPTH SCALE

1 : 50

LOGGED: EHS

CHECKED: AB

PROJECT		11-1191-0008		RECORD OF BOREHOLE No N9		1 OF 1 METRIC											
G.W.P.		5307-04-00		LOCATION		N 5515103.6; E 266142.9											
DIST		HWY 11		BOREHOLE TYPE		HQ Casing, NW Casing, NQ Coring											
DATUM		GEODETIC		DATE		August 8 and 9, 2013											
				ORIGINATED BY		EHS											
				COMPILED BY		AC											
				CHECKED BY		AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		WATER CONTENT (%)		γ		GR SA SI CL		
205.4	0.0	WATER SURFACE							20 40 60 80 100	20 40 60							
		WATER															
204.3		Cobbles and boulders observed on river bed.						205									
204.0	1.4	SAND and GRAVEL Very dense Brown Wet		1	SS	107		204									
		SILT, some clay, trace sand Compact to dense Brown to grey Wet		2	SS	31										0 1 83 16	
				3	SS	39		203									
				4	SS	44		202									
				5	SS	38		201									
		Clay layers (25 mm to 50 mm thick) below 4.5 m depth.		6	SS	21		200								1 1 50 48	
				7	SS	32											
				8	SS	30		199									
				9A	SS	42		198									
197.4	8.0	Silty SAND and GRAVEL (TILL) Very dense Grey Wet		9B				197								39 33 24 4	
195.9	9.5	GNEISS (BEDROCK)		10	SS	100/0.1		196									
		Bedrock cored from 9.5 m to 12.7 m depth.		1	RC	REC 100%		195								RQD = 81%	
		For coring details see Record of Drillhole N9.		2	RC	REC 100%		194								RQD = 84%	
				3	RC	REC 100%		193								RQD = 92%	
192.7	12.7	END OF BOREHOLE															
		Note: 1. Water level inside casing at a depth of 0.3 m below river surface (Elev. 205.1 m) upon completion of drilling.															

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 3007/14 DATA INPUT:

PROJECT: 11-1191-0008

RECORD OF DRILLHOLE: N9

SHEET 1 OF 1

LOCATION: N 5515103.6 ;E 266142.9

DRILLING DATE: August 8 and 9, 2013

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D-25 BARGE

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.																		NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	B Angle	DIP w.r.t CORE AXIS	DISCONTINUITY DATA			HYDRAULIC CONDUCTIVITY			Diametral Point Load Index (MPa)	RMC -Q AVG																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
								TOTAL CORE %	SOLID CORE %					TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	k ₁	k ₂			k ₃																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
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DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 11-1191-0008 DETAIL GPJ GAL-MISS.GDT 30/07/14 DATA INPUT:

PROJECT 11-1191-0008				RECORD OF BOREHOLE No N10				1 OF 1 METRIC									
G.W.P. 5307-04-00				LOCATION N 5515100.1; E 266136.7				ORIGINATED BY EHS									
DIST _____ HWY 11				BOREHOLE TYPE HQ Casing, NW Casing, NQ Coring				COMPILED BY AC									
DATUM GEODETIC				DATE August 13 and 19, 2013				CHECKED BY AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
205.1	WATER SURFACE							20	40	60	80	100					
0.0	WATER						205										
204.0	Cobbles and boulders observed on river bed.						204										
1.2	SAND and GRAVEL Brown Wet		1	SS	38		203										
	SILT, some clay, trace sand Compact to dense Brown to grey Wet		2	SS	35		202										
			3	SS	39		201										
			4	SS	41		200										
			5	SS	35		199										
			6	SS	41		198										
	Clay layers (25 mm thick) below 5.3 m depth.		7	SS	37		197										
			8	SS	28		196										
197.3	Silty SAND and GRAVEL (TILL)		9	SS	45/0.2		195										
197.0	Very dense Grey Wet		1	RC	REC 100%												RQD = 0%
8.1	GNEISS (BEDROCK)																
	Bedrock cored from 8.1 m to 11.0 m depth.		2	RC	REC 100%												RQD = 93%
	For coring details see Record of Drillhole N10.		3	RC	REC 100%												RQD = 100%
194.1	END OF BOREHOLE																
11.0	Note: 1. Water level inside casing at 1.0 m above river surface (Elev. 206.1 m) upon completion of drilling.																

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 30/07/14 DATA INPUT:

SHEET 1 OF 1

DATUM: GEODETIC

DRILLING CONTRACTOR: Walker

CHECKED: AB

SUD-RCK 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 30/07/14 DATA INPUT:

PROJECT 11-1191-0008			RECORD OF BOREHOLE No N11			1 OF 1 METRIC											
G.W.P. 5307-04-00			LOCATION N 5515094.0; E 266140.7			ORIGINATED BY EHS											
DIST _____ HWY 11			BOREHOLE TYPE HW Casing, NW Casing, NQ Coring			COMPILED BY AC											
DATUM GEODETIC			DATE August 19 and 20, 2013			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
							20 40 60 80 100	○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED	W _p	W	W _L	20 40 60				
205.1	WATER SURFACE						205										
0.0	WATER																
204.4	Cobbles and boulders observed on river bed.																
0.7	SAND and GRAVEL		1	SS	48		204										
203.8	Dense Brown Wet																
1.3	SILT, some clay, trace sand, trace gravel Compact to dense Brown to grey Wet		2	SS	30		203									0 2 82 16	
			3	SS	29												
	Clay layers (25 mm to 50 mm thick) below 3.0 m depth.		4	SS	36		202										
			5	SS	41		201									2 4 62 32	
200.6	Silty SAND and GRAVEL (TILL)		6	SS	100/0.1		200										
4.5	Very dense Grey Wet		7	SS	50/0.2												
199.1	GNEISS (BEDROCK)						199									RQD = 100%	
6.0	Bedrock cored from 6.0 m to 8.0 m depth. For coring details see Record of Drillhole N11.		1	RC	REC 100%												
			2	RC	REC 100%		198									RQD = 100%	
197.1	END OF BOREHOLE																
8.0	Note: 1. Water level inside casing at 0.9 m above river surface (Elev. 206.0 m) upon completion of drilling.																

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 30/07/14 DATA INPUT:

PROJECT: 11-1191-0008

RECORD OF DRILLHOLE: N11

SHEET 1 OF 1

LOCATION: N 5515094.0 ;E 266140.7

DRILLING DATE: August 19 and 20, 2013

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D-25 BARGE

DRILLING CONTRACTOR: Walker

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.										NOTES WATER LEVELS INSTRUMENTATION			
							RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY k, cm/s		Diametral Point Load Index (MPa)	RMC -Q AVG		
							TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn				
6	NQ	REFER TO PREVIOUS PAGE		199.1 6.0	1	GREY 100%							FORo MB MB							
7	NW	GNEISS with pegmatite sills and inclusions Very strong Fine to very coarse grained Fresh Grey			2	GREY 100%							JNFORo JNPLRo MB MB JNPLRo MB							UCS=110 MPa
8		END OF DRILLHOLE		197.1 8.0																
9																				
10																				
11																				
12																				
13																				
14																				
15																				
16																				

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 11-1191-0008 DETAIL GPJ GAL-MISS.GDT 30/07/14 DATA INPUT:

PROJECT 11-1191-0008			RECORD OF BOREHOLE No N12			1 OF 1 METRIC											
G.W.P. 5307-04-00			LOCATION N 5515068.9; E 266170.4			ORIGINATED BY EHS											
DIST _____ HWY 11			BOREHOLE TYPE NW Casing, NQ Coring			COMPILED BY AC											
DATUM GEODETIC			DATE August 24 to 26, 2013			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60				
209.3	GROUND SURFACE																
0.0	Silty sand to silt and sand, trace organics, trace to some gravel (FILL) Loose to dense Brown Moist		1	SS	7		209										
			2	SS	34		208										
207.8	Clayey silt, some sand, peat pockets/seams (FILL) Stiff Grey Wet		3	SS	9		207										
			4	SS	10												
206.3	SAND and GRAVEL, some silt, trace clay Dense Brown Wet		5	SS	45		206										48 33 14 5
205.6	SILT and SAND, some clay, trace gravel (TILL) Very dense Grey Wet		6	SS	92		205										
			7	SS	106		204										3 30 47 20
			8	SS	95		203										
			9	SS	110		202										1 30 51 18
			10	SS	115		201										
							200										
198.9	PEGMATITE (BEDROCK)						199										
10.4	Bedrock cored from 10.4 m to 11.6 m depth. For coring details see Record of Drillhole N12.		1	RC	REC 100%		198										RQD = 83%
197.7	END OF BOREHOLE																
11.6	Note: 1. Water level in open borehole at a depth of 1.6 m below ground surface (Elev. 207.7 m) upon completion of drilling.																

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS GDT 3007/14 DATA INPUT:

PROJECT: 11-1191-0008

RECORD OF DRILLHOLE: N12

SHEET 1 OF 1

LOCATION: N 5515068.9 ;E 266170.4

DRILLING DATE: August 24 to 26, 2013

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D-25 BARGE

DRILLING CONTRACTOR: Walker

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate										BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage										PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular										PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break										BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																							
									RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY		Diametral Point Load Index (MPa)	RMC -Q AVG																																																																																																																																																																																																																																																
									TOTAL CORE %	SOLID CORE %			B Angle		DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION		Jr	Ja	Jn	k, cm/s																																																																																																																																																																																																																																																					
									80 60 40 20 0	80 60 40 20 0			0 90 180 270 360	0 90 180 270 360		0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360			0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360					0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360	0 90 180 270 360

DEPTH SCALE

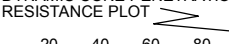


1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 11-1191-0008 DETAIL GPJ GAL-MISS.GDT 30/07/14 DATA INPUT:

PROJECT 11-1191-0008			RECORD OF BOREHOLE No N13			1 OF 1 METRIC					
G.W.P. 5307-04-00			LOCATION N 5515061.1; E 266189.1			ORIGINATED BY SA					
DIST _____ HWY 11			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing			COMPILED BY AC					
DATUM GEODETIC			DATE August 13 and 19, 2013			CHECKED BY AB					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
213.2	GROUND SURFACE										
0.0	Sand, trace to some gravel, some organics (FILL)		1	SS	WH		213				
12.6	Very loose Brown Moist		2	SS	9		212				2 14 59 25
0.6	Clayey silt to sandy silt, trace to some gravel (FILL)		3	SS	2						
	Very soft to stiff Brown to grey Moist to wet		4	SS	2		211				
	Peat pockets/seams noted in Samples 4 to 7.		5	SS	3		210				9 27 48 16
			6	SS	1		209				
			7	SS	5		208				
207.6	Sandy SILT, some clay, trace gravel (TILL)		8	SS	50/0.1		207				4 24 57 15
5.6	Very dense Grey Wet						206				
			9	SS	50/0.1						
205.0	END OF BOREHOLE						205				
8.2	Note: 1. Water level at a depth of 3.2 m below ground surface (Elev. 210.0m) upon completion of drilling.										

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 30/07/14 DATA INPUT:

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

PROJECT 11-1191-0008			RECORD OF BOREHOLE No N15			1 OF 2 METRIC														
G.W.P. 5307-04-00			LOCATION N 5515129.6; E 266101.5			ORIGINATED BY SA														
DIST _____ HWY 11			BOREHOLE TYPE 108 mm Continuous Flight Hollow Stem Augers, NW Casing			COMPILED BY AC														
DATUM GEODETIC			DATE August 10, 2013			CHECKED BY AB														
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m ³	GR SA SI CL			
							20 40 60 80 100	20 40 60 80 100	20 40 60	20 40 60	20 40 60									
211.3	GROUND SURFACE																			
0.0	Sand and gravel, trace organics (FILL) Brown Moist		1	SS	6		211													
0.2	Sandy SILT to SILT and SAND, some clay, trace to some gravel (TILL) Very dense Brown turning grey below 1.5 m depth Moist to wet		2	SS	64		210										10 30 45 15			
			3	SS	77		209													
			4	SS	68		208										12 16 54 18			
			5	SS	92		207													
	75 mm cobble at 4.2 m depth.		6	SS	86/0.25		206													
			7	SS	120		205										6 29 49 16			
			8	SS	129		204													
204.0	SILT, some clay, trace sand Very dense Grey Wet		9	SS	119		203													
7.3			10	SS	81		202													
			11	SS	79		201										0 1 79 20			
			12	SS	60		200													
			13	SS	51		199													
							198													
							197													

Continued Next Page

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


SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 3007/14 DATA INPUT:

PROJECT <u>11-1191-0008</u>			RECORD OF BOREHOLE No N15				2 OF 2 METRIC	
G.W.P. <u>5307-04-00</u>			LOCATION <u>N 5515129.6; E 266101.5</u>				ORIGINATED BY <u>SA</u>	
DIST <u> </u> HWY <u>11</u>			BOREHOLE TYPE <u>108 mm Continuous Flight Hollow Stem Augers, NW Casing</u>				COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>			DATE <u>August 10, 2013</u>				CHECKED BY <u>AB</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W		
195.5	SILT, some clay, trace sand Very dense Grey Wet	14	SS	58		196										
15.8	END OF BOREHOLE Note: 1. Water level at a depth of 2.9 m below ground surface (Elev. 208.4 m) on August 11, 2013 after leaving borehole open overnight.															

PROJECT 11-1191-0008			RECORD OF BOREHOLE No N16			1 OF 1 METRIC																				
G.W.P. 5307-04-00			LOCATION N 5515068.8; E 266166.0			ORIGINATED BY EHS																				
DIST _____ HWY 11			BOREHOLE TYPE NW Casing, NQ Coring			COMPILED BY AC																				
DATUM GEODETIC			DATE August 27, 2013			CHECKED BY AB																				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE	20	40	60	80	100	W _p	W	W _L	γ	GR	SA	SI	CL						
208.1	GROUND SURFACE						208																			
0.0	Sand to silty sand, trace gravel, trace asphalt, trace organics (FILL) Loose Brown Moist to wet		1	SS	5		207																			
206.9			2	SS	5		206																			
206.6	BOULDER		-	RC	REC 92%		205																			
1.5	SAND and GRAVEL, some silt Compact Brown Wet		3	SS	15		204																			
205.9			4a	SS	21		203																			
205.5	SILT, clay Compact Brown to grey Wet		4b	SS	21		202																			
2.6	SILT and SAND, some clay, trace to some gravel (TILL) Very dense Grey Wet		5	SS	62		201																			
			6	SS	104		200																			
			7	SS	110																					
			8	SS	107/0.2																					
			9	SS	113																					
200.0	END OF BOREHOLE																									
8.1	Note: 1. Water level inside casing at a depth of 0.8 m below ground surface (Elev. 207.3 m) upon completion of drilling.																									

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 30/07/14 DATA INPUT:

PROJECT <u>11-1191-0008</u>				RECORD OF BOREHOLE No N17				1 OF 2 METRIC						
G.W.P. <u>5307-04-00</u>				LOCATION <u>N 5515055.8; E 266184.5</u>				ORIGINATED BY <u>SA</u>						
DIST <u> </u> HWY <u>11</u>				BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing</u>				COMPILED BY <u>AC</u>						
DATUM <u>GEODETIC</u>				DATE <u>August 12 and 13, 2013</u>				CHECKED BY <u>AB</u>						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
213.9	GROUND SURFACE													
0.0	Sand, trace organics (FILL) Very loose Brown Moist		1	SS	2									
213.2	Silt, some sand (FILL) Compact Brown to grey Moist		2	SS	14									
0.7			3	SS	22									
211.7	Clayey silt, some sand, trace gravel, trace organics (FILL) Soft Brown to grey Wet		4	SS	2									
2.2			5	SS	4									
210.2	Sandy silt, some clay, trace gravel (FILL) Very loose Grey Wet		6	SS	2									
3.7			7	SS	2									
208.3	Clayey silt, some sand, trace gravel, trace organics (FILL) Stiff Grey Wet		8	SS	9									
5.6			9	RC	REC 92%									
207.2	Sandy SILT, some clay, trace gravel (TILL) Very dense Grey Wet		10	SS	60									
6.7			11	SS	70									
			12	SS	85									
			13	SS	100									
			14	SS	80									
199.6														
14.3														

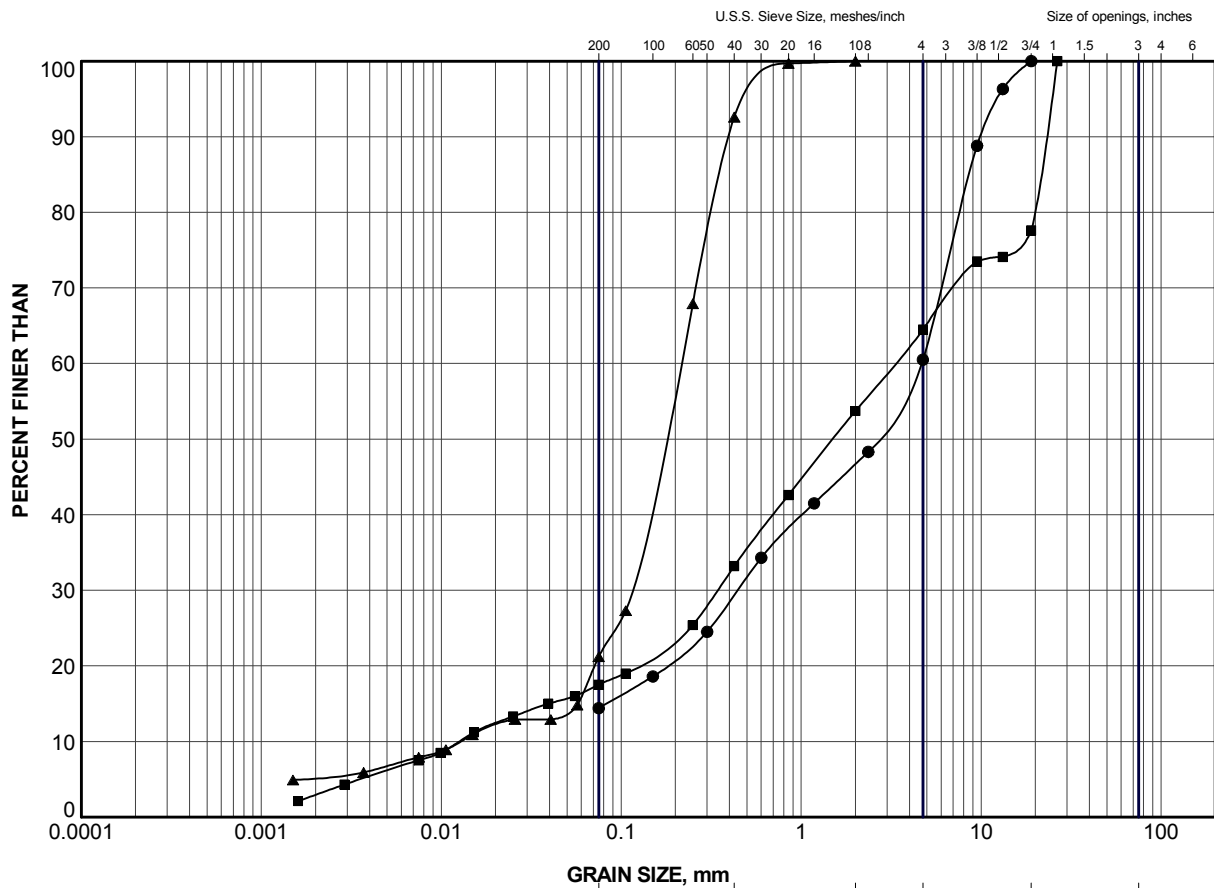
Continued Next Page

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

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 3007/14 DATA INPUT:


PROJECT <u>11-1191-0008</u>		RECORD OF BOREHOLE No N17				2 OF 2 METRIC							
G.W.P. <u>5307-04-00</u>		LOCATION <u>N 5515055.8; E 266184.5</u>				ORIGINATED BY <u>SA</u>							
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, NW Casing</u>				COMPILED BY <u>AC</u>							
DATUM <u>GEODETIC</u>		DATE <u>August 12 and 13, 2013</u>				CHECKED BY <u>AB</u>							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa		W _p	W		
	--- CONTINUED FROM PREVIOUS PAGE ---												
	END OF BOREHOLE Note: 1. Water level at a depth of 3.2 m below ground surface (Elev. 210.7 m) upon completion of drilling. 2. Water level at a depth of 3.1 m below ground surface (Elev. 210.8 m) in piezometer on August 27, 2013.												

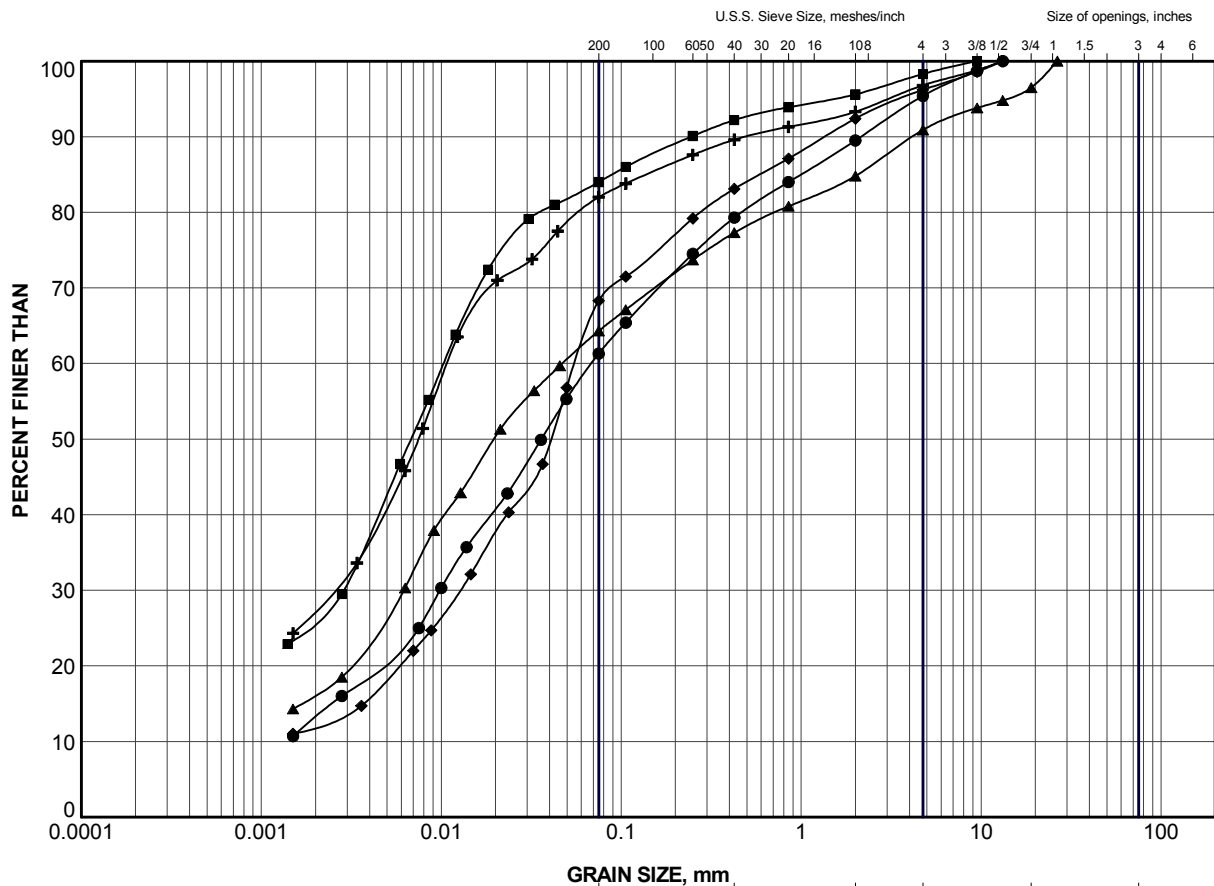
SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 30/07/14 DATA INPUT:



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	N1	1b	211.5
■	N3	2	205.7
▲	N14	3	210.3


PROJECT					HIGHWAY 11 NAGAGAMI RIVER BRIDGE				
TITLE					GRAIN SIZE DISTRIBUTION SAND to SAND AND GRAVEL (FILL)				
PROJECT No.		11-1191-0008		FILE No.		1191-0008 DETAIL.GPJ			
DRAWN	JJL	Jan 2014		SCALE	N/A	REV.			
CHECK	AB	Jan 2014		FIGURE A1					
APPR	JMAC	Jan 2014							
 Golder Associates SUDBURY, ONTARIO									

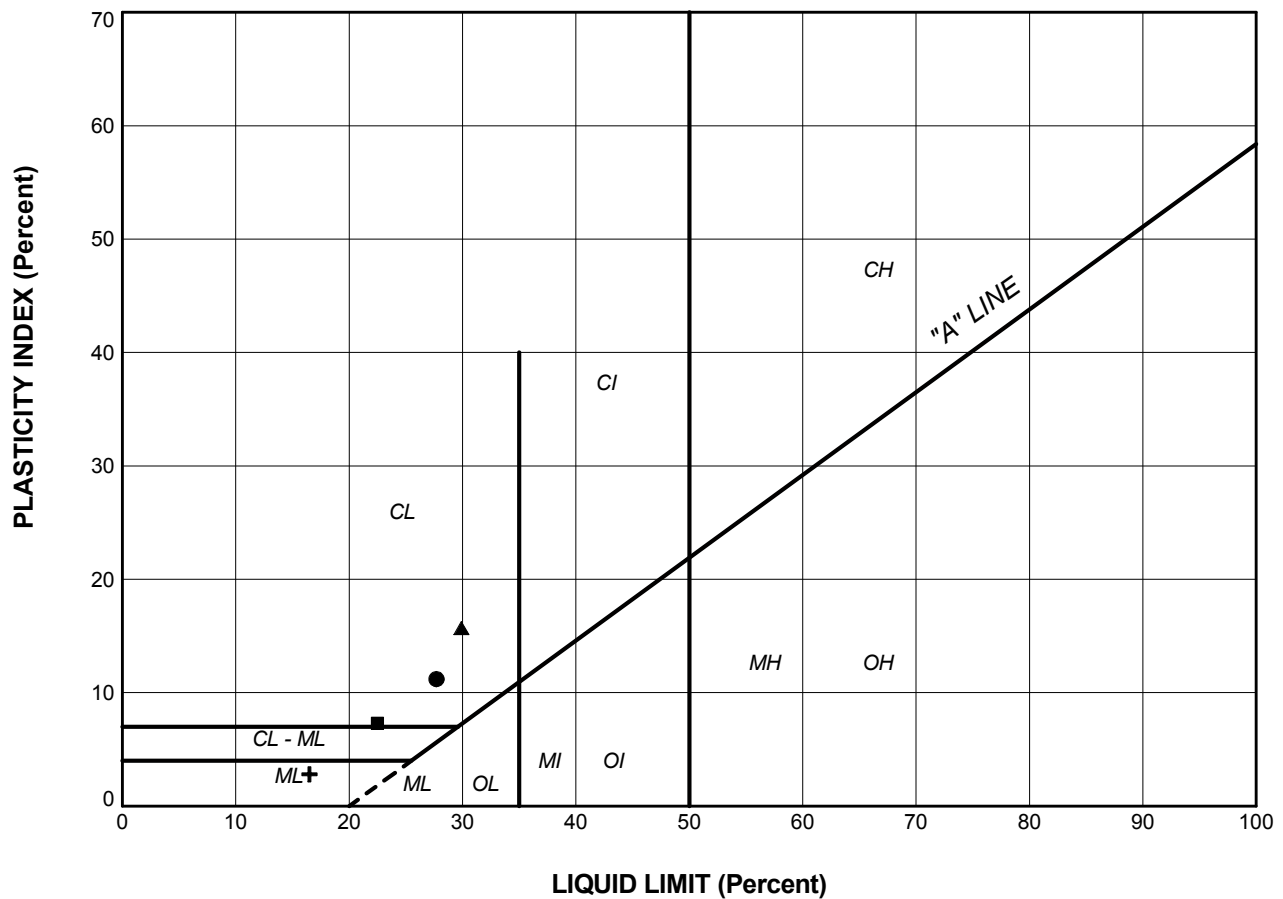


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	N2	1b	208.4
■	N13	2	212.1
▲	N13	5	209.8
⊕	N17	5	210.5
◆	N17	7	209.0

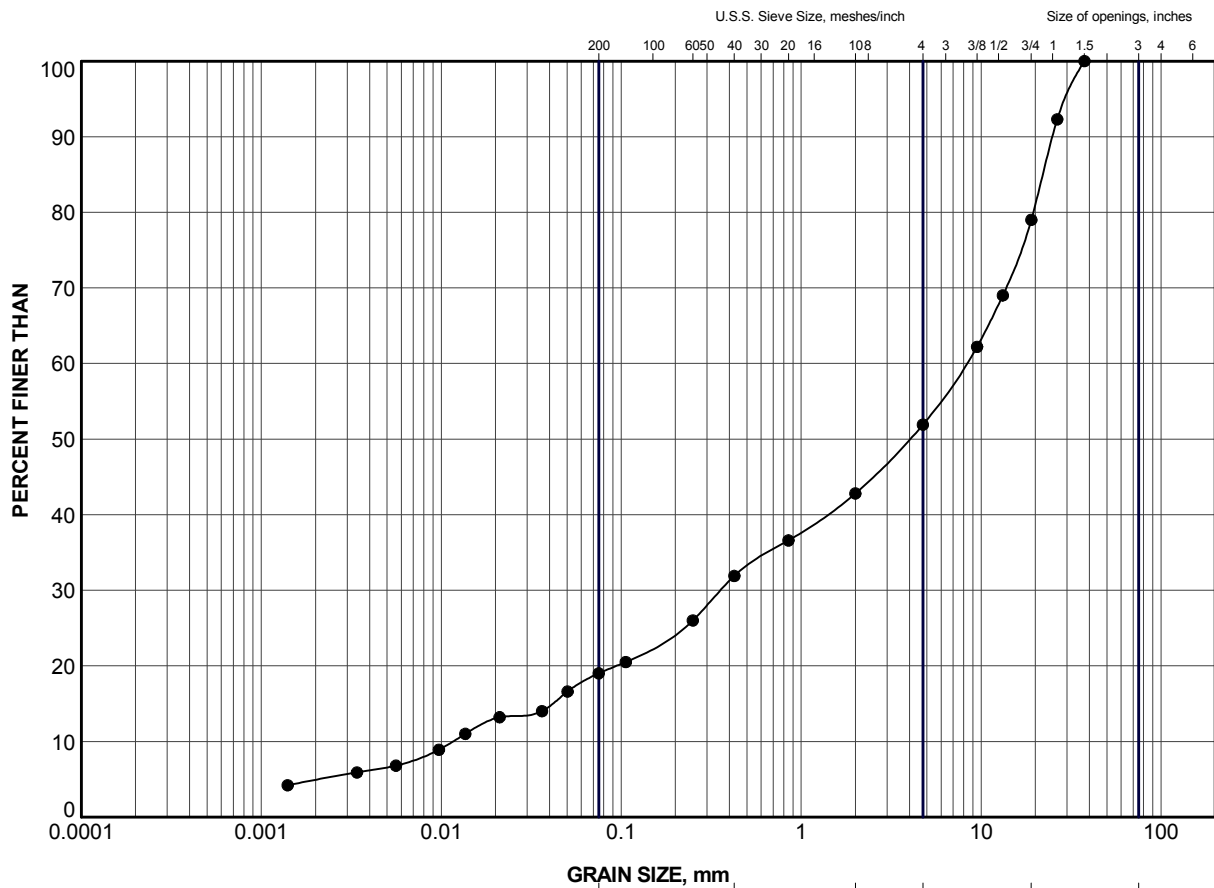
PROJECT						HIGHWAY 11 NAGAGAMI RIVER BRIDGE											
TITLE						GRAIN SIZE DISTRIBUTION SANDY SILT to SILT and SAND to CLAYEY SILT (FILL)											
PROJECT No.						11-1191-0008						FILE No. 1191-0008 DETAIL.GPJ					
DRAWN		J.J.L.		Jan 2014		SCALE		N/A		REV.							
CHECK		AB		Jan 2014													
APPR		JMAC		Jan 2014													
 Golder Associates SUDBURY, ONTARIO						FIGURE A2											



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	N13	2	27.7	16.5	11.2
■	N13	5	22.5	15.2	7.3
▲	N17	5	29.9	14.2	15.7
+	N17	7	16.5	13.7	2.8


PROJECT					HIGHWAY 11 NAGAGAMI RIVER BRIDGE						
TITLE					PLASTICITY CHART SANDY SILT to CLAYEY SILT (FILL)						
PROJECT No. 11-1191-0008			FILE No. 11-1191-0008 DETAIL.GPJ		DRAWN J.J.L. Jan 2014			SCALE N/A		REV.	
CHECK AB Jan 2014			APPR JMAC Jan 2014			Golder Associates SUDBURY, ONTARIO			FIGURE A3		

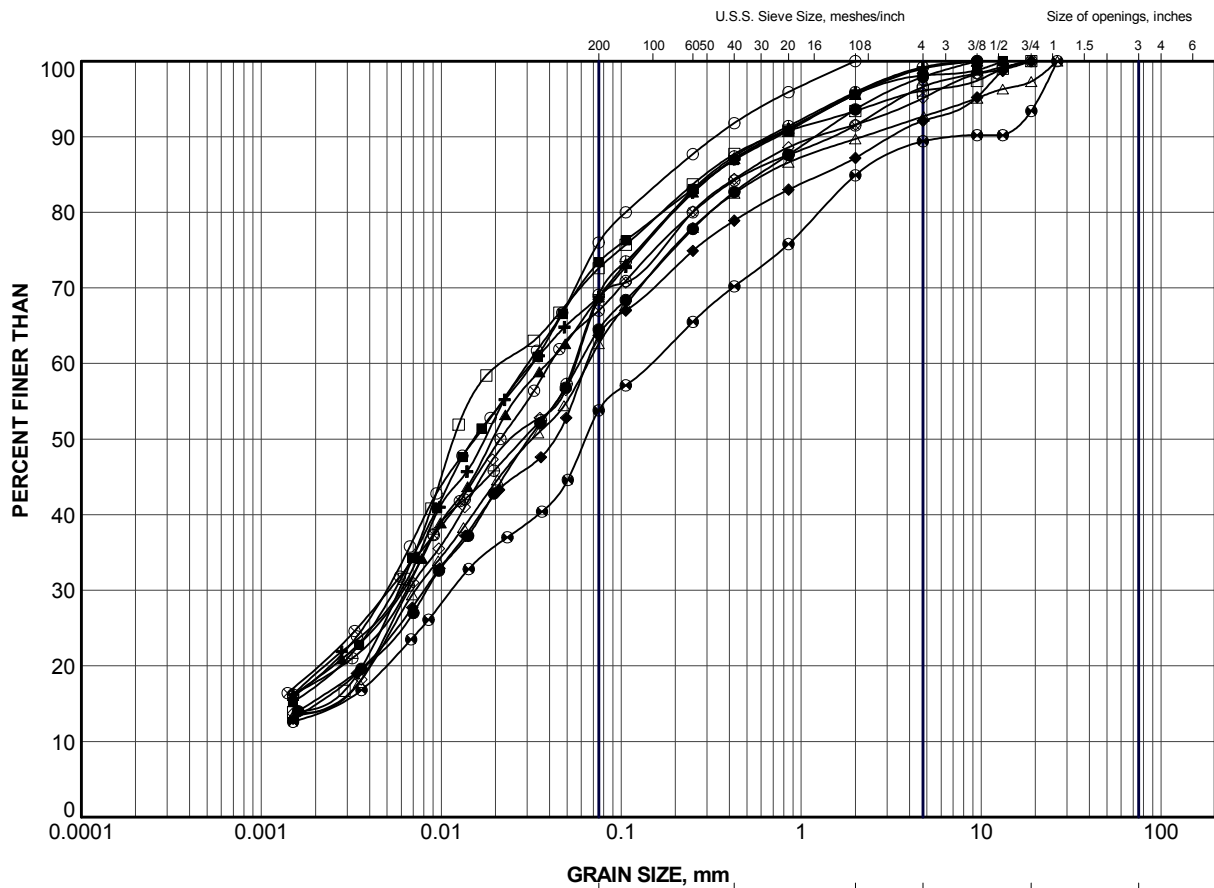


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	N12	5	205.9

PROJECT					HIGHWAY 11 NAGAGAMI RIVER BRIDGE				
TITLE					GRAIN SIZE DISTRIBUTION SAND and GRAVEL (UPPER DEPOSIT)				
PROJECT No.		11-1191-0008		FILE No.		1191-0008 DETAIL.GPJ			
DRAWN	JJL	Jan 2014		SCALE	N/A	REV.			
CHECK	AB	Jan 2014		FIGURE A4					
APPR	JMAC	Jan 2014							
 Golder Associates SUDBURY, ONTARIO									



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	N1	5	208.7
■	N1	9	204.0
▲	N3	5	203.4
+	N3	8	200.6
◆	N5	4	209.3
◇	N5	6	207.7
○	N6	3	209.3
△	N6	6	207.1
⊗	N12	7	204.5
⊕	N12	9	201.5
□	N13	8	207.0
⊗	N14	8	205.9

PROJECT

HIGHWAY 11
NAGAGAMI RIVER BRIDGE

TITLE

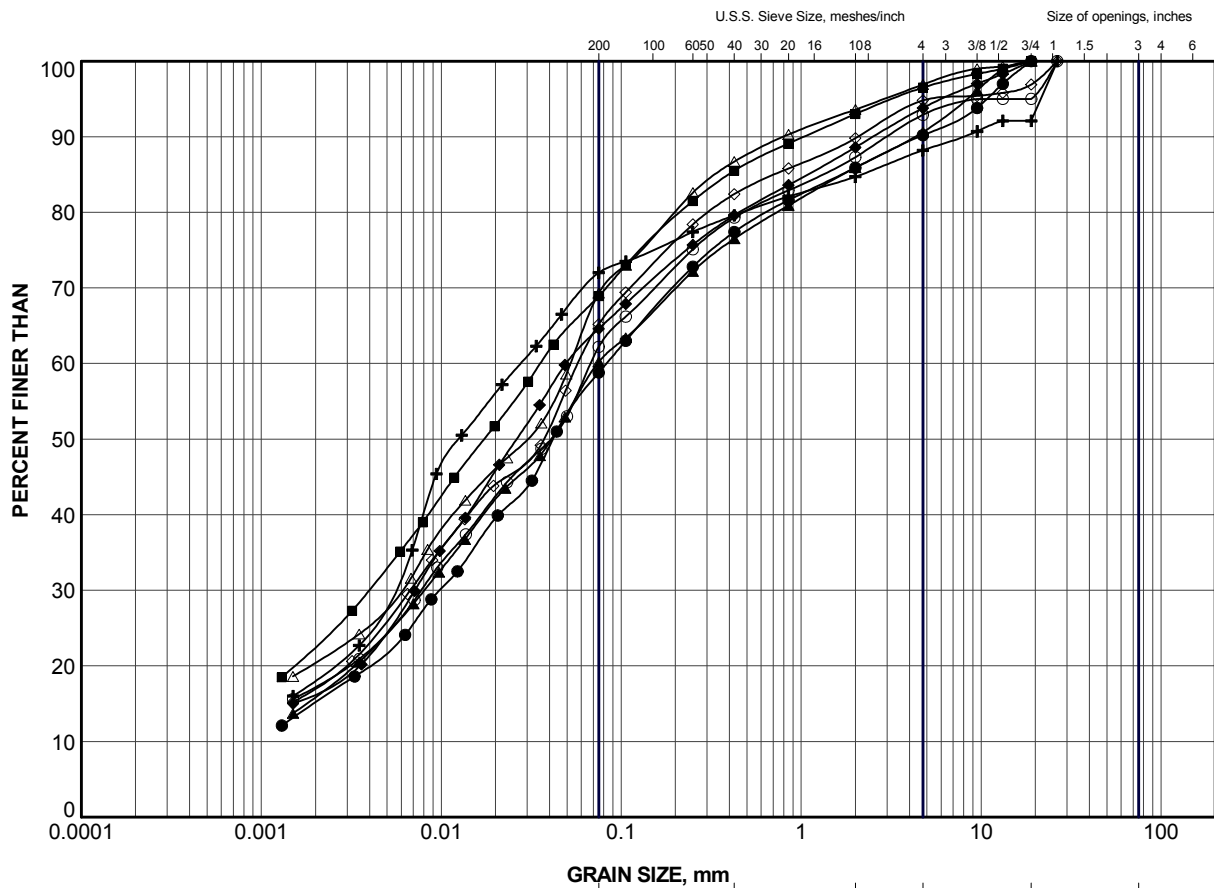
GRAIN SIZE DISTRIBUTION
SANDY SILT to SILT and SAND (TILL), UPPER DEPOSIT



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SUDBURY, ONTARIO

PROJECT No.	11-1191-0008	FILE No.	11-1191-0008 DETAIL.GPJ
DRAWN	JJL	Jul 2014	SCALE N/A
CHECK	AB	Jul 2014	REV.
APPR	JMAC	Jul 2014	

FIGURE A5.1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	N1	4	184.3
■	N1	7	175.1
▲	N15	2	210.2
+	N15	4	208.7
◆	N15	7	206.5
◇	N16	6	204.1
○	N16	8	201.8
△	N17	12	202.9

PROJECT

HIGHWAY 11
NAGAGAMI RIVER BRIDGE

TITLE

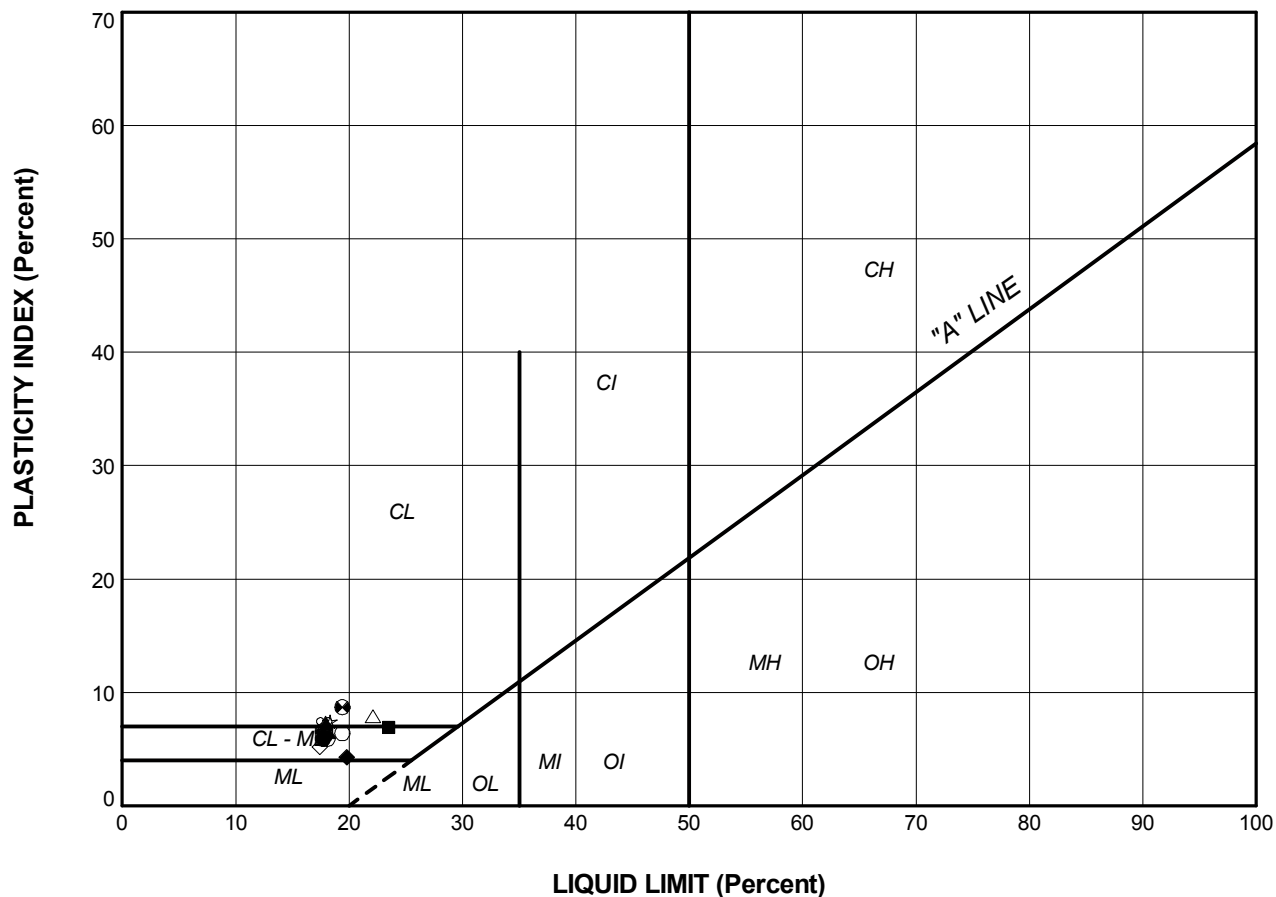
GRAIN SIZE DISTRIBUTION
SANDY SILT to SILT and SAND (TILL), UPPER DEPOSIT



Golder Associates
SUDBURY, ONTARIO

PROJECT No. 11-1191-0008		FILE No. 1191-0008 DETAIL.GPJ	
DRAWN	JJL	Jul 2014	SCALE N/A
CHECK	AB	Jul 2014	REV.
APPR	JMAC	Jul 2014	

FIGURE A5.2

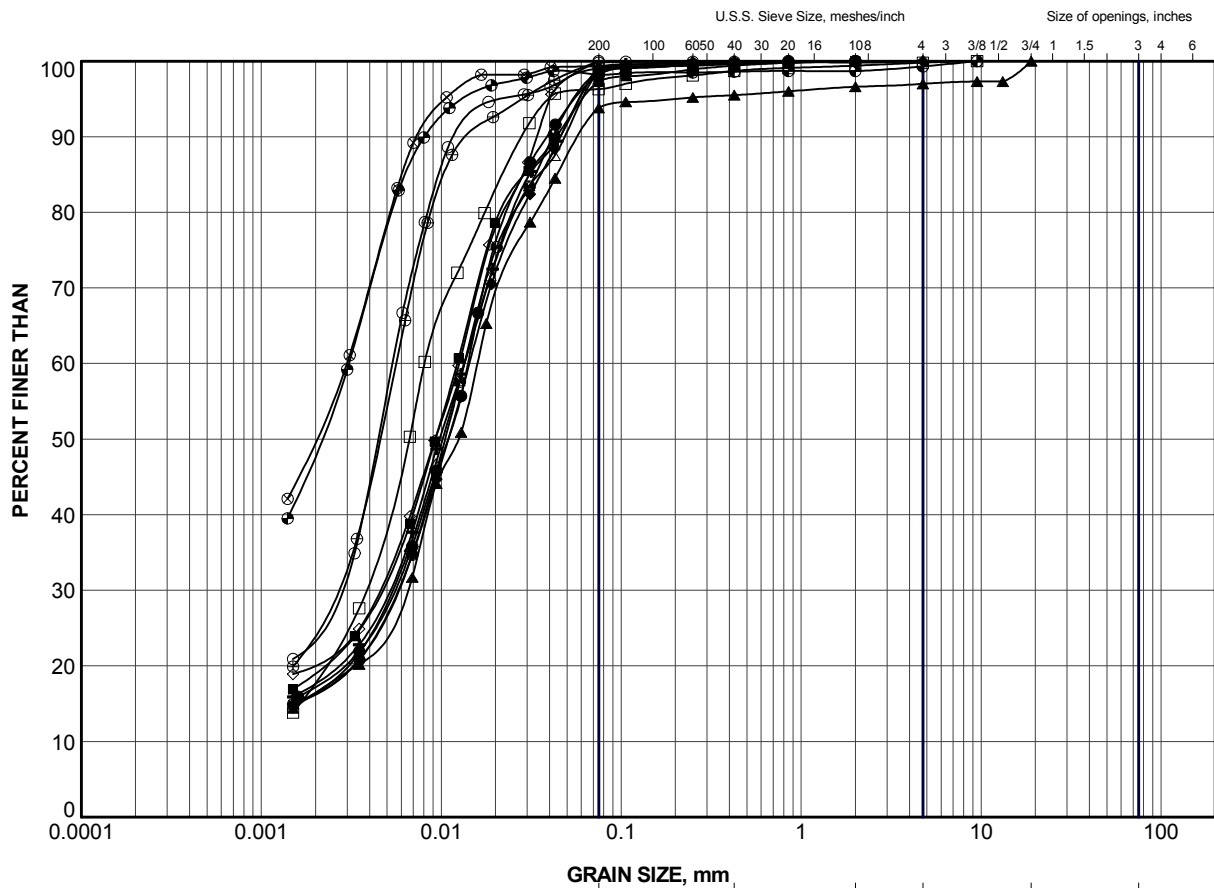


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	N1	6	17.7	11.8	5.9
■	N1	10	23.5	16.6	6.9
▲	N12	7	17.9	10.6	7.3
+	N12	9	17.7	11.2	6.5
◆	N13	8	19.8	15.5	4.3
◇	N14	8	17.4	12.2	5.2
○	N15	2	19.4	13.0	6.4
△	N15	4	22.1	14.2	7.9
⊗	N15	7	17.9	11.6	6.3
⊕	N16	6	17.9	11.2	6.7
□	N16	8	17.7	11.3	6.4
⊗	N17	12	19.4	10.7	8.7
⊕	N5	4	18.1	12.2	5.9
☆	N5	6	18.3	10.9	7.4
⊗	N6	6	17.8	10.7	7.1

PROJECT				
HIGHWAY 11 NAGAGAMI RIVER BRIDGE				
TITLE				
PLASTICITY CHART				
SANDY SILT to SILT and SAND (TILL), UPPER DEPOSIT				
PROJECT No. 11-1191-0008		FILE No. 11-1191-0008 DETAIL.GPJ		
DRAWN	JJL	Jul 2014	SCALE	N/A
CHECK	AB	Jul 2014	REV.	
APPR	JMAC	Jul 2014	FIGURE A6	





LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	N1	12	199.4
■	N6	10	201.8
▲	N6	13	197.2
+	N6	16	192.6
◆	N7	2	202.6
◇	N7	5	200.3
○	N7	7	198.8
△	N8	3	202.6
⊗	N8	5	201.1
⊕	N8	7	199.6
□	N8	9	197.3
⊙	N9	2	203.6
⊗	N9	6	200.5

PROJECT

HIGHWAY 11
NAGAGAMI RIVER BRIDGE

TITLE

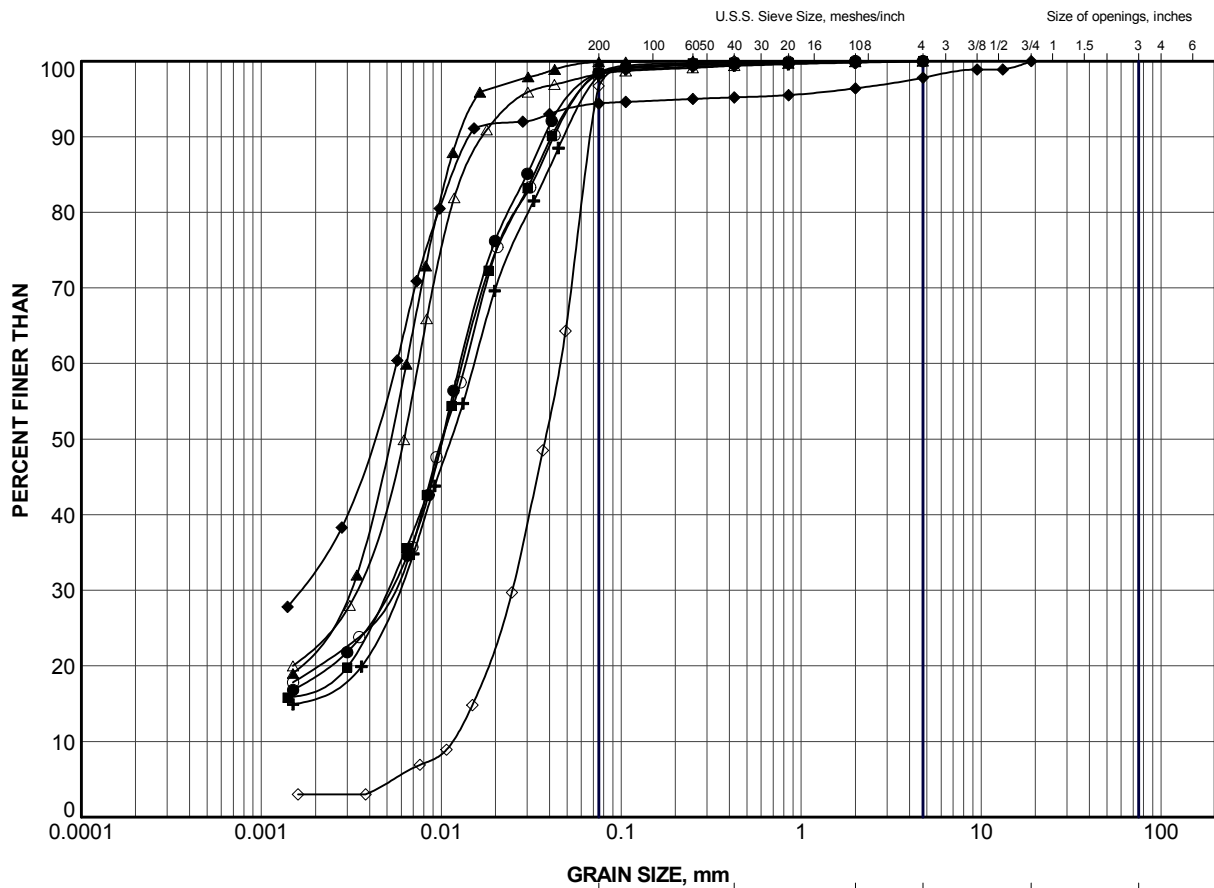
GRAIN SIZE DISTRIBUTION
SILT



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Associates**
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PROJECT No.	11-1191-0008	FILE No.	11-1191-0008 DETAIL.GPJ
DRAWN	JJL	Jul 2014	SCALE N/A
CHECK	AB	Jul 2014	REV.
APPR	JMAC	Jul 2014	

FIGURE A7.1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	N10	2	203.3
■	N10	5	201.0
▲	N10	8	198.7
+	N11	2	203.3
◆	N11	5	201.0
◇	N14	6	208.1
○	N15	11	200.3
△	N16	4a	205.7

PROJECT

HIGHWAY 11
NAGAGAMI RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

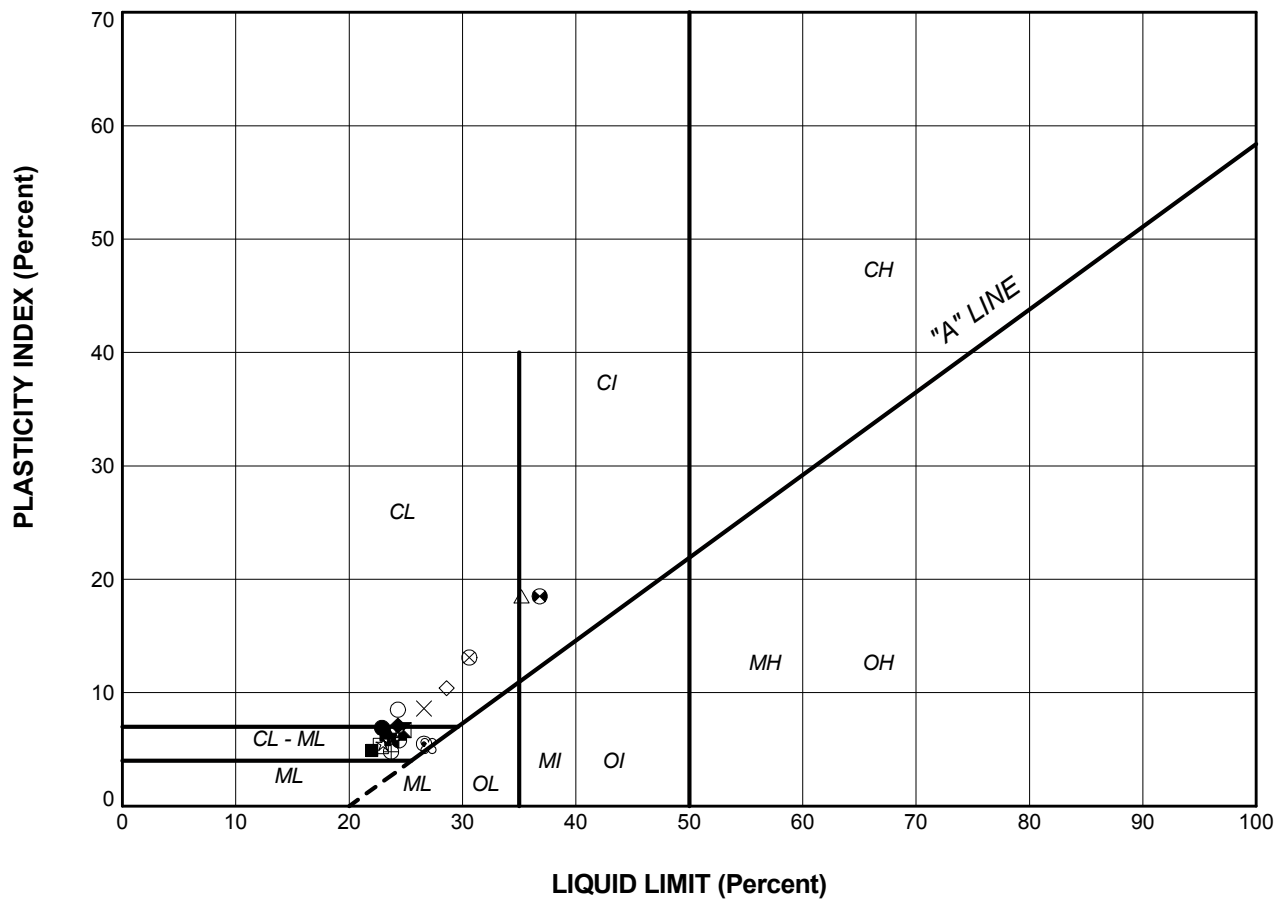
SILT



Golder Associates
SUDBURY, ONTARIO


PROJECT No. 11-1191-0008		FILE No. 1191-0008 DETAIL.GPJ	
DRAWN	JJL	Jul 2014	SCALE N/A
CHECK	AB	Jul 2014	REV.
APPR	JMAC	Jul 2014	

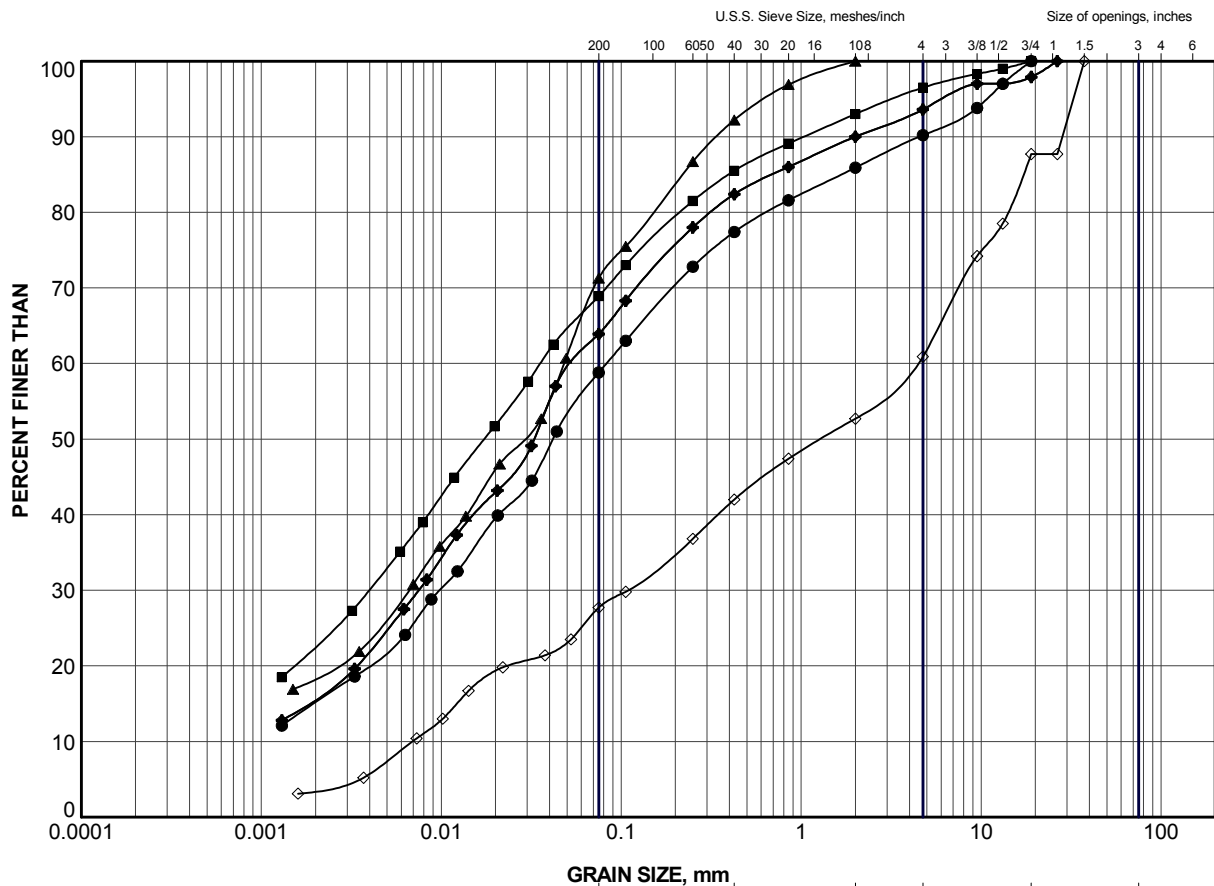
FIGURE A7.2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	N6	10	22.9	16.0	6.9
■	N6	13	22.0	17.1	4.9
▲	N6	16	23.4	16.9	6.5
+	N7	2	23.5	17.2	6.3
◆	N7	5	24.3	17.2	7.1
◇	N7	7	28.6	18.2	10.4
○	N8	3	24.3	15.8	8.5
△	N8	5	35.2	16.7	18.5
⊗	N8	7	30.6	17.5	13.1
⊕	N8	9	23.7	18.9	4.8
□	N9	2	22.8	17.5	5.3
⊗	N9	6	36.8	18.3	18.5
⊕	N10	2	24.4	18.6	5.8
☆	N10	5	23.0	17.5	5.5
⊗	N10	8	27.0	21.7	5.3
⊕	N11	2	24.8	18.1	6.7
⊗	N11	5	26.6	21.1	5.5
⊕	N15	11	23.4	17.4	6.0
×	N16	4a	26.6	18.0	8.6


PROJECT						HIGHWAY 11 NAGAGAMI RIVER BRIDGE					
TITLE						PLASTICITY CHART SILT, SOME CLAY					
PROJECT No. 11-1191-0008			FILE No. 11-1191-0008 DETAIL.GPJ								
DRAWN	JJL	Jul 2014	SCALE	N/A	REV.						
CHECK	AB	Jul 2014									
APPR	JMAC	Jul 2014									
 Golder Associates SUDBURY, ONTARIO			FIGURE A8								

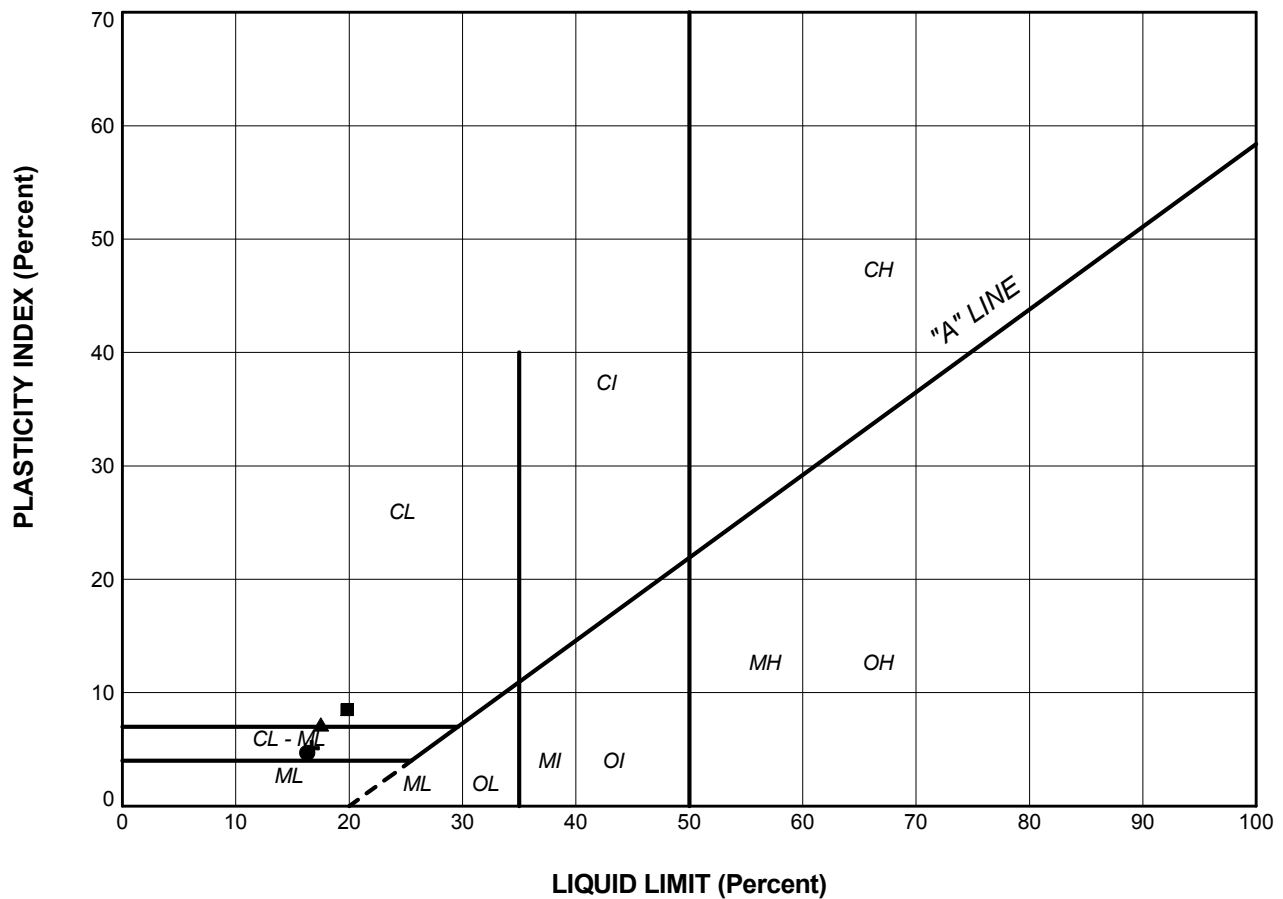


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	N1a	4	184.3
■	N1a	7	175.1
▲	N7	10	194.3
+	N8	2	203.4
◆	N8a	2	193.8
◇	N9	9b	197.3

PROJECT						HIGHWAY 11 NAGAGAMI RIVER BRIDGE					
TITLE						GRAIN SIZE DISTRIBUTION SANDY SILT to SILTY SAND and GRAVEL (TILL)					
PROJECT No.						11-1191-0008			FILE No. 1191-0008 DETAIL.GPJ		
DRAWN		JUL		JUL 2014		SCALE		N/A		REV.	
CHECK		AB		JUL 2014							
APPR		JMAC		JUL 2014							
 Golder Associates SUDBURY, ONTARIO						FIGURE A9					



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	N1a	4	16.3	11.6	4.7
■	N1a	7	19.8	11.3	8.5
▲	N7	10	17.5	10.3	7.2
+	N8a	2	16.7	11.6	5.1

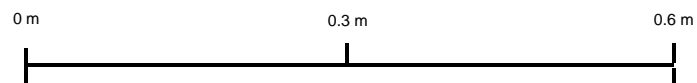
PROJECT						HIGHWAY 11 NAGAGAMI RIVER BRIDGE					
TITLE						PLASTICITY CHART SILT and SAND (TILL)					
PROJECT No. 11-1191-0008			FILE No. 11-1191-0008 DETAIL.GPJ								
DRAWN	JJL	Jul 2014	SCALE	N/A	REV.						
CHECK	AB	Jul 2014									
APPR	JMAC	Jul 2014									
 Golder Associates SUDBURY, ONTARIO			FIGURE A10								




Borehole N1a
Elevation 171.7 m to 168.6 m



Borehole N8a
Elevation 193.2 m to 190.1 m



PROJECT		HIGHWAY 11 NAGAGAMI RIVER BRIDGE			
TITLE		ROCK CORE PHOTOGRAPHS			
	PROJECT No. 11-1191-0008		FILE No. ----		
	DESIGN	AC	NOV 2013	SCALE AS SHOWN	
	CADD	--		REV.	
	CHECK	AB	NOV 2013	FIGURE A11a	
	REVIEW				



Borehole N9
Elevation 195.9 m to 192.7 m



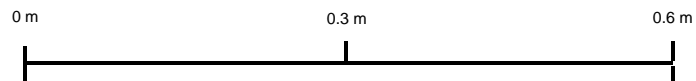
Borehole N10
Elevation 197.0 m to 194.1 m




Borehole N11
Elevation 199.1 m to 197.1 m



Borehole N12
Elevation 198.9 m to 197.7 m

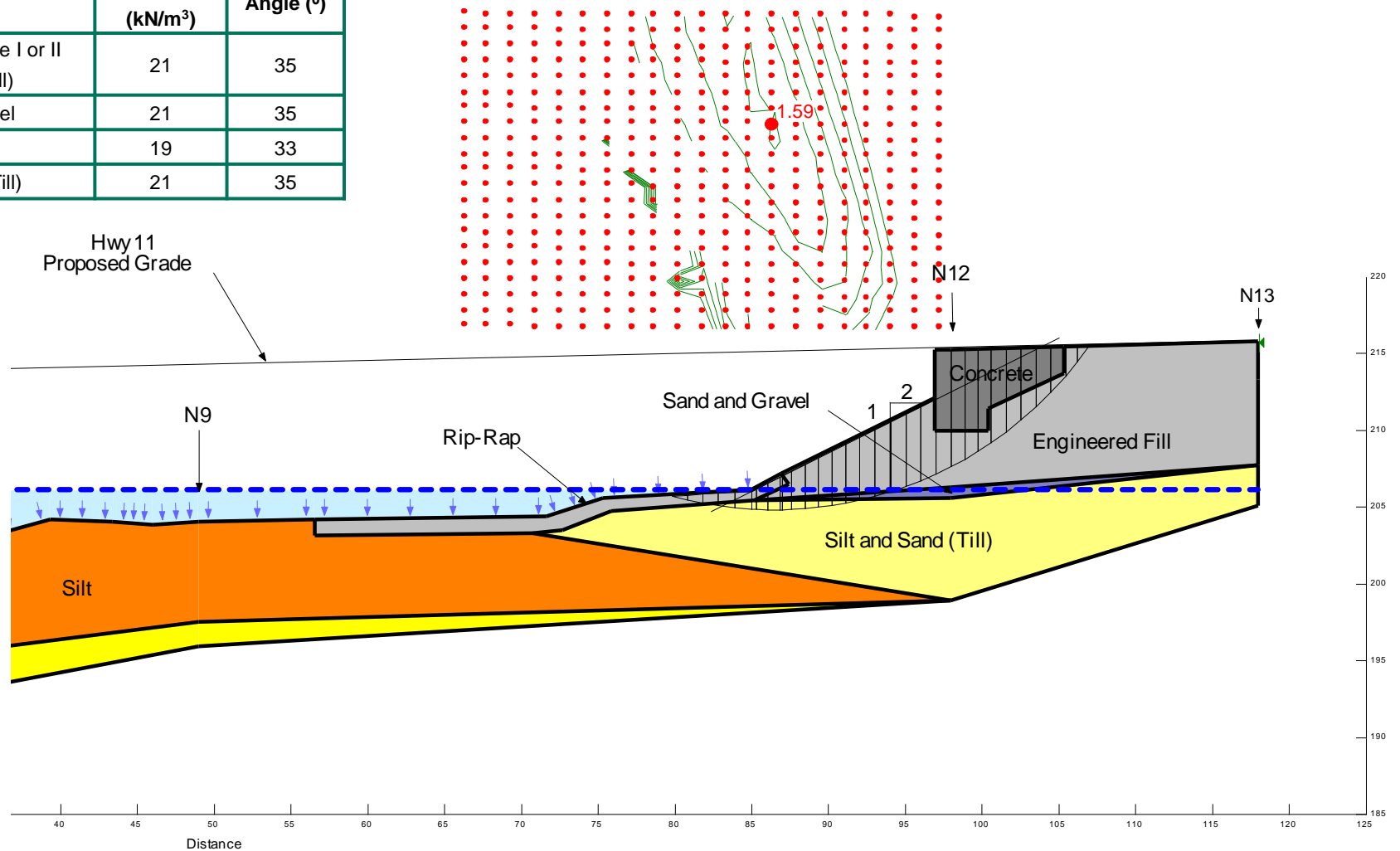


PROJECT			HIGHWAY 11 NAGAGAMI RIVER BRIDGE	
TITLE			ROCK CORE PHOTOGRAPHS	
	PROJECT No.	11-1191-0008	FILE No.	----
	DESIGN	AC	NOV 2013	SCALE AS SHOWN
	CADD	--		REV.
	CHECK	AB	NOV 2013	FIGURE A11b
	REVIEW			

Highway 11 Nagagami River Bridge Stability Analysis – East Approach Front Slope

Figure A12

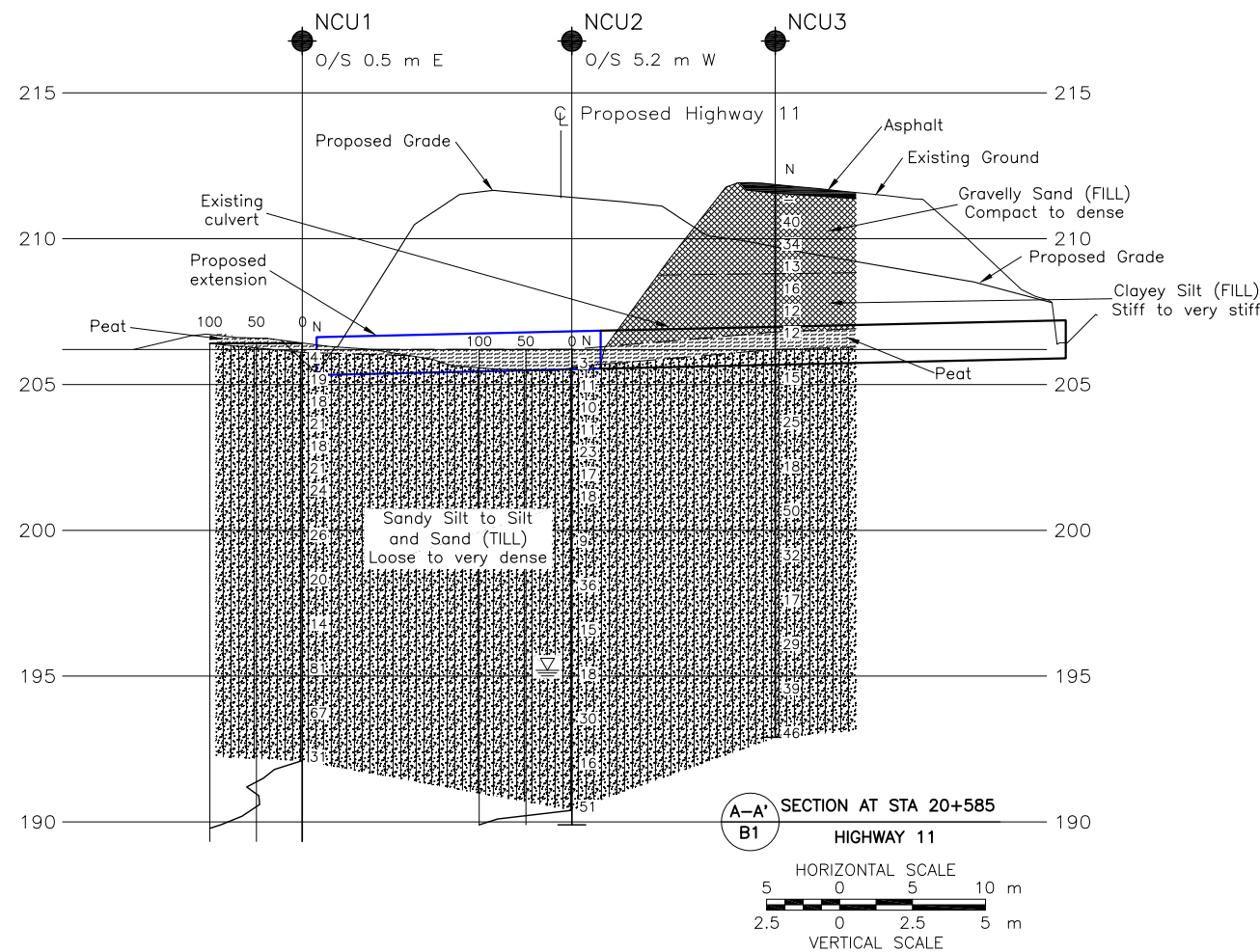
Material Name	Unit Weight (kN/m ³)	Friction Angle (°)
New Granular B Type I or II (Engineered Fill)	21	35
Sand and Gravel	21	35
Silt	19	33
Silt and Sand (Till)	21	35





APPENDIX B

Culvert Replacement – STA 20+585



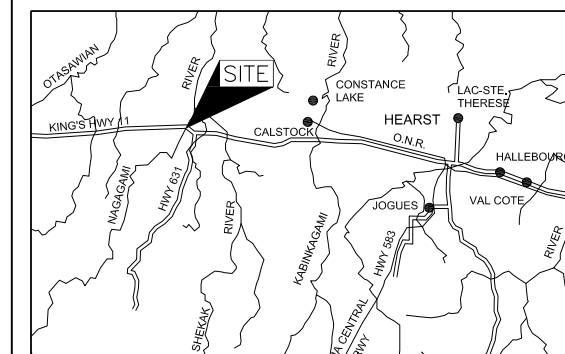
CONT No.
GWP No. 5307-04-00



HIGHWAY 11
NAGAGAMI RIVER BRIDGE
CULVERT EXTENSION - STA 20+585
BOREHOLE LOCATIONS AND SOIL STRATA

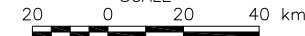


Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA





KEY PLAN

SCALE



LEGEND

- | | |
|---|--|
|  | Borehole |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
|  | WL upon completion of drilling |

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
NCU1	206.4	5515358.2	265872.6
NCU2	206.2	5515348.0	265856.2
NCU3	211.8	5515334.1	265850.9
NCU4	211.9	5515329.1	265859.9

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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by LEA, drawing file nos. 8960-Nag-S01.dwg received on May 28, 2013 and x8960 Nagagami Base.dwg, received Sept 25, 2013. Culvert profile provided by LEA, drawing file no. x8960 Naga Prof.dwg, received Nov 26, 2013.

NO.	DATE	BY	REVISION
Geocres No. 42F-28			
HWY. 11		PROJECT NO. 11-1191-000B	DIST.
SUBM'D. AC	CHKD.	DATE: SEP 2014	SITE:
DRAWN: TB	CHKD. AB	APPD. JMAC	DWG. B1

PROJECT 11-1191-0008		RECORD OF BOREHOLE No NCU1		1 OF 2 METRIC																				
G.W.P. 5307-04-00		LOCATION N 5515358.2; E 265872.6		ORIGINATED BY SA																				
DIST _____ HWY 11		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AC																				
DATUM GEODETIC		DATE July 20, 2013		CHECKED BY AB																				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																			
206.4	GROUND SURFACE																							
0.0	PEAT (Fibrous)		1	SS	4																			
206.1	Brown																							
0.3	Moist																							
	Sandy SILT to SILT and SAND, some clay, trace to some gravel (TILL)		2	SS	19																			
	Compact to very dense																							
	Brown, grey below 1.5 m depth		3	SS	18																			
	Moist to wet																							
			4	SS	21																			
			5	SS	18																			
			6	SS	21																			
			7	SS	24																			
			8	SS	26																			
			9	SS	20																			
			10	SS	14																			
			11	SS	81																			
			12	SS	67																			
			13	SS	31																			
192.1	END OF BOREHOLE																							
14.3	START OF DCPT																							

SUD-MTO 001 11-1191-0008 DETAIL.GPJ CAL-MISS.GDT 24/04/14 DATA INPUT:

Continued Next Page


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

PROJECT <u>11-1191-0008</u>		RECORD OF BOREHOLE No NCU1		2 OF 2 METRIC	
G.W.P. <u>5307-04-00</u>		LOCATION <u>N 5515358.2; E 265872.6</u>		ORIGINATED BY <u>SA</u>	
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>		DATE <u>July 20, 2013</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			W _p	W	W _L		WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED											
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100				20 40 60			GR SA SI CL				
	END OF BOREHOLE START OF DCPT						191												
							190												
189.0																			
17.4	END OF DCPT						189												
	Note: 1. Borehole dry upon completion of drilling.																		

SUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 24/04/14 DATA INPUT:

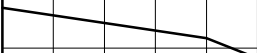
PROJECT 11-1191-0008		RECORD OF BOREHOLE No NCU2		1 OF 2 METRIC	
G.W.P. 5307-04-00		LOCATION N 5515348.0; E 265856.2		ORIGINATED BY SA	
DIST _____ HWY 11		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AC	
DATUM GEODETIC		DATE July 21, 2013		CHECKED BY AB	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
206.2	GROUND SURFACE																			
0.0	PEAT (Fibrous), some sand			1	SS	3														
205.6	Soft																			
0.6	Brown																			
	Moist																			
	SILT and SAND, some clay, trace to																			
	some gravel (TILL)																			
	Loose to compact																			
	Grey																			
	Moist to wet																			

Continued Next Page

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

SUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 24/04/14 DATA INPUT:


PROJECT <u>11-1191-0008</u>		RECORD OF BOREHOLE No NCU2				2 OF 2 METRIC											
G.W.P. <u>5307-04-00</u>		LOCATION <u>N 5515348.0; E 265856.2</u>				ORIGINATED BY <u>SA</u>											
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>AC</u>											
DATUM <u>GEODETIC</u>		DATE <u>July 21, 2013</u>				CHECKED BY <u>AB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>					
190.4		14	SS	51		191						<div style="display: flex; justify-content: space-between;"> 20 40 60 </div>					
15.8	END OF BOREHOLE START OF DCPT					190											
189.9																	
16.3	END OF DCPT Note: 1. Water level at a depth of 11.0 m below ground surface (Elev. 195.2 m) upon completion of drilling.																

PROJECT 11-1191-0008			RECORD OF BOREHOLE No NCU3			1 OF 2 METRIC											
G.W.P. 5307-04-00			LOCATION N 5515334.1; E 265850.9			ORIGINATED BY SA											
DIST _____ HWY 11			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY AC											
DATUM GEODETIC			DATE July 18, 2013			CHECKED BY AB											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED								WATER CONTENT (%)	
211.8	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALT (300 mm)		1	AS	-		211										
0.3	Gravelly sand, some silt, trace clay (FILL) Compact to dense Brown to grey Moist		2	SS	40		210									28 59 10 3	
			3	SS	34												
			4	SS	13		209										
208.8	Clayey silt, trace to some sand, trace gravel (FILL) Stiff to very stiff Brown Moist		5	SS	16		208										
3.0			6	SS	12		207										
206.8	PEAT (Fibrous) Brown Moist		7	SS	12		206										
5.0							205										
206.2	Sandy SILT to SILT and SAND, some clay, trace to some gravel (TILL) Compact to very dense Grey Moist to wet		8	SS	15		204									10 31 46 13	
5.6			9	SS	25		203										
			10	SS	18		202										
			11	SS	50		201									10 29 50 11	
			12	SS	32		200										
							199										
			13	SS	17		198									11 31 44 14	
							197										

Continued Next Page

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

SUD-MTO 001 11-1191-0008 DETAIL.GPJ CAL-MISS.GDT 24/04/14 DATA INPUT:

PROJECT <u>11-1191-0008</u>			RECORD OF BOREHOLE No NCU3			2 OF 2 METRIC											
G.W.P. <u>5307-04-00</u>			LOCATION <u>N 5515334.1; E 265850.9</u>			ORIGINATED BY <u>SA</u>											
DIST <u> </u> HWY <u>11</u>			BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers</u>			COMPILED BY <u>AC</u>											
DATUM <u>GEODETIC</u>			DATE <u>July 18, 2013</u>			CHECKED BY <u>AB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 20 40 60					
192.9	Sandy SILT to SILT and SAND, some clay, trace to some gravel (TILL) Compact to very dense Grey Moist to wet		14	SS	29												
			15	SS	39												
			16	SS	46												
18.9	END OF BOREHOLE																
	Note: 1. Borehole dry upon completion of drilling.																

PROJECT 11-1191-0008			RECORD OF BOREHOLE No NCU4			1 OF 2 METRIC															
G.W.P. 5307-04-00			LOCATION N 5515329.1; E 265859.9			ORIGINATED BY EHS															
DIST _____ HWY 11			BOREHOLE TYPE NW Casing and Wash Boring			COMPILED BY AC															
DATUM GEODETIC			DATE April 3 and 7, 2014			CHECKED BY AB															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
211.9	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	20 40 60	W _p W W _L										
0.0	Gravelly sand to sand and gravel, trace to some silt (FILL) Brown Frozen		1	SS	42		211														
			2	SS	75		210													23 62 (15)	
209.3	Clayey silt, trace to some sand, trace to some gravel (FILL) Firm to stiff Brown Moist to wet		3	SS	58		209														
2.6			4	SS	13		208													6 14 54 26	
			5	SS	6		207														
207.0	PEAT (Amorphous) Black		6	SS	7		206														
5.0	SILT and SAND, trace to some gravel, trace to some clay (TILL) Compact to very dense Brown to grey Moist		7	SS	18		205													8 31 45 16	
			8	SS	21		204														
			9	SS	20		203														
			10	SS	24		202														
			11	SS	92		201													8 33 43 15	
			12	SS	74		200														
							199														
							198														
							197														

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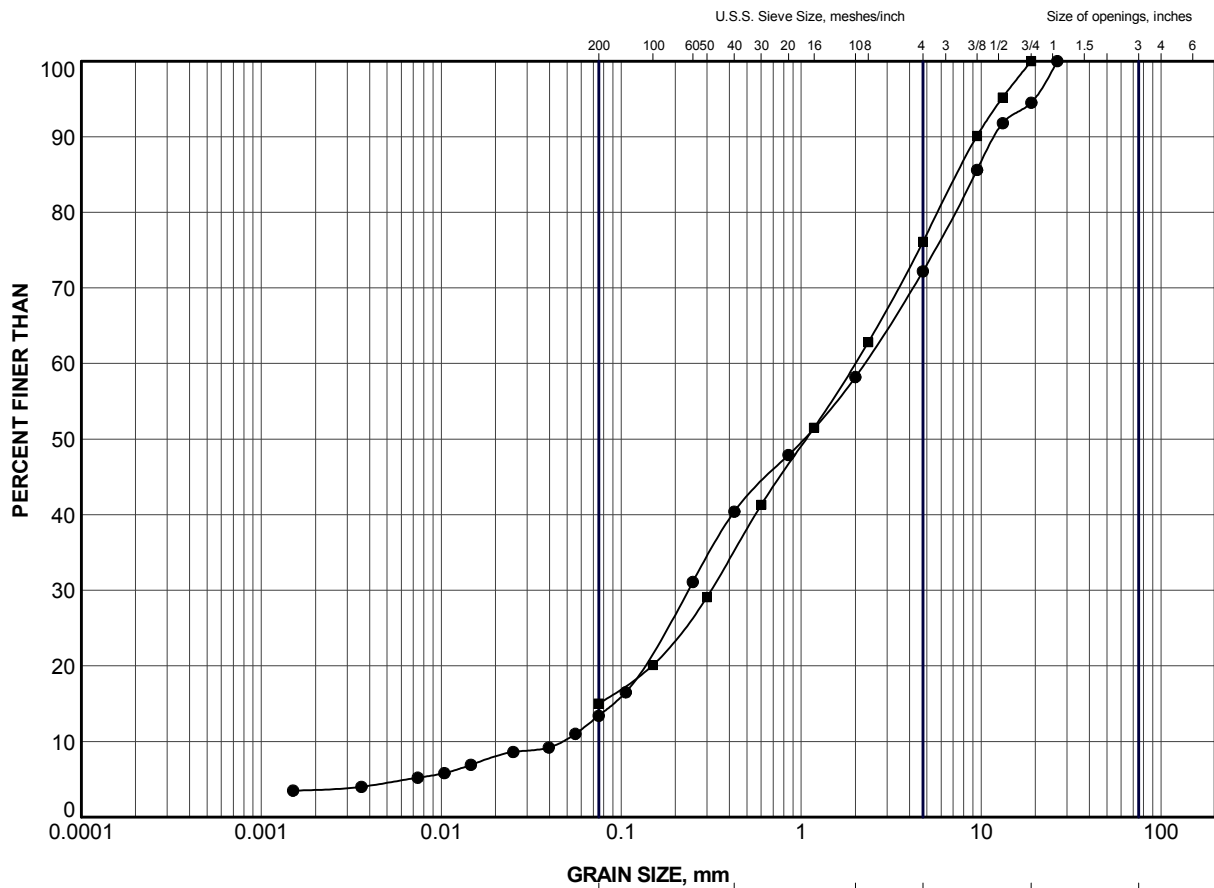
+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

SUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 24/04/14 DATA INPUT:



PROJECT		RECORD OF BOREHOLE No NCU4				2 OF 2 METRIC										
G.W.P. 5307-04-00		LOCATION N 5515329.1; E 265859.9				ORIGINATED BY EHS										
DIST _____ HWY 11		BOREHOLE TYPE NW Casing and Wash Boring				COMPILED BY AC										
DATUM GEODETIC		DATE April 3 and 7, 2014				CHECKED BY AB										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
196.1			13	SS	77											
15.8	END OF BOREHOLE															
	Note: 1. Borehole dry upon completion of drilling.															


SUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 24/04/14 DATA INPUT:



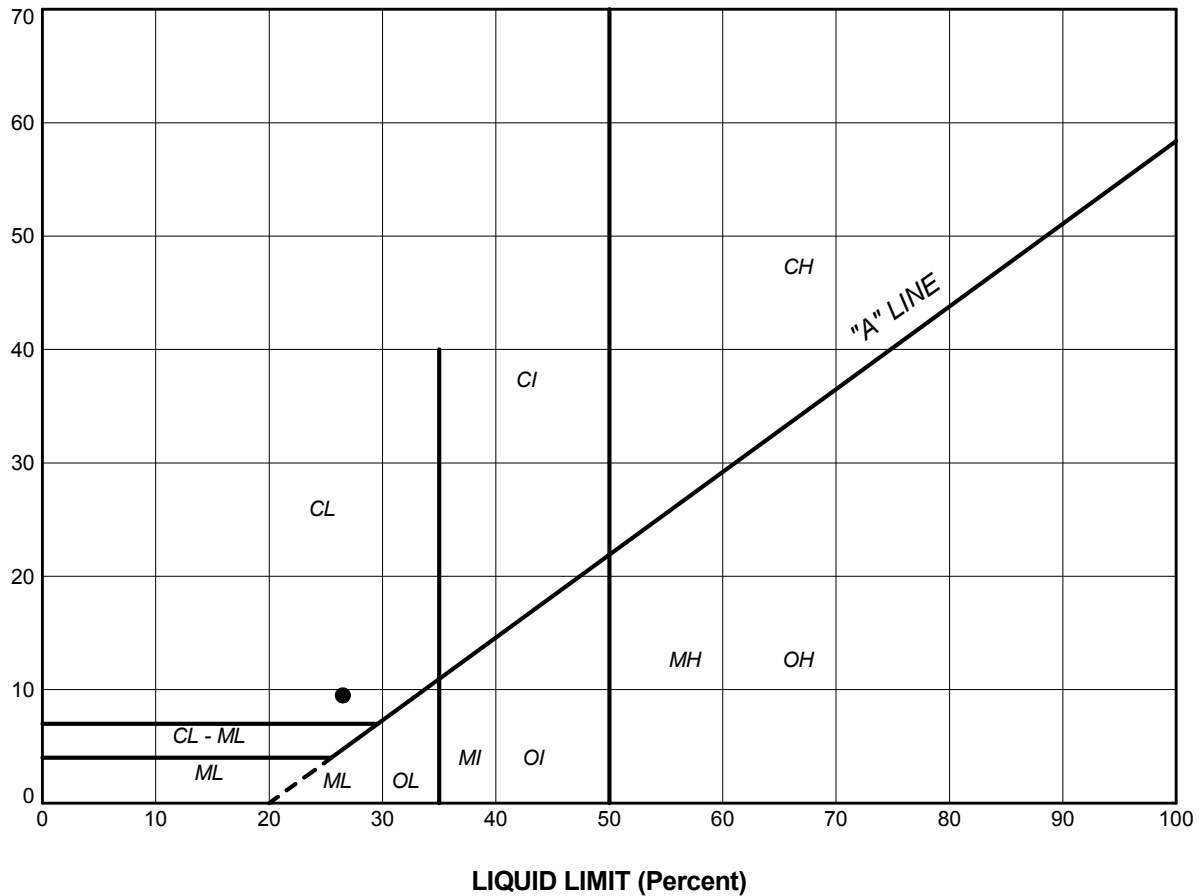
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NCU3	3	210.0
■	NCU4	2	210.1

PROJECT						HIGHWAY 11 NAGAGAMI RIVER BRIDGE CULVERT EXTENSION - STA 20+585					
TITLE						GRAIN SIZE DISTRIBUTION GRAVELLY SAND (FILL)					
PROJECT No.			11-1191-0008			FILE No.			1191-0008 DETAIL.GPJ		
DRAWN	TB	Apr 2014	SCALE	N/A	REV.						
CHECK	AB	Apr 2014									
APPR	JMAC	Apr 2014									
 Golder Associates SUDBURY, ONTARIO						FIGURE B1					

PLASTICITY INDEX (Percent)




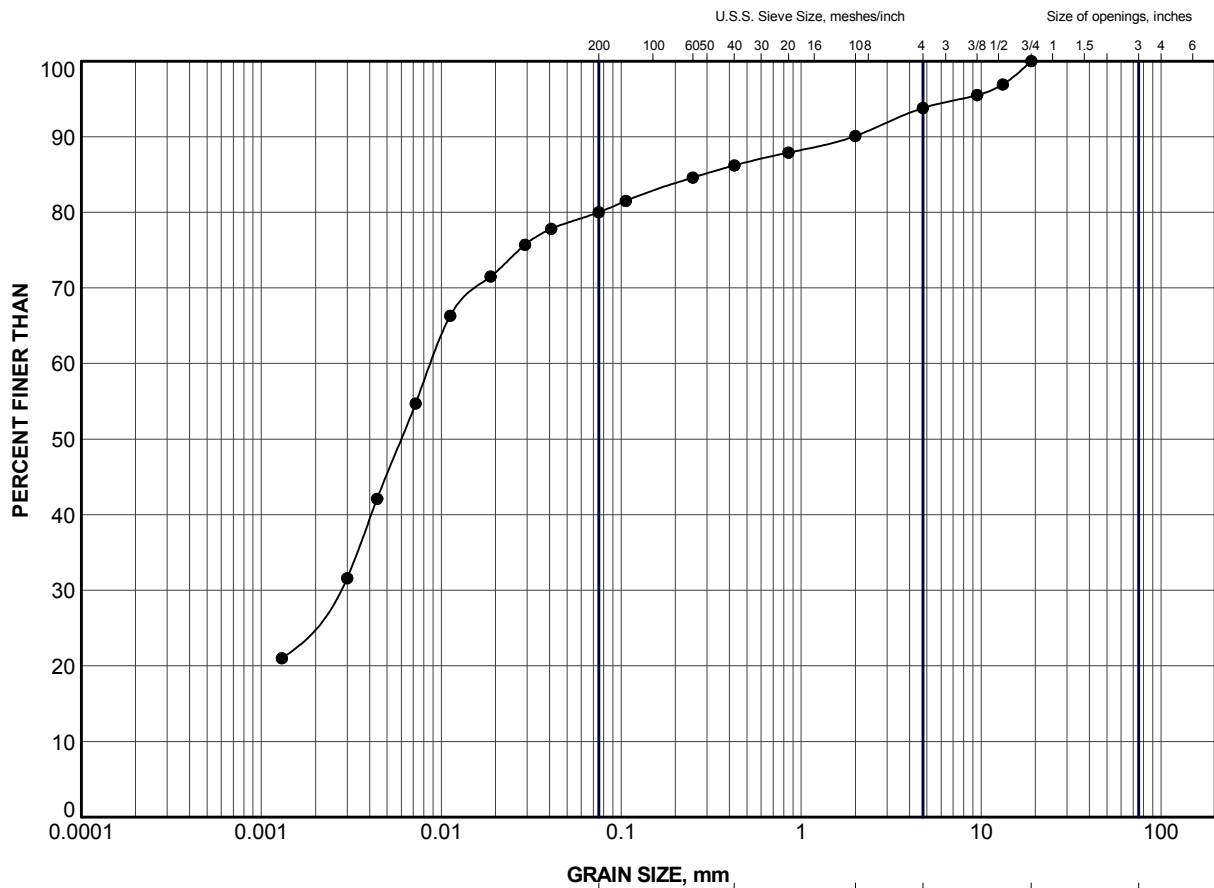
SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	NCU4	4	26.5	17.0	9.5


PROJECT			HIGHWAY 11 NAGAGAMI RIVER BRIDGE CULVERT EXTENSION - STA 20+585		
TITLE			PLASTICITY CHART CLAYEY SILT (FILL)		
PROJECT No.		11-1191-0008	FILE No. 11-1191-0008 DETAIL.GPJ		
DRAWN	TB	Sep 2014	SCALE	N/A	REV.
CHECK	AB	Sep 2014			
APPR	JMAC	Sep 2014			
 Golder Associates SUDBURY, ONTARIO			FIGURE B2		

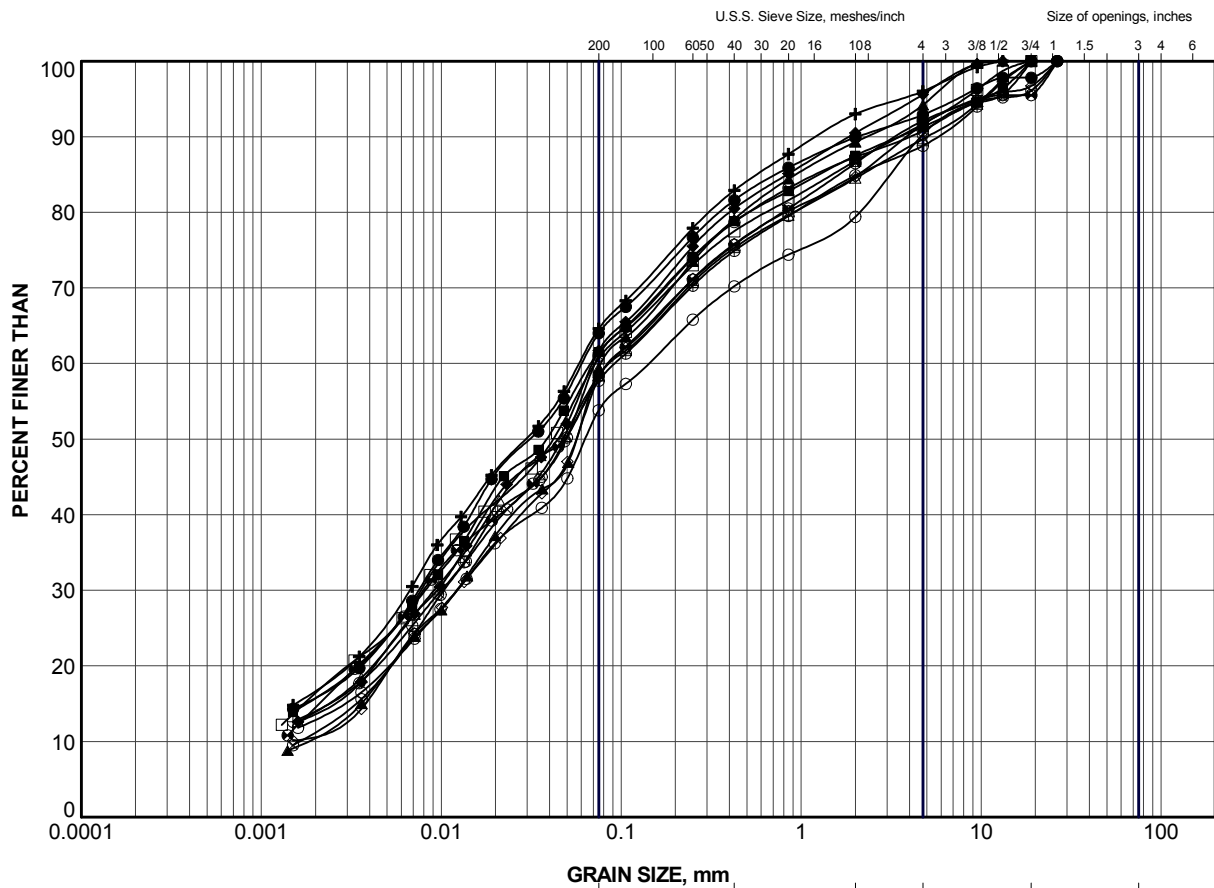


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NCU4	4	208.6

PROJECT					HIGHWAY 11 NAGAGAMI RIVER BRIDGE CULVERT EXTENSION - STA 20+585				
TITLE					GRAIN SIZE DISTRIBUTION CLAYEY SILT (FILL)				
PROJECT No.		11-1191-0008		FILE No.		1191-0008 DETAIL.GPJ			
DRAWN	TB	Apr 2014		SCALE	N/A	REV.			
CHECK	AB	Apr 2014		FIGURE B3					
APPR	JMAC	Apr 2014							
 Golder Associates SUDBURY, ONTARIO									



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NCU1	2	205.3
■	NCU1	6	202.3
▲	NCU1	11	195.4
+	NCU2	3	204.4
◆	NCU2	6	202.1
◇	NCU2	9	198.3
○	NCU2	12	193.7
△	NCU3	9	203.9
⊗	NCU3	11	200.8
⊕	NCU3	13	197.8
□	NCU4	7	205.5
⊙	NCU4	10	200.9

PROJECT

HIGHWAY 11
NAGAGAMI RIVER BRIDGE
CULVERT EXTENSION - STA 20+585

TITLE

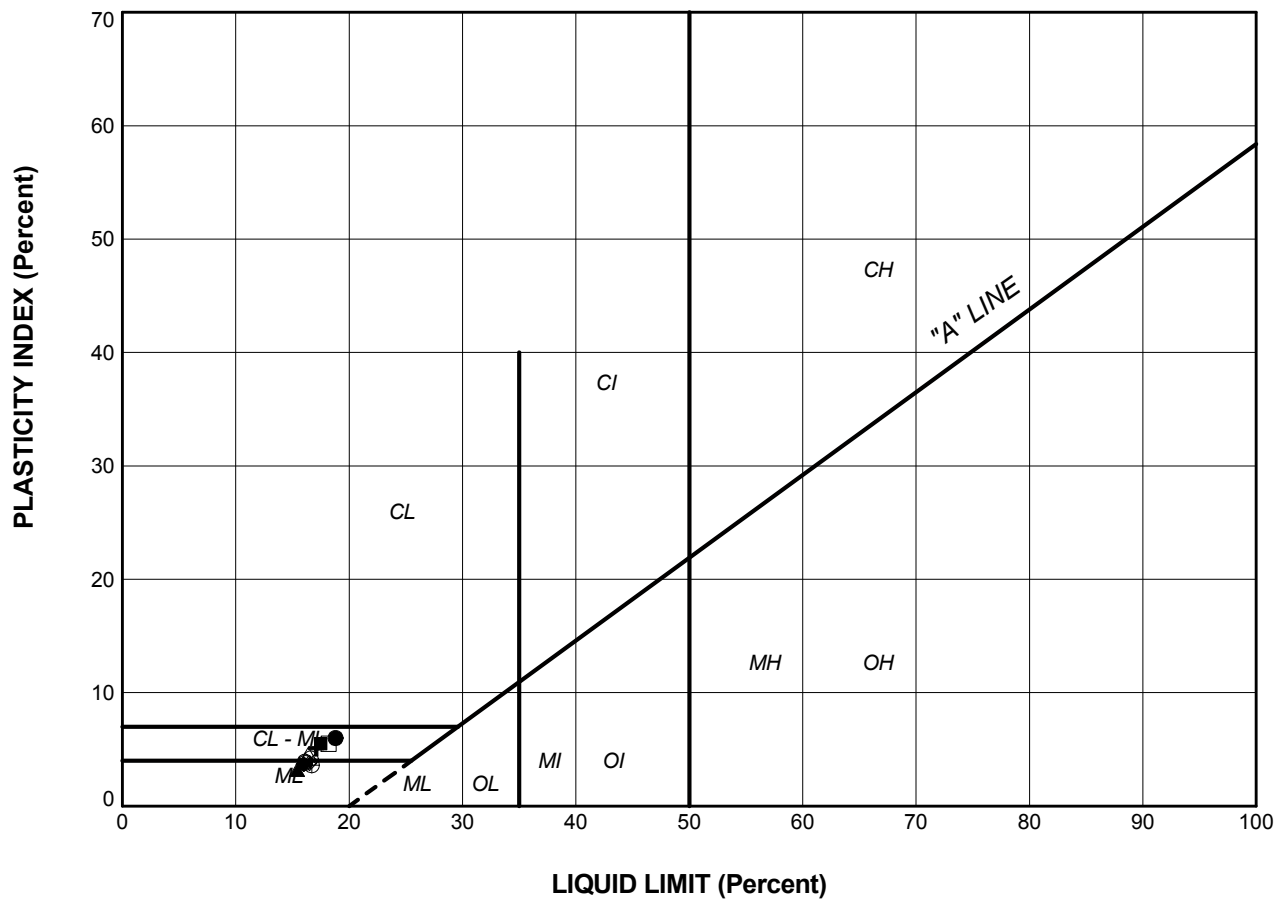
GRAIN SIZE DISTRIBUTION
SANDY SILT to SILT and SAND (TILL)



Golder Associates
SUDBURY, ONTARIO


PROJECT No. 11-1191-0008		FILE No. 1191-0008 DETAIL.GPJ	
DRAWN	TB	Apr 2014	SCALE N/A
CHECK	AB	Apr 2014	REV.
APPR	JMAC	Apr 2014	

FIGURE B4



LEGEND

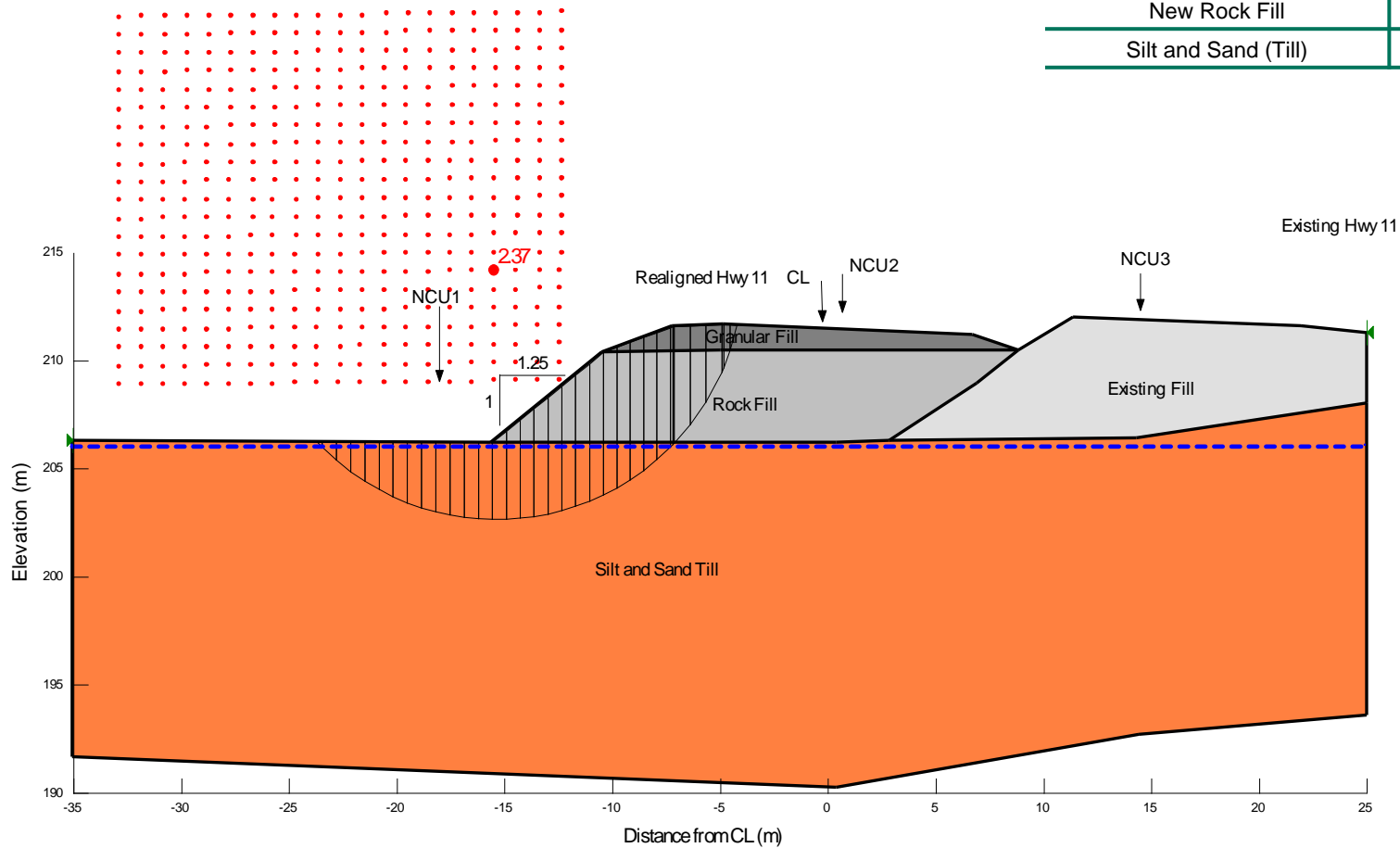
SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	NCU1	2	18.8	12.8	6.0
■	NCU1	6	17.5	12.0	5.5
▲	NCU1	11	15.4	12.2	3.2
+	NCU2	3	17.1	12.0	5.1
◆	NCU2	6	16.0	12.3	3.7
◇	NCU2	9	16.5	12.7	3.8
○	NCU2	12	16.1	12.2	3.9
△	NCU3	9	16.4	11.8	4.6
⊗	NCU3	11	16.6	12.4	4.2
⊕	NCU3	13	16.7	13.1	3.6
□	NCU4	7	18.2	12.7	5.5
⊗	NCU4	10	16.1	12.3	3.8

PROJECT		HIGHWAY 11 NAGAGAMI RIVER BRIDGE CULVERT EXTENSION - STA 20+585		
TITLE		PLASTICITY CHART SANDY SILT to SILT and SAND (TILL)		
 Golder Associates SUDBURY, ONTARIO		PROJECT No. 11-1191-0008 DRAWN TB Apr 2014 CHECK AB Apr 2014 APPR JMAC Apr 2014	FILE No. 11-1191-0008 DETAIL.GPJ SCALE N/A REV.	
		FIGURE B5		

Highway 11 Nagagami River Culvert Replacement Stability Analysis – STA 20+585

Figure B6

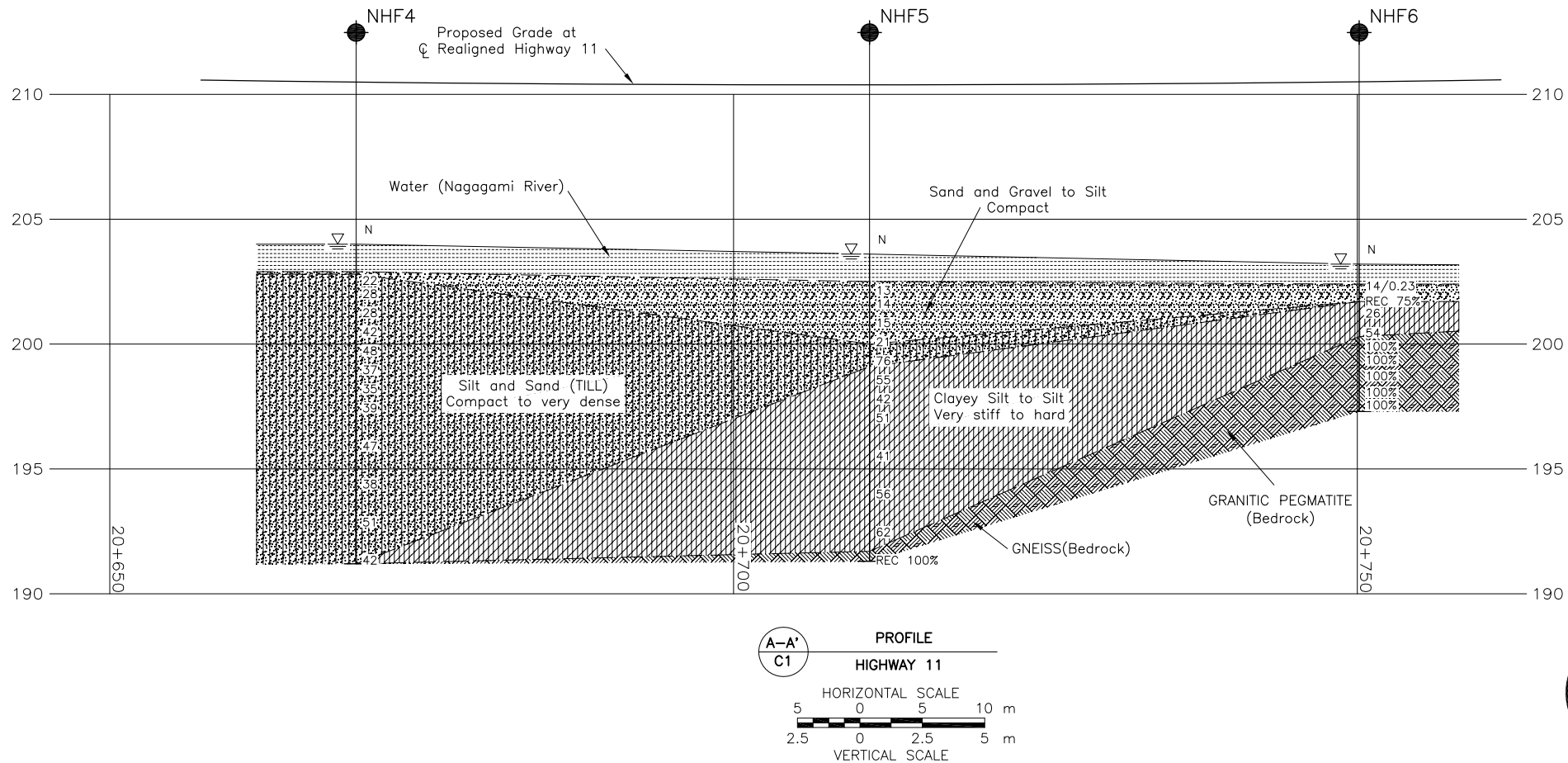
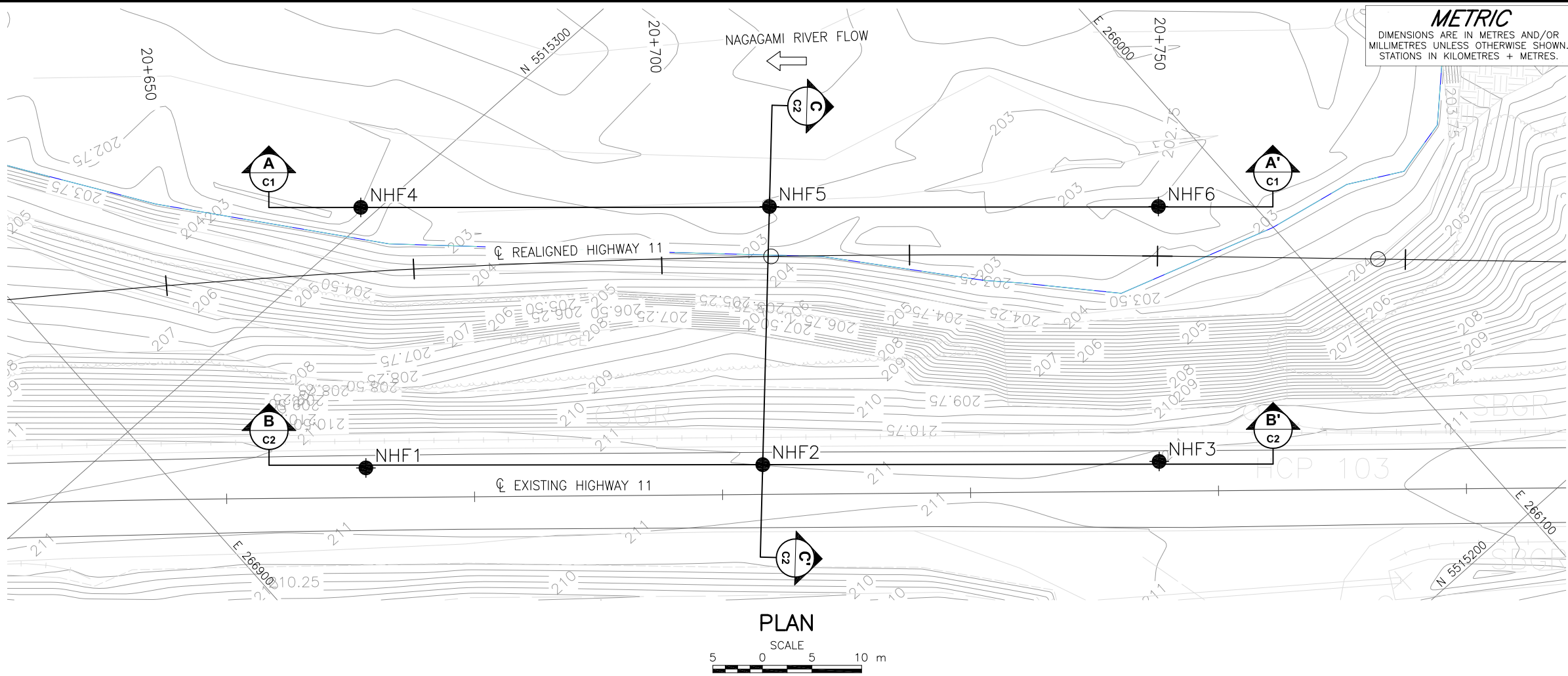
Material Name	Unit Weight (kN/m ³)	Friction Angle (°)
New Granular Fill	21	35
New Rock Fill	19	40
Silt and Sand (Till)	21	35





APPENDIX C

High Fill – STA 20+650 to 20+775



CONT No.
GWP No. 5307-04-00

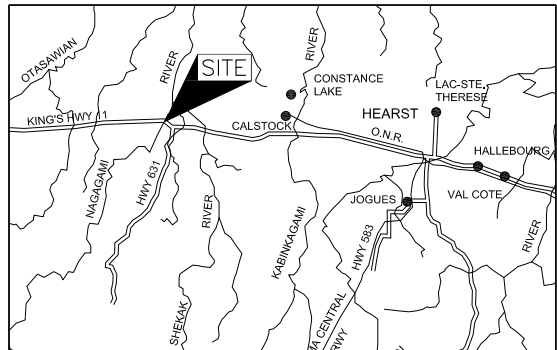


HIGHWAY 11
NAGAGAMI RIVER BRIDGE
HIGH FILL - STA 20+650 TO 20+775
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN

SCALE
0 20 40 km

LEGEND

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

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
NHF1	211.2	5515281.3	265915.5
NHF2	211.0	5515255.0	265945.6
NHF3	211.0	5515228.7	265975.7
NHF4	204.0	5515301.3	265932.6
NHF5	203.6	5515274.0	265963.4
NHF6	203.2	5515247.9	265992.7

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.

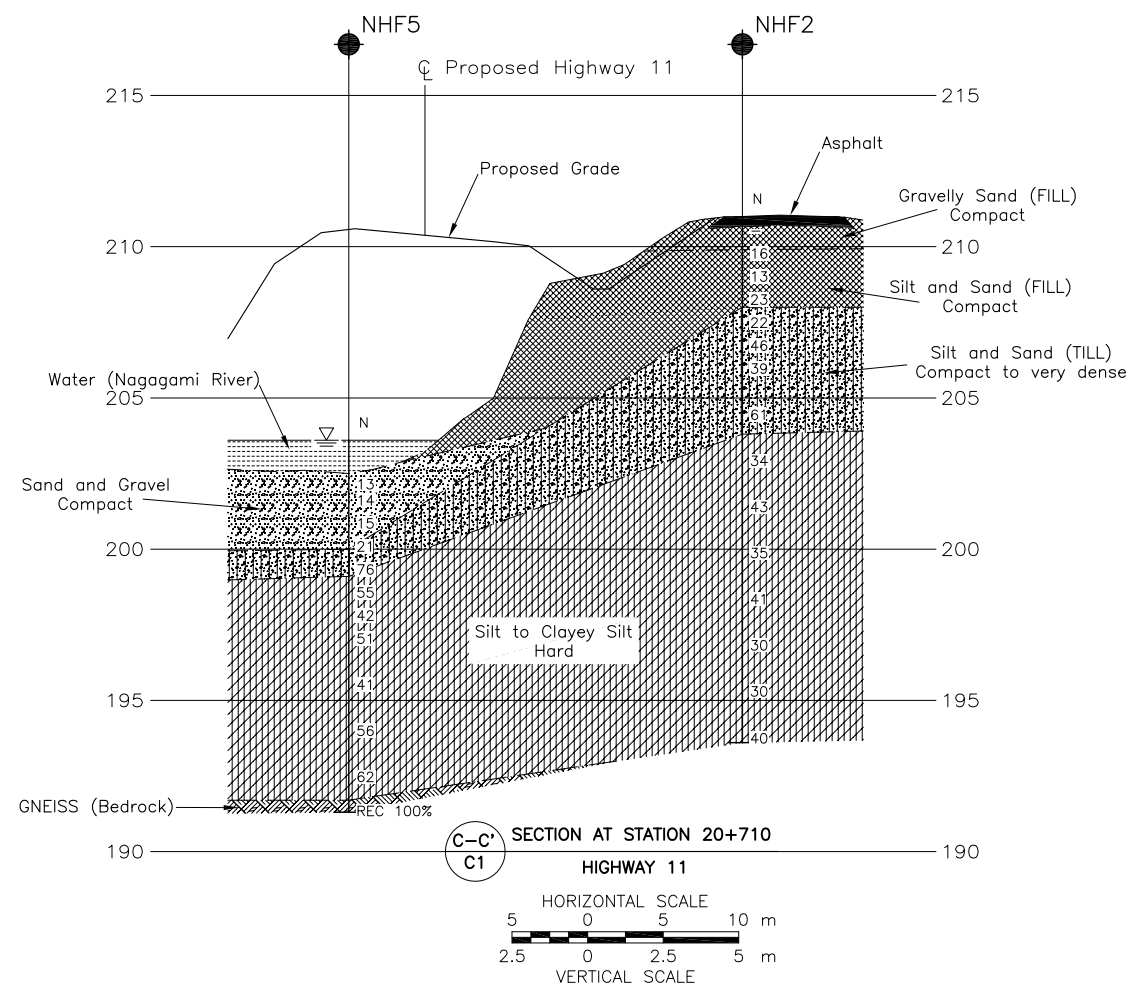
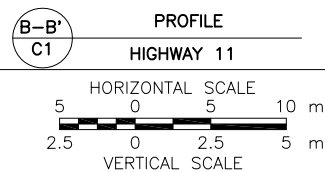
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by LEA, drawing file nos. 8960-Nag-S01.dwg received on May 28, 2013 and x8960 Nagagami Base.dwg, received Sept 25, 2013.



NO.	DATE	BY	REVISION
Geocres No. 42F-28			
HWY. 11	PROJECT NO. 11-1191-0008		DIST.
SUBM'D. AC	CHKD.	DATE: SEP 2014	SITE:
DRAWN: TB	CHKD. AB	APPD. JMAC	DWG. C1



CONT No.
GWP No. 5307-04-00





HIGHWAY 11
NAGAGAMI RIVER BRIDGE
HIGH FILL - STA 20+650 TO 20+775
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

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

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
NHF1	211.2	5515281.3	265915.5
NHF2	211.0	5515255.0	265945.6
NHF3	211.0	5515228.7	265975.7
NHF4	204.0	5515301.3	265932.6
NHF5	203.6	5515274.0	265963.4
NHF6	203.2	5515247.9	265992.7

NOTES

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NO.	DATE	BY	REVISION		
Geocres No. 42F-28					
HWY. 11			PROJECT NO. 11-1191-0008		DIST.
SUBM'D. AC		CHKD.	DATE: SEP 2014		SITE:
DRAWN: TB		CHKD. AB	APPD. JMAC		DWG. C2

PROJECT <u>11-1191-0008</u>		RECORD OF BOREHOLE No NHF1		1 OF 2 METRIC	
G.W.P. <u>5307-04-00</u>		LOCATION <u>N 5515281.3; E 265915.5</u>		ORIGINATED BY <u>SA</u>	
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>		COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>		DATE <u>July 22 and 23, 2013</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	20	40	60						20
211.2	GROUND SURFACE																				
0.0	ASPHALT (400 mm)		1a	AS	-																
210.8			1b																		
0.4	Sand, some gravel, some silt, trace clay (FILL) Compact Brown Moist		2	SS	23													17	65	13	5
209.7																					
1.5	Silt, some sand, trace gravel (FILL) Compact Brown Moist		3	SS	15																
	Peat pockets in Sample 4.		4	SS	14																
208.2																					
3.0	SILT and SAND, some clay, trace to some gravel (TILL) Compact to very dense Grey Moist to wet		5	SS	14													8	32	48	12
			6	SS	35																
			7	SS	39																
			8	SS	37																
			9	SS	31																
			10	SS	33													10	32	45	13
			11	SS	63																
			12	SS	50																
			13	SS	51																

Continued Next Page

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

SUD-MTO 001 11-1191-0008 DETAIL.GPJ CAL-MISS.GDT 08/01/14 DATA INPUT:



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

USUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 08/01/14 DATA INPUT:

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

PROJECT <u>11-1191-0008</u>				RECORD OF BOREHOLE No NHF2				2 OF 2 METRIC									
G.W.P. <u>5307-04-00</u>				LOCATION <u>N 5515255.0; E 265945.6</u>				ORIGINATED BY <u>SA</u>									
DIST <u> </u> HWY <u>11</u>				BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>AC</u>									
DATUM <u>GEODETIC</u>				DATE <u>July 23 and 24, 2013</u>				CHECKED BY <u>AB</u>									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)				
								20	40	60	80	100	20	40	60		
	SILT to CLAYEY SILT, trace sand, trace gravel Hard Grey Wet		14	SS	30		195										6 7 69 18
193.6			15	SS	40		194										
17.4	END OF BOREHOLE Note: 1. Water level not recorded upon completion of drilling.																

SUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 16/01/14 DATA INPUT:

PROJECT		11-1191-0008		RECORD OF BOREHOLE No NHF3		1 OF 1 METRIC								
G.W.P.		5307-04-00		LOCATION		N 5515228.7; E 265975.7								
DIST		HWY 11		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers								
DATUM		GEODETIC		DATE		July 29, 2013								
				ORIGINATED BY		SA								
				COMPILED BY		AC								
				CHECKED BY		AB								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
211.0	GROUND SURFACE													
0.0	ASPHALT (400 mm)		1a	AS	-									
210.6			1b											
0.4	Gravelly sand to sand and gravel, trace to some silt (FILL) Loose to dense Brown Moist		2	SS	20									
			3	SS	11									
			4	SS	4									
			5	SS	34									
			6	SS	42									
			7	SS	39									
205.4														
5.6	SAND and GRAVEL, some silt Very dense Grey Wet		8	SS	61									
203.8														
7.2	Sandy SILT, some clay, trace to some gravel (TILL) Very dense Grey Moist to wet		9	SS	81									
202.3														
8.7	CLAYEY SILT, trace sand Hard Brown Wet		10	SS	55									
	Augers grinding between 10.4 m and 10.8 m depth.		11	SS	51									
			12	SS	58									
198.0														
13.0	END OF BOREHOLE AUGER REFUSAL													
	Note: 1. Water level at a depth of 9.8 m below ground surface (Elev. 201.2 m) upon completion of drilling.													





SUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 08/01/14 DATA INPUT:

PROJECT 11-1191-0008		RECORD OF BOREHOLE No NHF4				1 OF 1 METRIC										
G.W.P. 5307-04-00		LOCATION N 5515301.3; E 265932.6				ORIGINATED BY EHS										
DIST _____ HWY 11		BOREHOLE TYPE HW Casing, NW Casing				COMPILED BY AC										
DATUM GEODETIC		DATE July 28 and 29, 2013				CHECKED BY AB										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
204.0	WATER SURFACE															
0.0	WATER															
202.9																
1.2	SAND and GRAVEL Grey Wet		1	SS	22											
	SILT and SAND, some clay, trace to some gravel (TILL) Compact to very dense Grey Wet		2	SS	28											
			3	SS	28											
			4	SS	42											
			5	SS	48											
			6	SS	37											
			7	SS	35											
			8	SS	39											
			9	SS	47											
			10	SS	38											
			11	SS	51											
			12	SS	42											
191.2	END OF BOREHOLE															
12.8	Note: 1. Water level inside casing at water surface (Elev. 204.0 m) upon completion of drilling.															

SUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 08/01/14 DATA INPUT:

PROJECT 11-1191-0008		RECORD OF BOREHOLE No NHF5				1 OF 1 METRIC								
G.W.P. 5307-04-00		LOCATION N 5515274.0; E 265963.4				ORIGINATED BY EHS								
DIST _____ HWY 11		BOREHOLE TYPE HW Casing, NW Casing and NQ Coring				COMPILED BY AC								
DATUM GEODETIC		DATE July 27 and 28, 2013				CHECKED BY AB								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
203.6	WATER SURFACE													
0.0	WATER													
202.5														
1.1	SAND and GRAVEL, trace to some silt, trace clay Compact Brown Wet		1	SS	13									
			2	SS	14									
			3	SS	15									38 50 10 2
			4	SS	21									
200.0														
3.6	SILT and SAND, some clay, trace to some gravel (TILL) Very dense Grey Moist		5	SS	76									6 35 47 12
199.1														
4.5	CLAYEY SILT, trace to some sand, trace gravel Hard Brown to grey Moist to wet		6	SS	55									
			7	SS	42									0 4 74 22
			8	SS	51									
			9	SS	41									
			10	SS	56									
			11	SS	62									1 1 77 21
191.7	GNEISS (BEDROCK)		-	RC	REC 100%									
191.3	END OF BOREHOLE													
12.3	Note: 1. Water level inside casing at water surface (Elev. 203.6 m) upon completion of drilling.													

SUD-MTO 001 11-1191-0008 DETAIL.GPJ CAL-MISS.GDT 08/01/14 DATA INPUT:

PROJECT 11-1191-0008		RECORD OF BOREHOLE No NHF6				1 OF 1 METRIC								
G.W.P. 5307-04-00		LOCATION N 5515247.9; E 265992.7				ORIGINATED BY EHS								
DIST _____ HWY 11		BOREHOLE TYPE HW Casing, NW Casing and NQ Coring				COMPILED BY AC								
DATUM GEODETIC		DATE July 25 to 27, 2013				CHECKED BY AB								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
203.2 0.0	WATER SURFACE WATER													
202.4 0.8	SAND and GRAVEL Compact Brown Wet		1	SS	14/0.2									
201.7 1.5	CLAYEY SILT, trace sand Very stiff to hard Brown Wet		-	RC	REC 75%									
			2	SS	26									
			3	SS	54									
200.3 2.9	GRANITIC PEGMATITE (BEDROCK) Bedrock cored from 2.9 m to 5.9 m depth. For coring details see Record of Drillhole NHF6.		1	RC	REC 100%									RQD = 100%
			2	RC	REC 100%									RQD = 100%
			3	RC	REC 100%									RQD = 100%
			4	RC	REC 100%									RQD = 100%
			5	RC	REC 100%									RQD = 100%
197.3 5.9	END OF BOREHOLE Note: 1. Water level inside casing at water surface (Elev. 203.2 m) upon completion of drilling.													

SUD-MTO 001 11-1191-0008 DETAIL.GPJ CAL-MISS.GDT 08/01/14 DATA INPUT:

PROJECT: 11-1191-0008

RECORD OF DRILLHOLE: NHF6

SHEET 1 OF 1

LOCATION: N 5515247.9 ; E 265992.7

DRILLING DATE: July 25 to 27, 2013

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 BOMB

DRILLING CONTRACTOR: Landcore

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate										BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage										PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular										PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break										BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
							RECOVERY					R.Q.D. %					FRACT. INDEX METRES					DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY					Diametral Point Load Index (MPa)					RMC -Q' AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
							TOTAL CORE %					SOLID CORE %										B Angle					DIP w.r.t. CORE AXIS					TYPE AND SURFACE DESCRIPTION										Jr Ja Jn								k, cm/s																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
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DEPTH SCALE

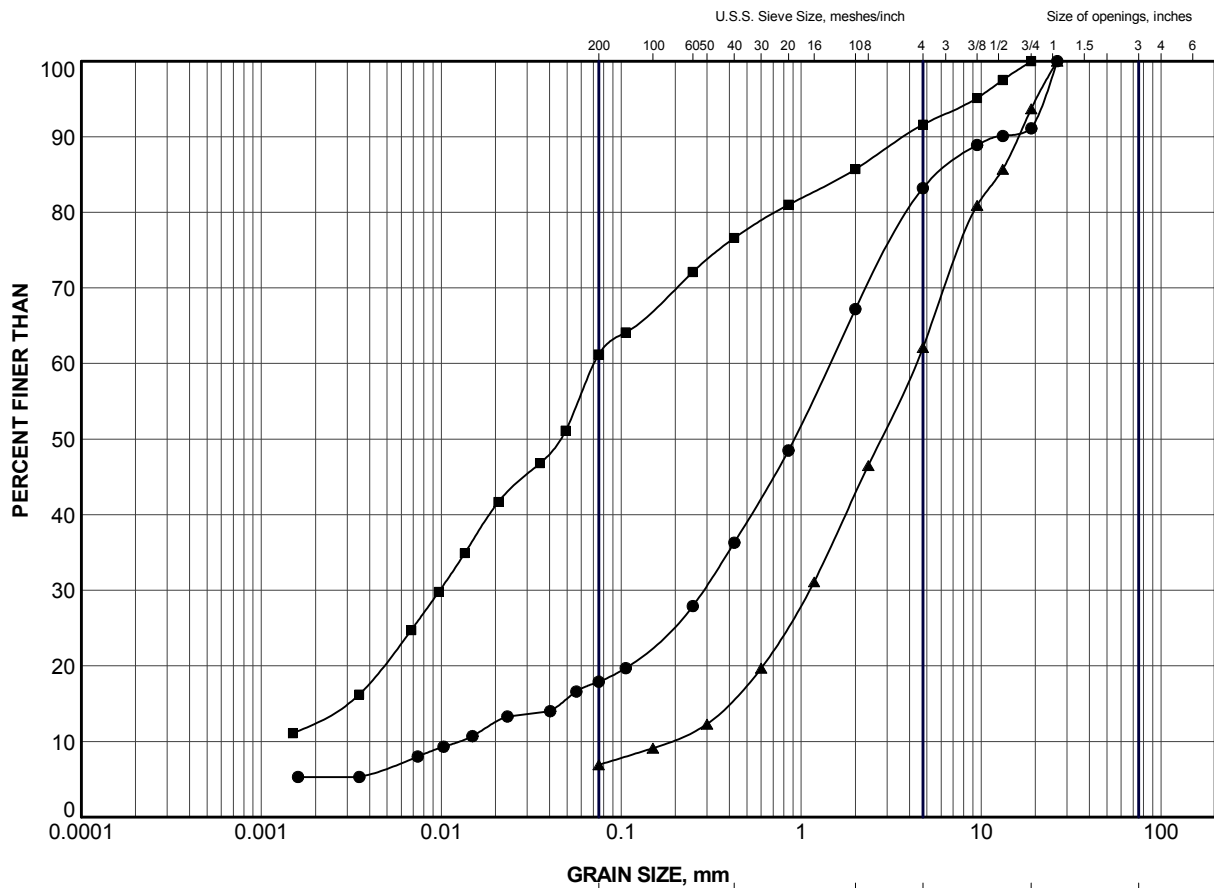
1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 11-1191-0008 DETAIL GPJ GAL-MISS GDT 08/01/14 DATA INPUT:



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NHF1	1	210.8
■	NHF2	3	209.2
▲	NHF3	6	206.9

PROJECT

HIGHWAY 11
NAGAGAMI RIVER BRIDGE

TITLE

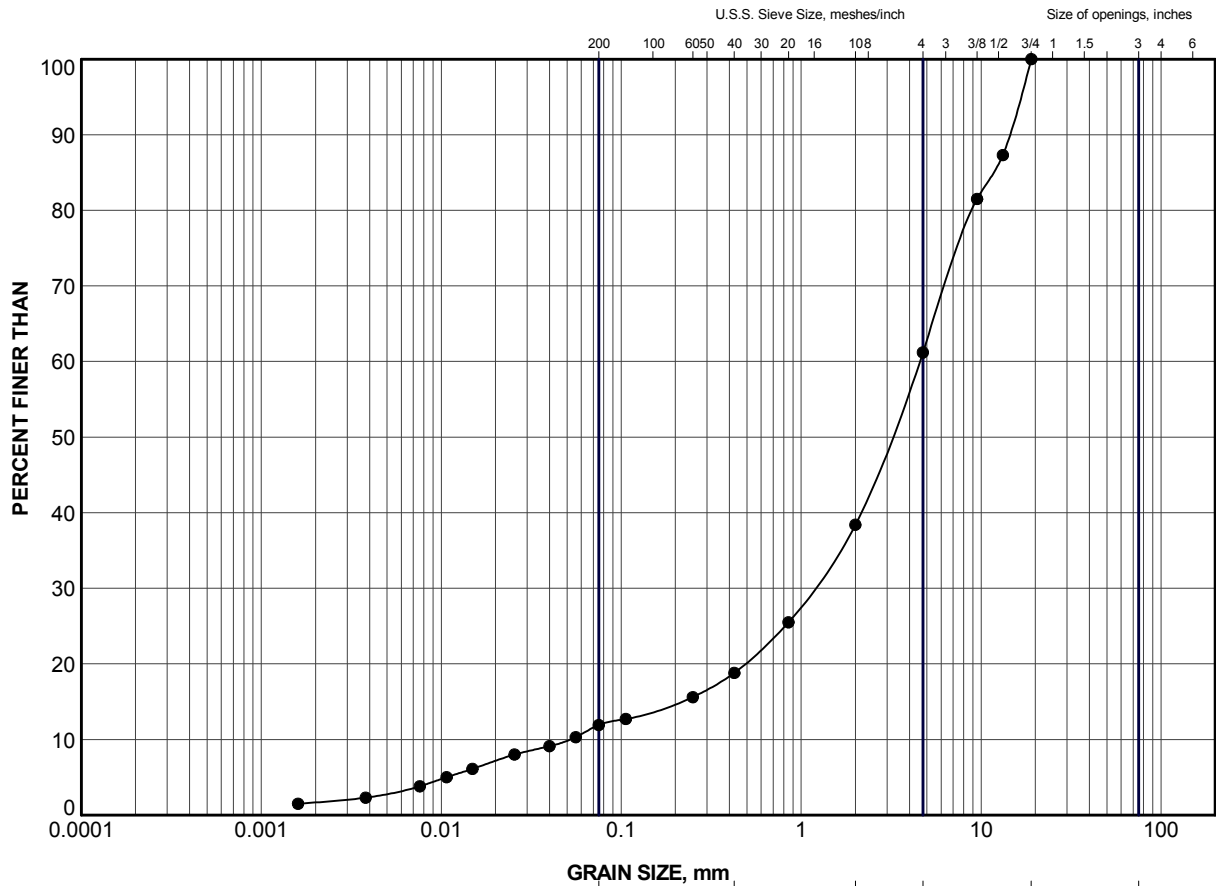
GRAIN SIZE DISTRIBUTION
SILT and SAND to SAND and GRAVEL (FILL)



Golder Associates
SUDBURY, ONTARIO


PROJECT No.	11-1191-0008	FILE No.	11-1191-0008 DETAIL.GPJ
DRAWN	JJL	Jan 2014	SCALE N/A REV.
CHECK	AB	Jan 2014	
APPR	JMAC	Jan 2014	

FIGURE C1

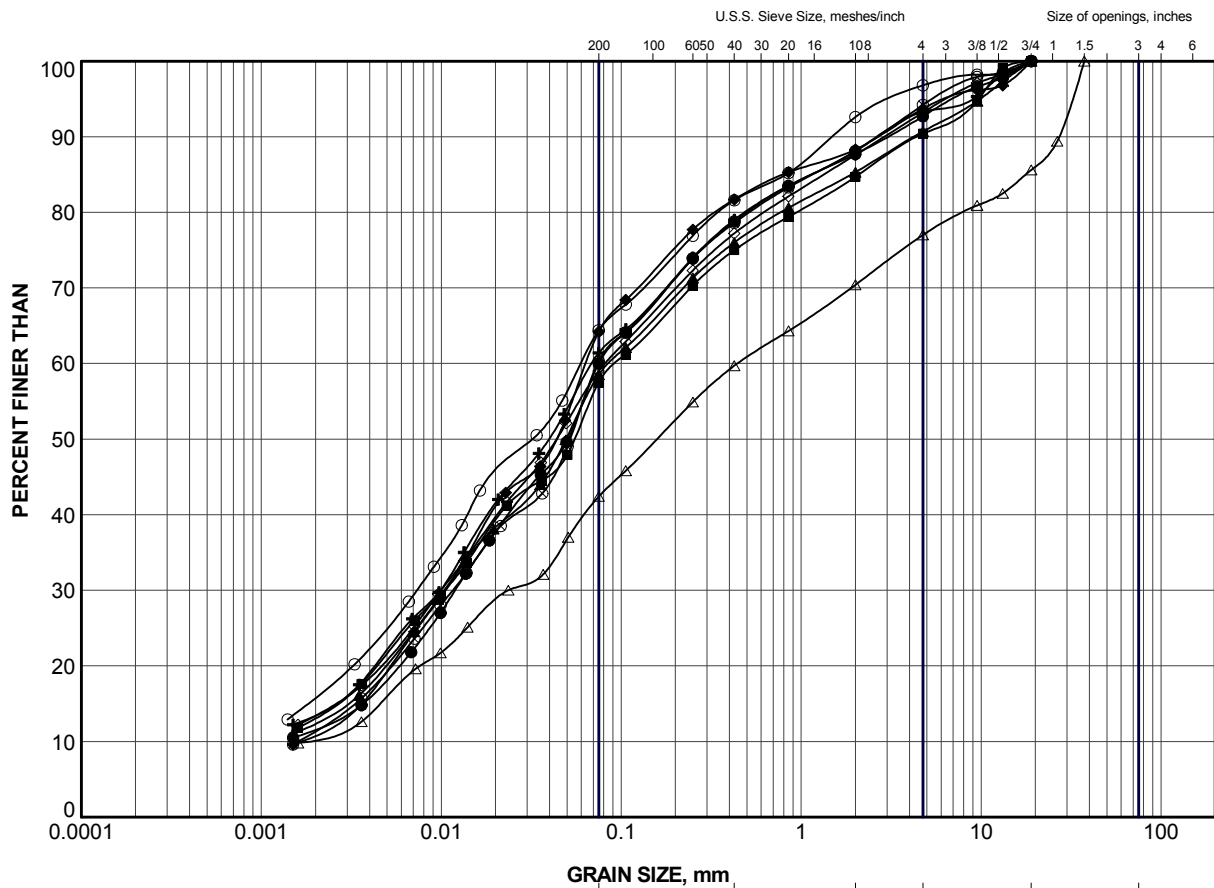


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NHF5	3	201.0

PROJECT					HIGHWAY 11 NAGAGAMI RIVER BRIDGE				
TITLE					GRAIN SIZE DISTRIBUTION SAND and GRAVEL				
		PROJECT No. 11-1191-0008			FILE No. 1191-0008 DETAIL.GPJ				
		DRAWN	JJL	Jan 2014	SCALE	N/A	REV.		
		CHECK	AB	Jan 2014	FIGURE C2				
		APPR	JMAC	Jan 2014					

SUD-MTO GSD (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NHF1	5	207.9
■	NHF1	10	201.8
▲	NHF1	14	195.7
+	NHF2	8	204.6
◆	NHF3	9	203.1
◇	NHF4	3	201.4
○	NHF4	8	197.6
△	NHF4	12	191.5
⊗	NHF5	5	199.5

PROJECT

HIGHWAY 11
NAGAGAMI RIVER BRIDGE

TITLE

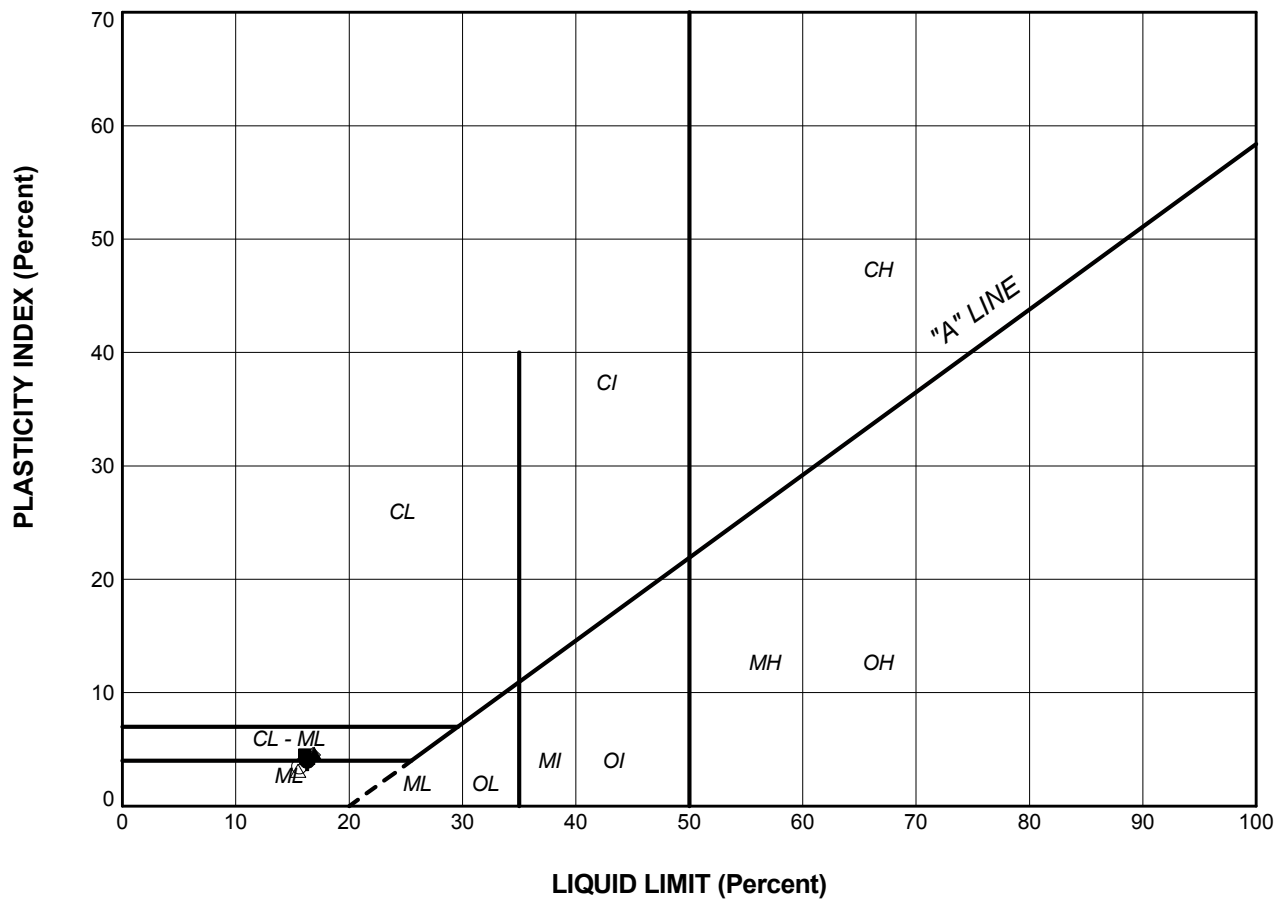
GRAIN SIZE DISTRIBUTION
SANDY SILT to SILT and SAND (TILL)



**Golder
Associates**
SUDBURY, ONTARIO


PROJECT No.	11-1191-0008	FILE No.	1191-0008 DETAIL.GPJ
DRAWN	JJL	Jan 2014	SCALE N/A
CHECK	AB	Jan 2014	REV.
APPR	JMAC	Jan 2014	

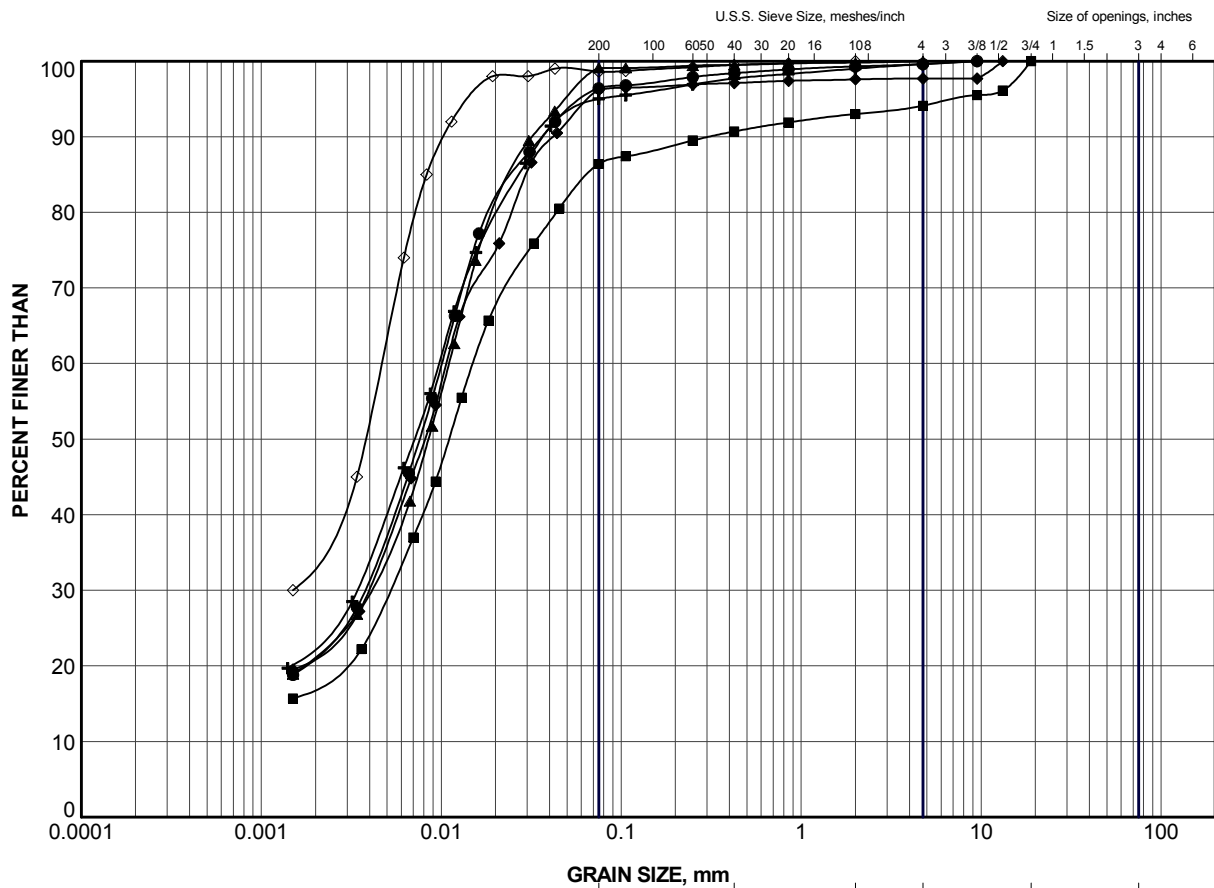
FIGURE C3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	NHF1	10	16.3	12.3	4.0
■	NHF1	14	16.1	11.6	4.5
▲	NHF2	8	16.9	12.3	4.6
+	NHF3	9	16.3	12.5	3.8
◆	NHF4	3	16.1	12.0	4.1
◇	NHF4	8	16.8	12.3	4.5
○	NHF4	12	15.6	12.1	3.5
△	NHF5	5	15.5	12.4	3.1

PROJECT					HIGHWAY 11 NAGAGAMI RIVER BRIDGE				
TITLE					PLASTICITY CHART SANDY SILT to SILT and SAND (TILL)				
PROJECT No. 11-1191-0008			FILE No. 11-1191-0008 DETAIL.GPJ						
DRAWN	JJL	Jan 2014	SCALE	N/A	REV.				
CHECK	AB	Jan 2014							
APPR	JMAC	Jan 2014							
 Golder Associates SUDBURY, ONTARIO			FIGURE C4						



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NHF2	11	200.0
■	NHF2	14	195.5
▲	NHF3	11	200.0
+	NHF5	7	198.0
◆	NHF5	11	192.6
◇	NHF6	2	201.4

PROJECT

HIGHWAY 11
NAGAGAMI RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

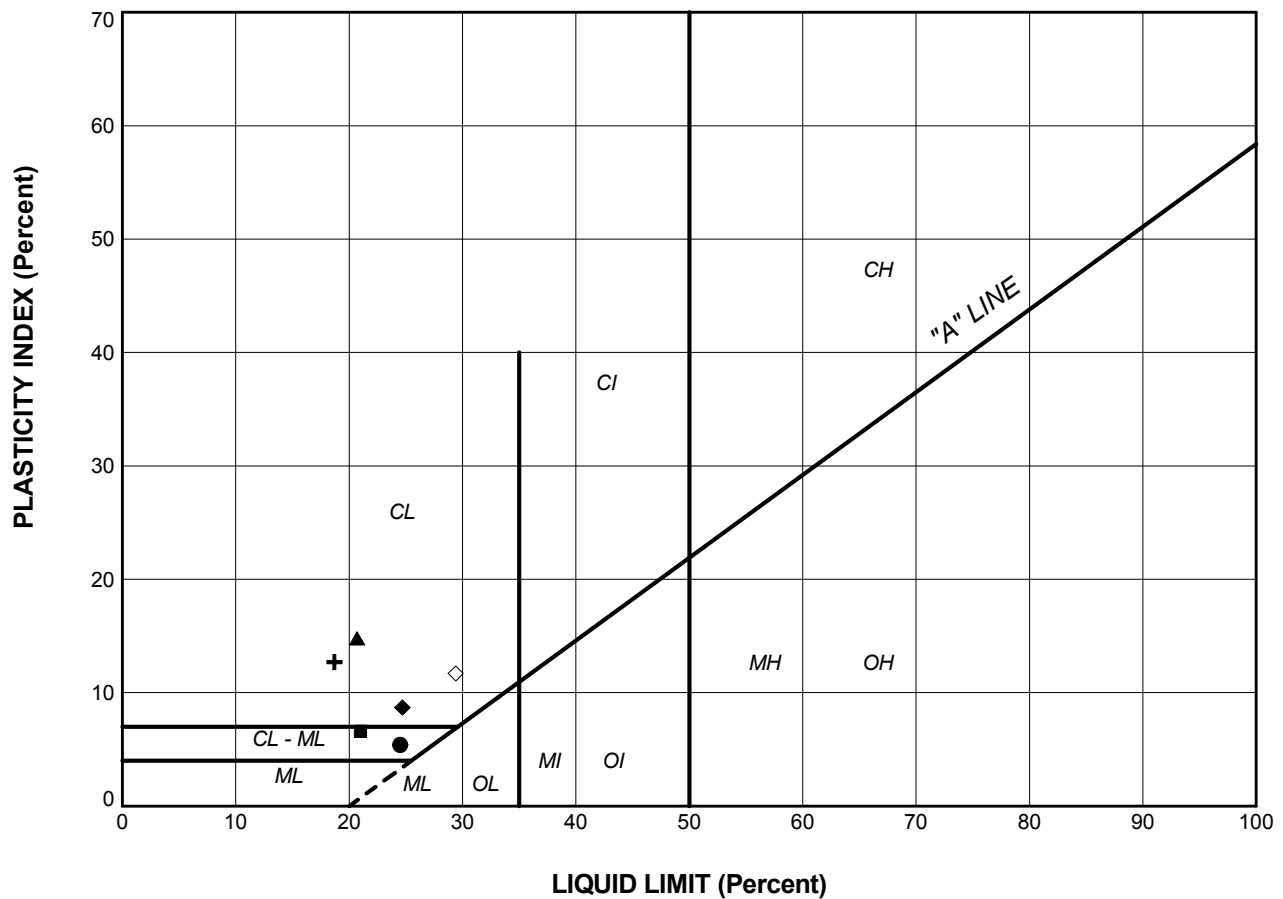
SILT to CLAYEY SILT



**Golder
Associates**
SUDBURY, ONTARIO


PROJECT No.	11-1191-0008	FILE No.	1191-0008 DETAIL.GPJ
DRAWN	JJL	Jan 2014	SCALE N/A
CHECK	AB	Jan 2014	REV.
APPR	JMAC	Jan 2014	

FIGURE C5



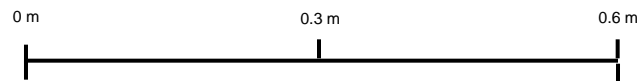
LEGEND


SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	NHF2	11	24.5	19.1	5.4
■	NHF2	14	21.0	14.4	6.6
▲	NHF3	11	20.7	5.9	14.8
+	NHF5	7	18.7	6.0	12.7
◆	NHF5	11	24.7	16.0	8.7
◇	NHF6	2	29.4	17.7	11.7

PROJECT					HIGHWAY 11 NAGAGAMI RIVER BRIDGE				
TITLE					PLASTICITY CHART SILT to CLAYEY SILT				
PROJECT No. 11-1191-0008			FILE No. 11-1191-0008 DETAIL.GPJ						
DRAWN	JJL	Jan 2014	SCALE	N/A	REV.				
CHECK	AB	Jan 2014							
APPR	JMAC	Jan 2014							
 Golder Associates SUDBURY, ONTARIO			FIGURE C6						



Borehole NHF6
Elevation 200.3 m to 197.3 m

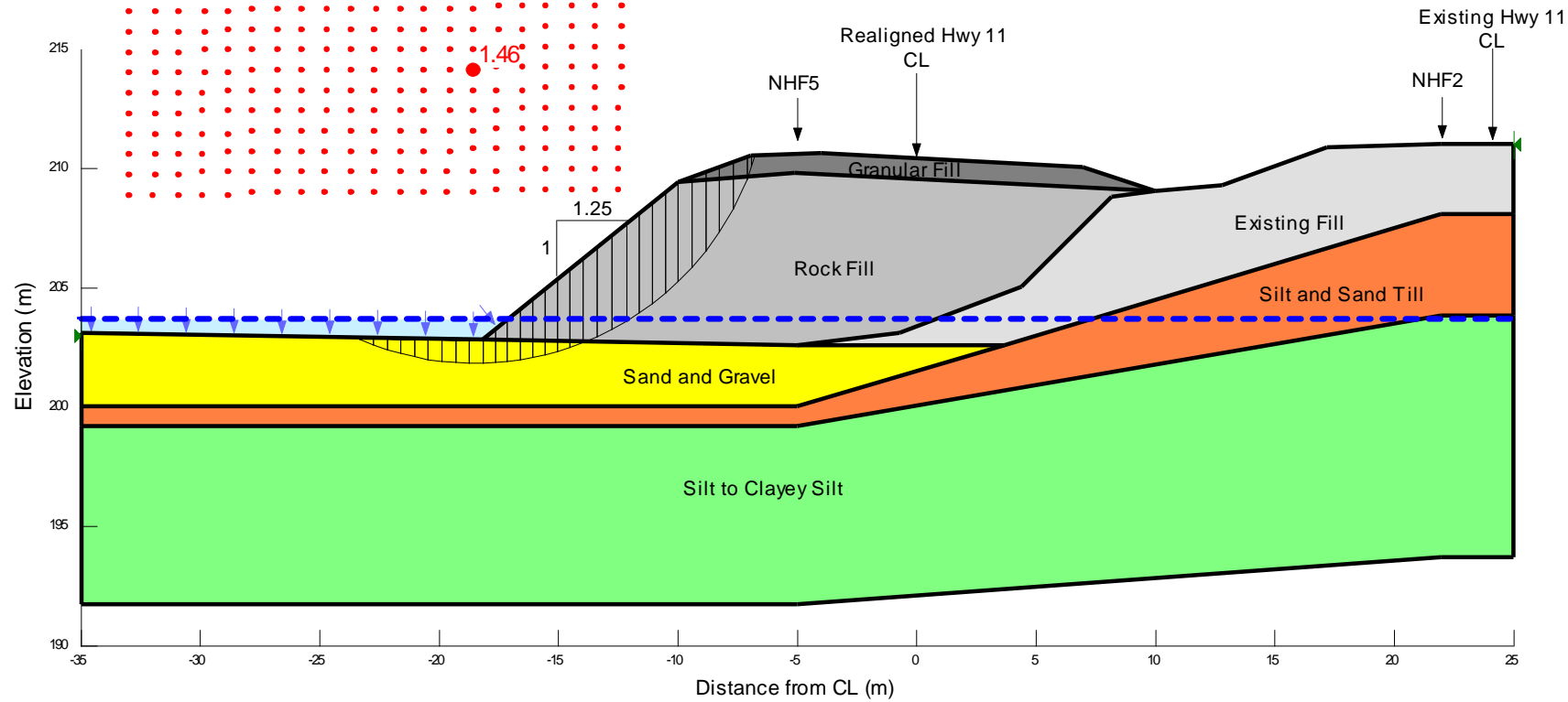


PROJECT		HIGHWAY 11 NAGAGAMI RIVER BRIDGE			
TITLE		ROCK CORE PHOTOGRAPH			
	PROJECT No. 11-1191-0008		FILE No. ----		
	DESIGN	AC	NOV 2013	SCALE AS SHOWN	REV.
	CADD	--			
	CHECK	AB	NOV 2013	FIGURE C7	
	REVIEW				

Highway 11 Nagagami River High Fill Embankment Stability Analysis – STA 20+710

Figure C8

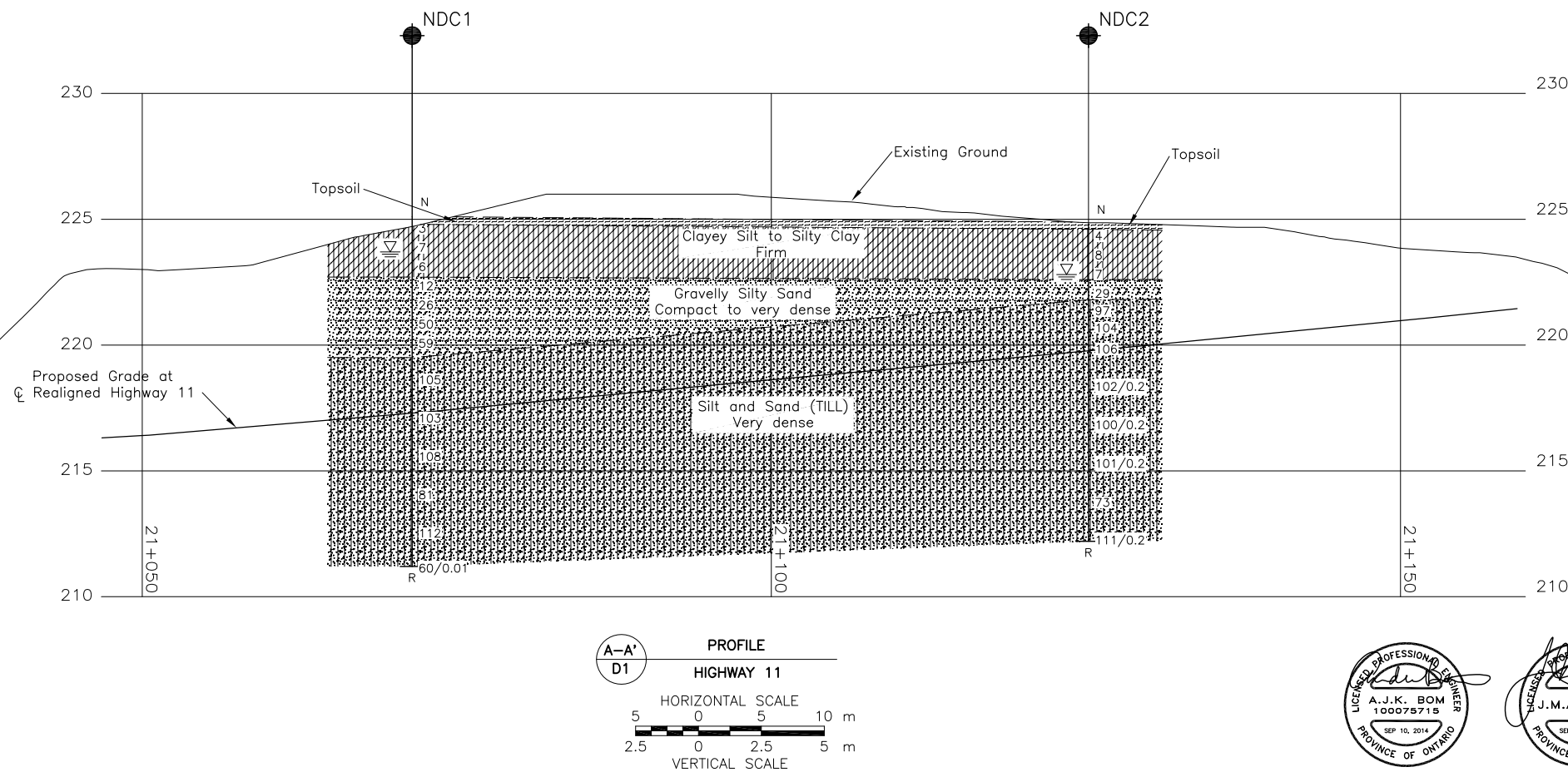
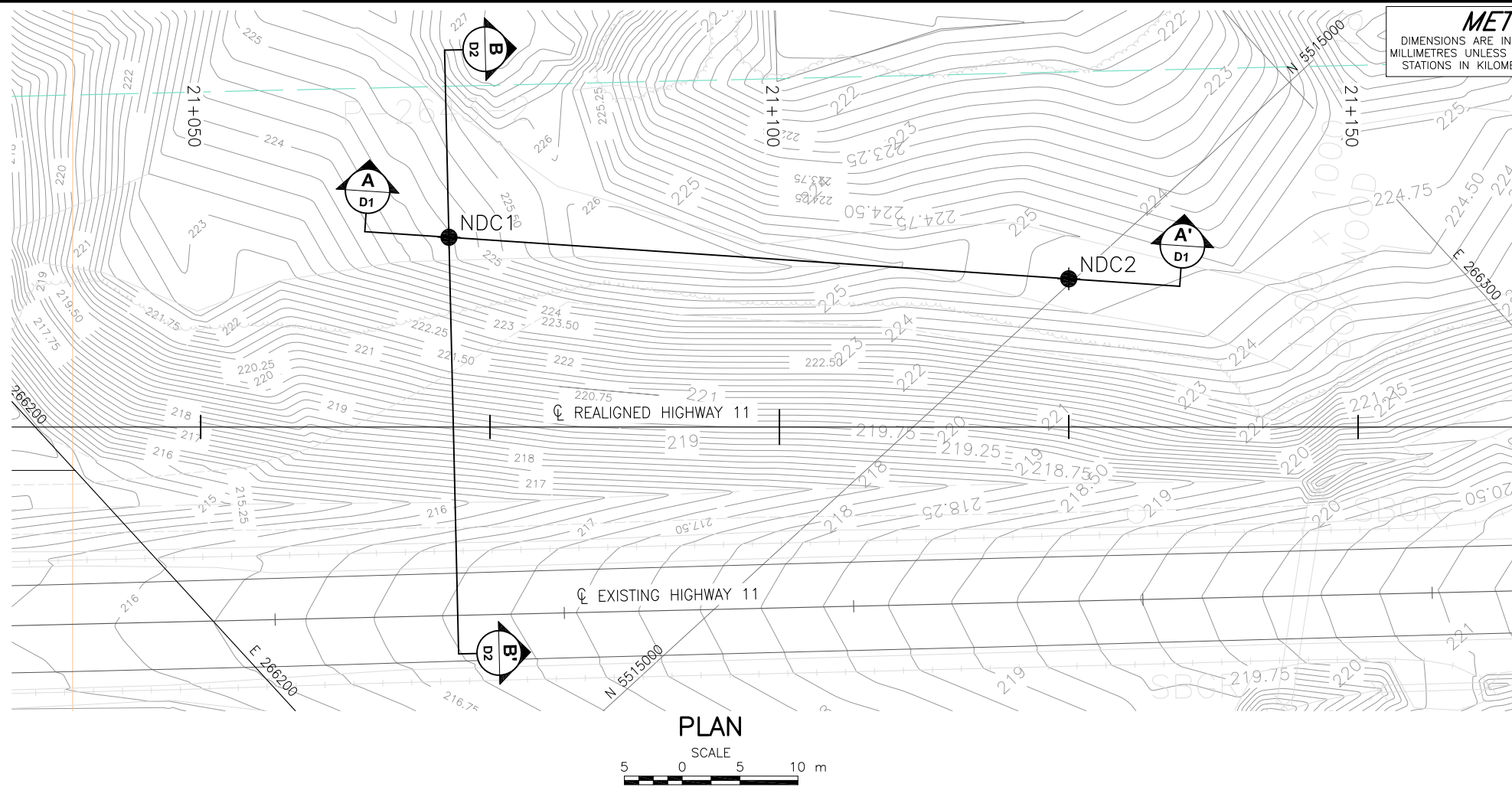
Material Name	Unit Weight (kN/m ³)	Friction Angle (°)	Cohesion (kPa)
Existing Fill	20	35	-
New Rock Fill	19	40	-
New Granular Fill	21	35	-
Sand and Gravel	21	35	-
Silt and Sand (Till)	21	35	-
Silt to Clayey Silt	19	-	200



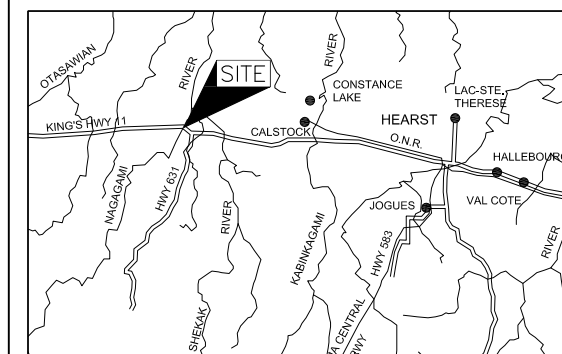


APPENDIX D

Deep Cut – STA 21+050 to 21+150

CONT No.
GWP No. 5307-04-00HIGHWAY 11
NAGAGAMI RIVER BRIDGE
DEEP CUT - STA 21+050 TO 21+150
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

KEY PLAN

SCALE

20 0 20 40 km

LEGEND

- Borehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
NDC1	225.1	5515039.2	266237.4
NDC2	224.8	5515000.3	266274.5

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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by LEA, drawing file nos. 8960-Nag-S01.dwg received on May 28, 2013 and x8960 Nagagami Base.dwg, received Sept 25, 2013.



NO.	DATE	BY	REVISION
Geocres No. 42F-28			
HWY. 11	PROJECT NO. 11-1191-0008		DIST.
SUBM'D. AC	CHKD.	DATE: SEP 2014	SITE:
DRAWN: TB	CHKD. AB	APPD. JMAC	DWG. D1

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

CONT No.
GWP No. 5307-04-00

HIGHWAY 11
NAGAGAMI RIVER BRIDGE
DEEP CUT - STA 21+050 TO 21+150
SOIL STRATA

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
NDC1	225.1	5515039.2	266237.4
NDC2	224.8	5515000.3	266274.5

NOTES

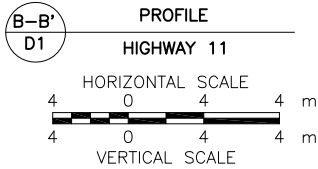
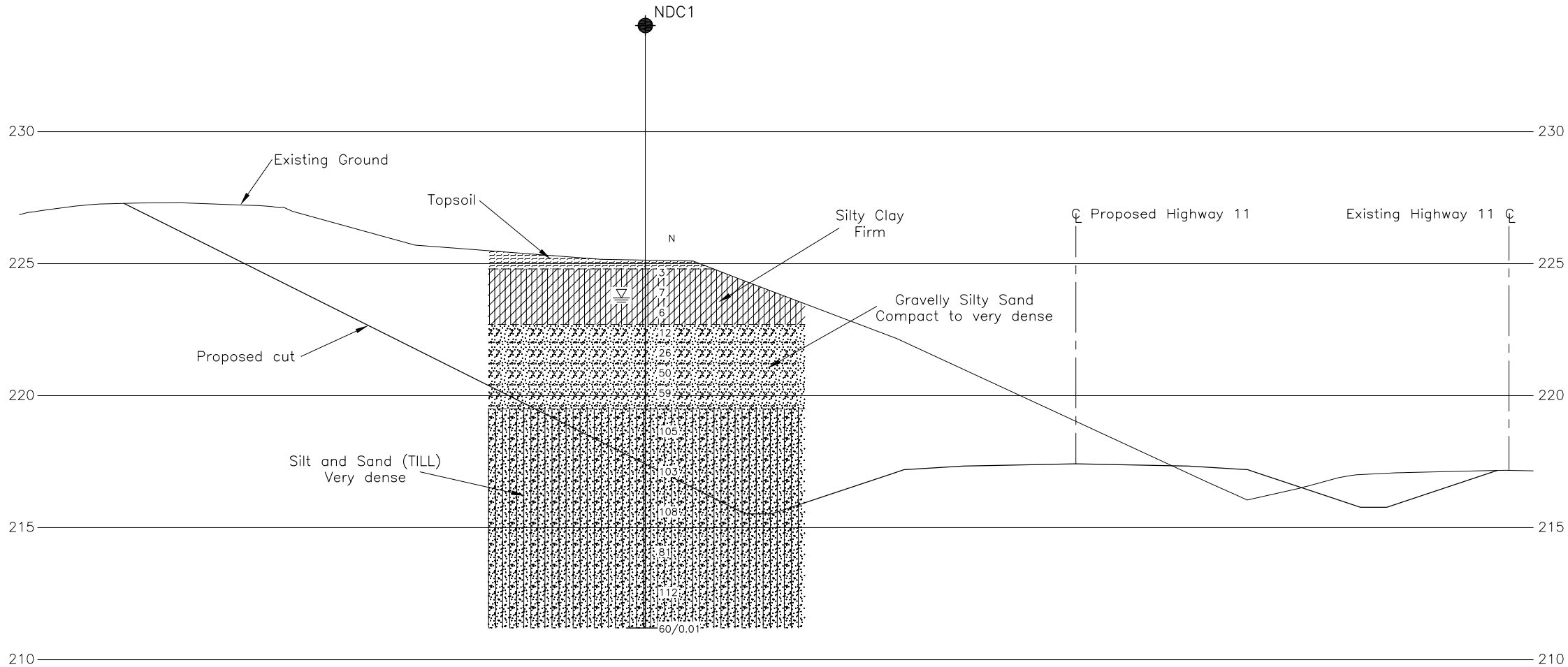
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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REFERENCE

Base plans provided in digital format by LEA, drawing file nos. 8960-Nag-S01.dwg received on May 28, 2013 and x8960 Nagagami Base.dwg, received Sept 25, 2013. Profile provided by LEA, drawing file no. x8960 Naga Prof.dwg, received Feb 5, 2014.



NO.	DATE	BY	REVISION
Geocres No. 42F-28			
HWY. 11	PROJECT NO. 11-1191-0008		DIST.
SUBM'D. AC	CHKD.	DATE: SEP 2014	SITE:
DRAWN: TB	CHKD. AB	APPD. JMAC	DWG. D2

PROJECT 11-1191-0008			RECORD OF BOREHOLE No NDC1			1 OF 2 METRIC																				
G.W.P. 5307-04-00			LOCATION N 5515039.2; E 266237.4			ORIGINATED BY EHS																				
DIST _____ HWY 11			BOREHOLE TYPE NW Casing			COMPILED BY AC																				
DATUM GEODETIC			DATE August 22, 2013			CHECKED BY AB																				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE	20	40	60	80	100	W _p	W	W _L	γ	GR	SA	SI	CL						
225.1	GROUND SURFACE																									
0.0	TOPSOIL		1	SS	3		225																			
0.3	Black Moist																									
	SILTY CLAY, trace sand		2	SS	7		224																			
	Firm																									
	Brown		3	SS	6																					
	Moist to wet																									
222.7							223																			
2.4	Gravelly Silty SAND, trace clay		4	SS	12																					
	Compact to very dense																									
	Brown		5	SS	26		222																			
	Wet																									
			6	SS	50		221																			
			7	SS	59		220																			
219.5																										
5.6	SILT and SAND, some clay, trace to some gravel (TILL)		8	SS	105		219																			
	Very dense																									
	Grey						218																			
	Wet																									
			9	SS	103		217																			
			10	SS	108		216																			
			11	SS	81		215																			
			12	SS	112		214																			
							213																			
							212																			
211.2			13	SS	80/0.01																					
13.9																										

Continued Next Page

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

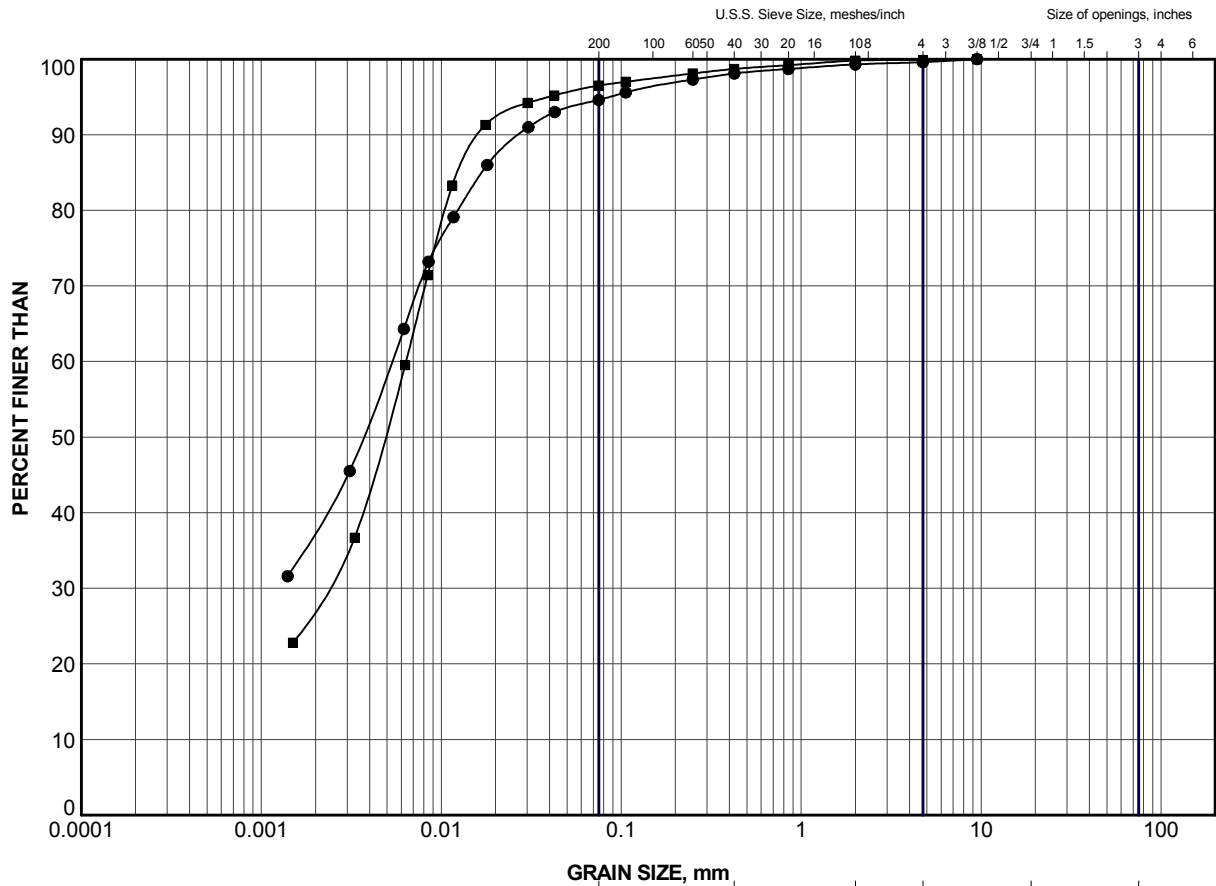
SUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 08/01/14 DATA INPUT:

PROJECT <u>11-1191-0008</u>		RECORD OF BOREHOLE No NDC1		2 OF 2 METRIC	
G.W.P. <u>5307-04-00</u>		LOCATION <u>N 5515039.2; E 266237.4</u>		ORIGINATED BY <u>EHS</u>	
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>NW Casing</u>		COMPILED BY <u>AC</u>	
DATUM <u>GEODETIC</u>		DATE <u>August 22, 2013</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL	
								○ UNCONFINED	● QUICK TRIAXIAL	+	×	FIELD VANE	REMOULDED	WATER CONTENT (%)							
	END OF BOREHOLE SPLIT SPOON REFUSAL Note: 1. Water level at a depth of 1.4 m below ground surface (Elev. 223.7 m) upon completion of drilling.																				

SUD-MTO 001 11-1191-0008 DETAIL.GPJ GAL-MISS.GDT 08/01/14 DATA INPUT:


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

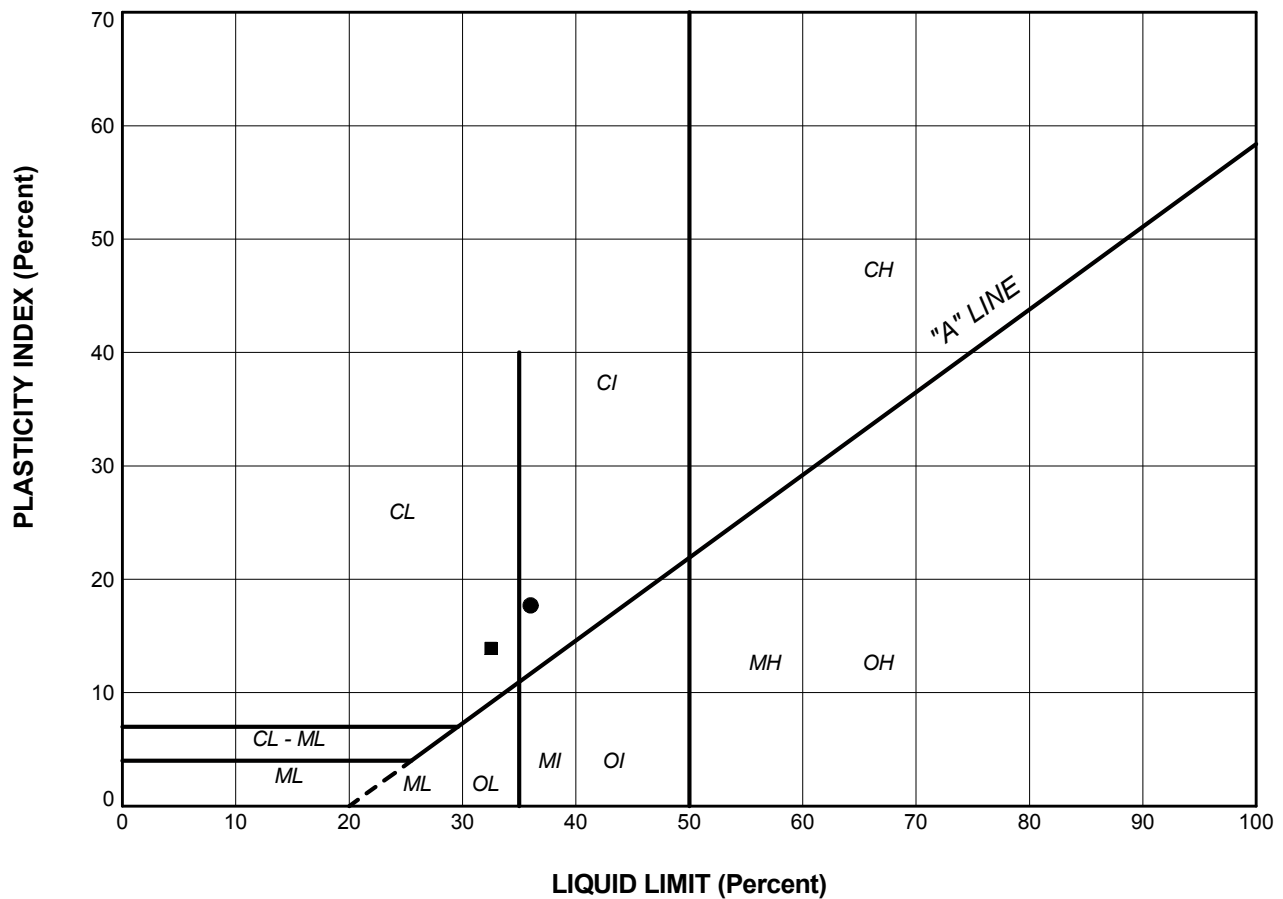


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NDC1	2	224.0
■	NDC2	2	223.7

PROJECT					
HIGHWAY 11 NAGAGAMI RIVER BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT to SILTY CLAY					
PROJECT No.		11-1191-0008		FILE No-1191-0008 DETAIL.GPJ	
DRAWN	JJL	Jan 2014	SCALE	N/A	REV.
CHECK	AB	Jan 2014			
APPR	JMAC	Jan 2014			
 Golder Associates SUDBURY, ONTARIO			FIGURE D1		

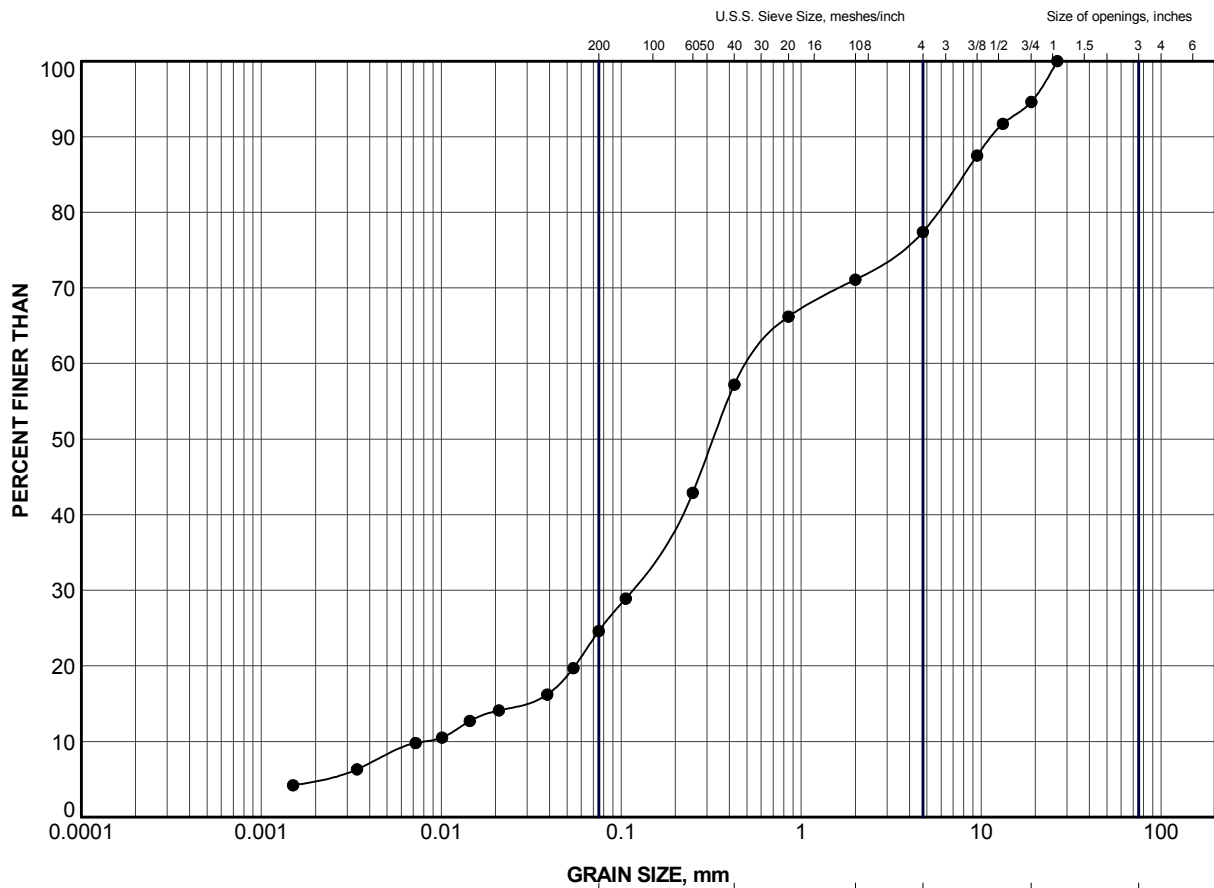


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	NDC1	2	36.0	18.3	17.7
■	NDC2	2	32.5	18.6	13.9

PROJECT			HIGHWAY 11 NAGAGAMI RIVER BRIDGE		
TITLE			PLASTICITY CHART CLAYEY SILT to SILTY CLAY		
PROJECT No.		11-1191-0008	FILE No. 11-1191-0008 DETAIL.GPJ		
DRAWN	JJL	Jan 2014	SCALE	N/A	REV.
CHECK	AB	Jan 2014	FIGURE D2		
APPR	JMAC	Jan 2014			





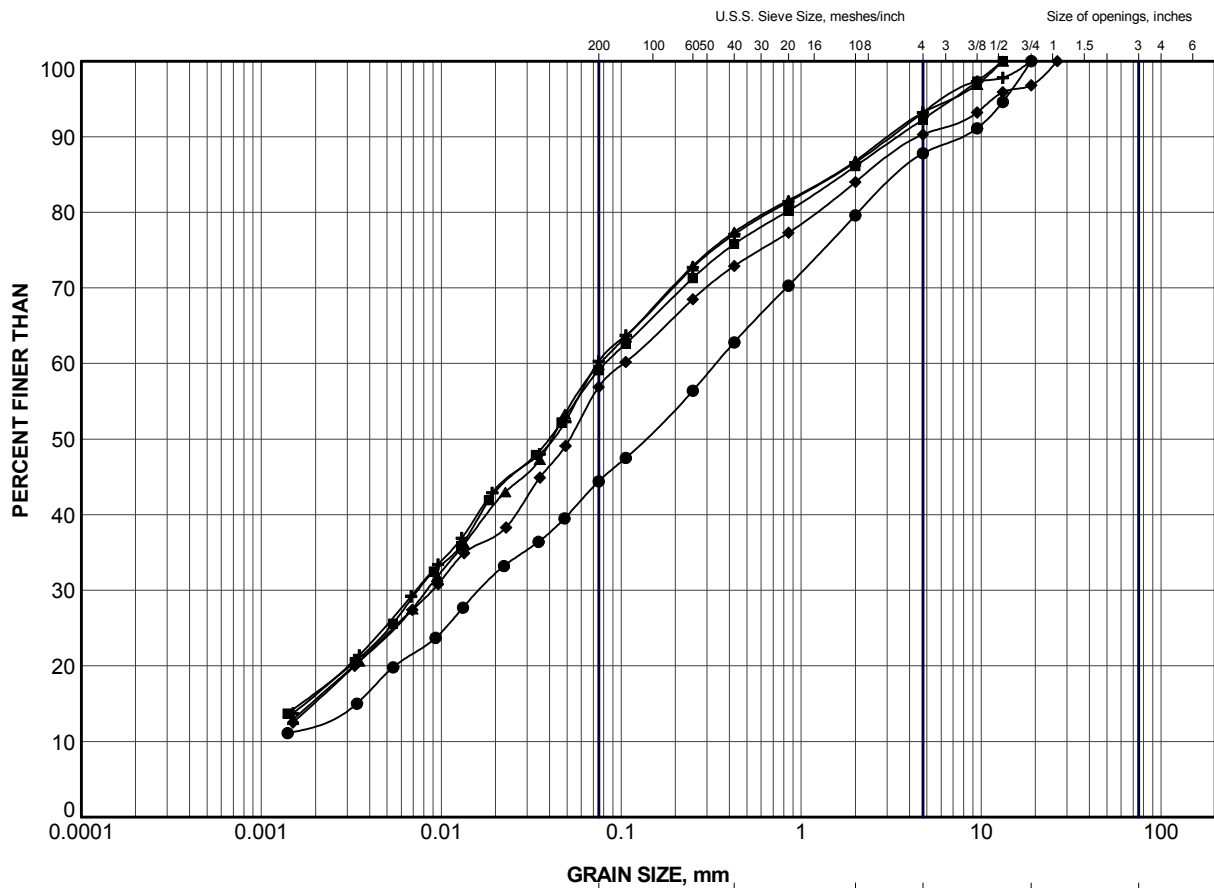
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NDC1	6	221.0

PROJECT					HIGHWAY 11 NAGAGAMI RIVER BRIDGE				
TITLE					GRAIN SIZE DISTRIBUTION GRAVELLY SILTY SAND				
PROJECT No.		11-1191-0008		FILE No.		1191-0008 DETAIL.GPJ			
DRAWN	JJL	Jan 2014		SCALE	N/A	REV.			
CHECK	AB	Jan 2014		FIGURE D3					
APPR	JMAC	Jan 2014							





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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NDC1	8	218.8
■	NDC1	12	212.7
▲	NDC2	6	220.8
+	NDC2	9	217.0
◆	NDC2	11	213.9

PROJECT

HIGHWAY 11
NAGAGAMI RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

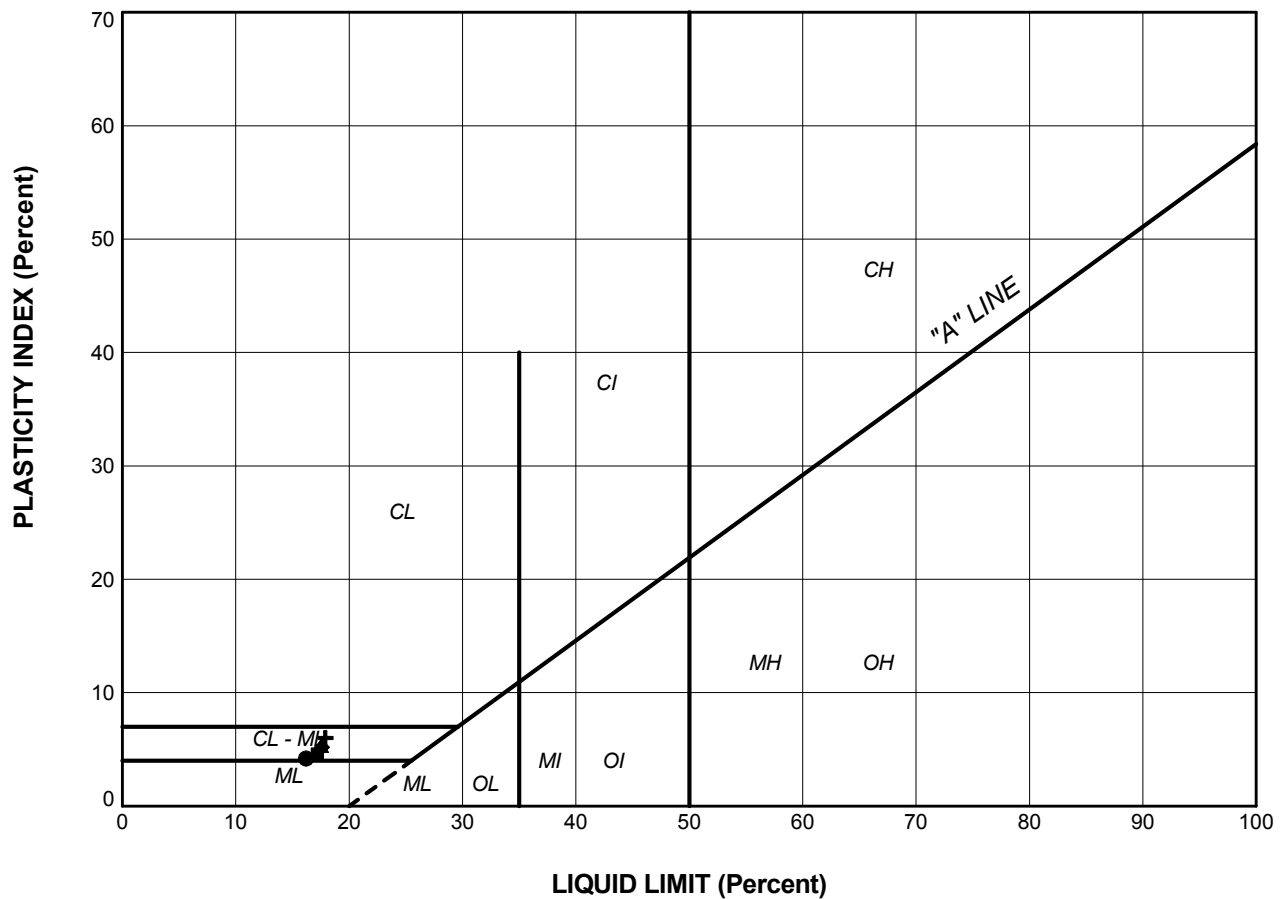
SILT and SAND (TILL)



Golder Associates
SUDBURY, ONTARIO


PROJECT No. 11-1191-0008		FILE No. 1191-0008 DETAIL.GPJ	
DRAWN	JJL	Jan 2014	SCALE N/A
CHECK	AB	Jan 2014	REV.
APPR	JMAC	Jan 2014	

FIGURE D4



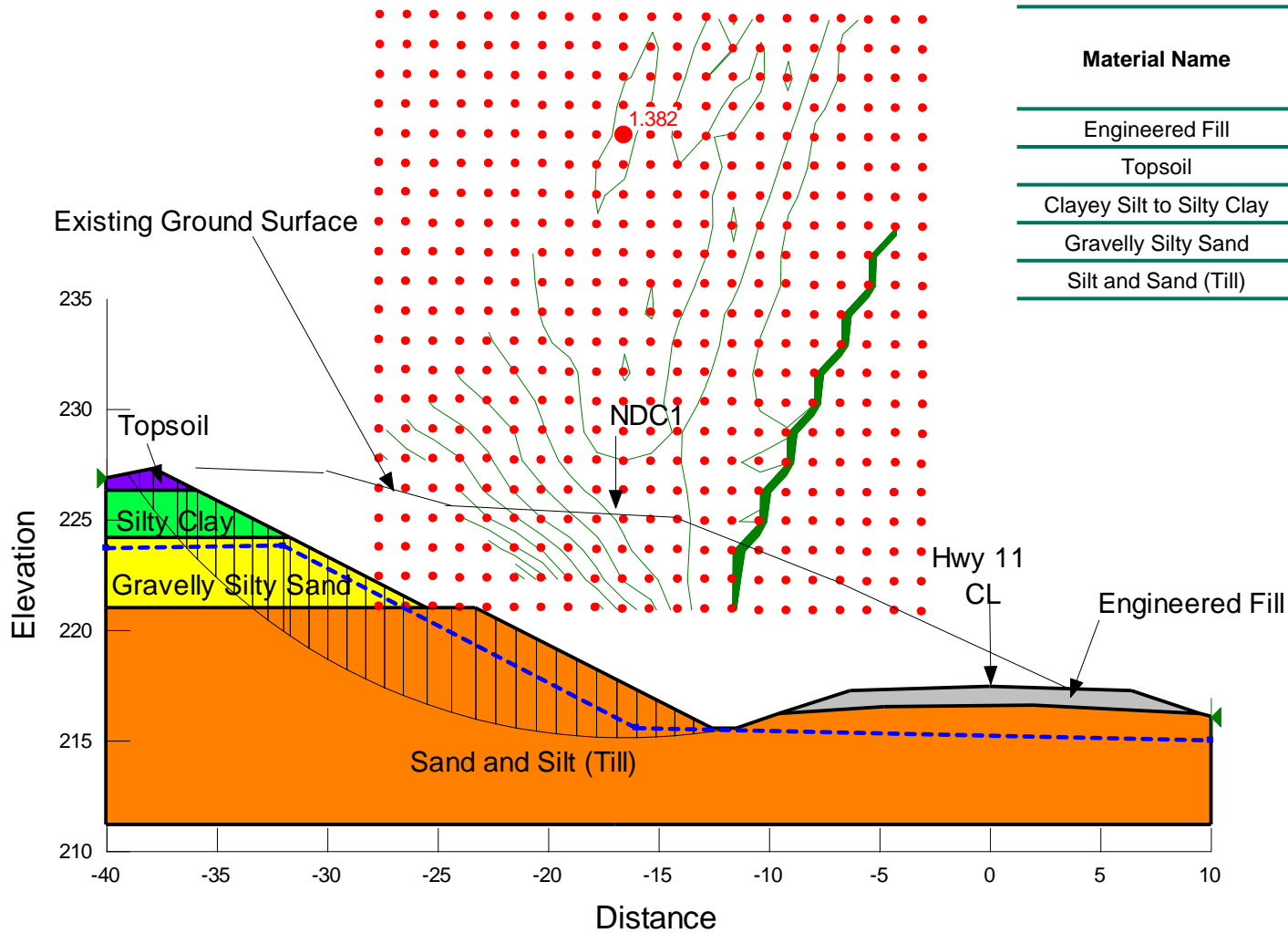
LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	NDC1	8	16.2	12.0	4.2
■	NDC1	12	17.2	12.6	4.6
▲	NDC2	6	17.5	12.2	5.3
+	NDC2	9	17.9	11.9	6.0
◆	NDC2	11	17.6	12.4	5.2

PROJECT					
HIGHWAY 11 NAGAGAMI RIVER BRIDGE					
TITLE					
PLASTICITY CHART SILT and SAND (TILL)					
PROJECT No. 11-1191-0008		FILE No. 11-1191-0008 DETAIL.GPJ			
DRAWN	JJL	Jan 2014	SCALE	N/A	REV.
CHECK	AB	Jan 2014			
APPR	JMAC	Jan 2014			
 Golder Associates SUDBURY, ONTARIO			FIGURE D5		

Highway 11 Nagagami River Deep Cut Slope Stability Analysis – 21+073

Figure D6

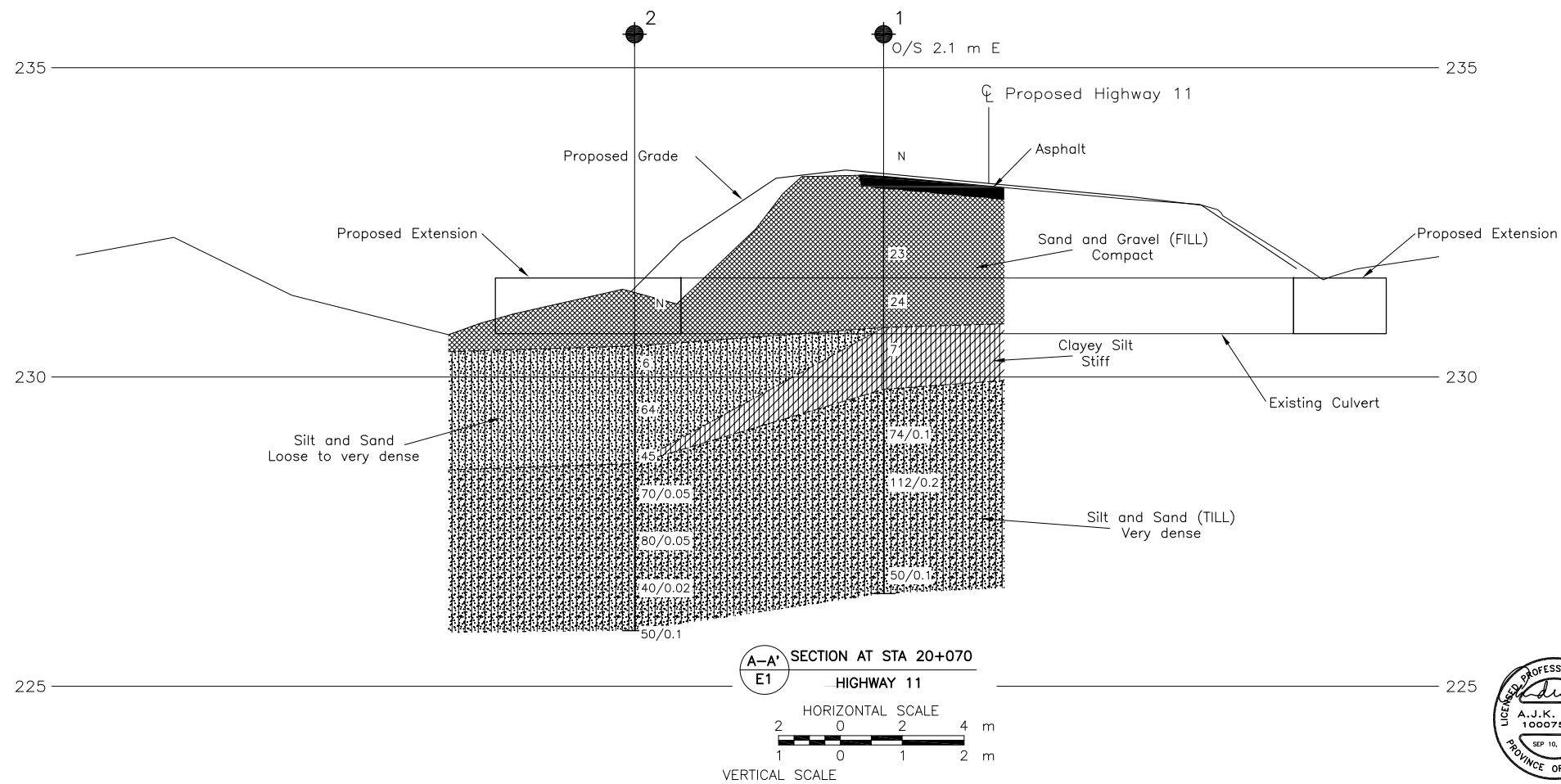


Material Name	Unit Weight (kN/m ³)	Friction Angle (°)	Cohesion (kPa)
Engineered Fill	21	35	-
Topsoil	12	27	1
Clayey Silt to Silty Clay	18	-	40
Gravelly Silty Sand	19	33	-
Silt and Sand (Till)	21	35	-



APPENDIX E

Culvert Extension – STA 20+070



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

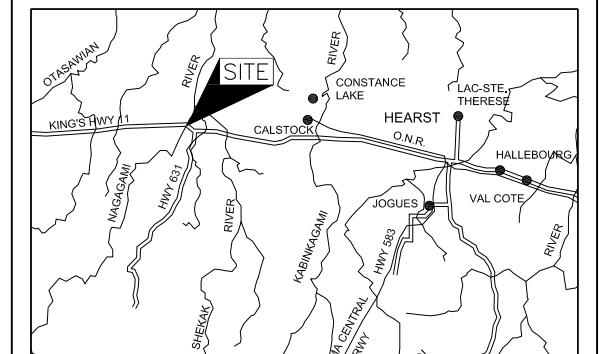
CONT No.
GWP No. 5307-04-00



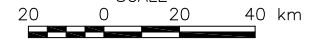
<p style="text-align: center;"> HIGHWAY 11 NAGAGAMI RIVER BRIDGE CULVERT EXTENSION - STA 20+070 BOREHOLE LOCATIONS AND SOIL STRATA </p>	
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Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN
SCALE



LEGEND

- | | |
|---|--|
|  | Borehole |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
|  | WL upon completion of drilling |

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
1	233.2	5515505.1	265379.6
2	230.7	5515513.4	265379.2

NOTES

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REFERENCE

Base plans provided in digital format by LEA, drawing file nos. 8960-Nag-S01.dwg received on May 28, 2013 and x8960 Nagagami Base.dwg, received Sept 25, 2013. Culvert profile provided by LEA, drawing file no. x8960 Naga Prof.dwg, received Nov 26, 2013.

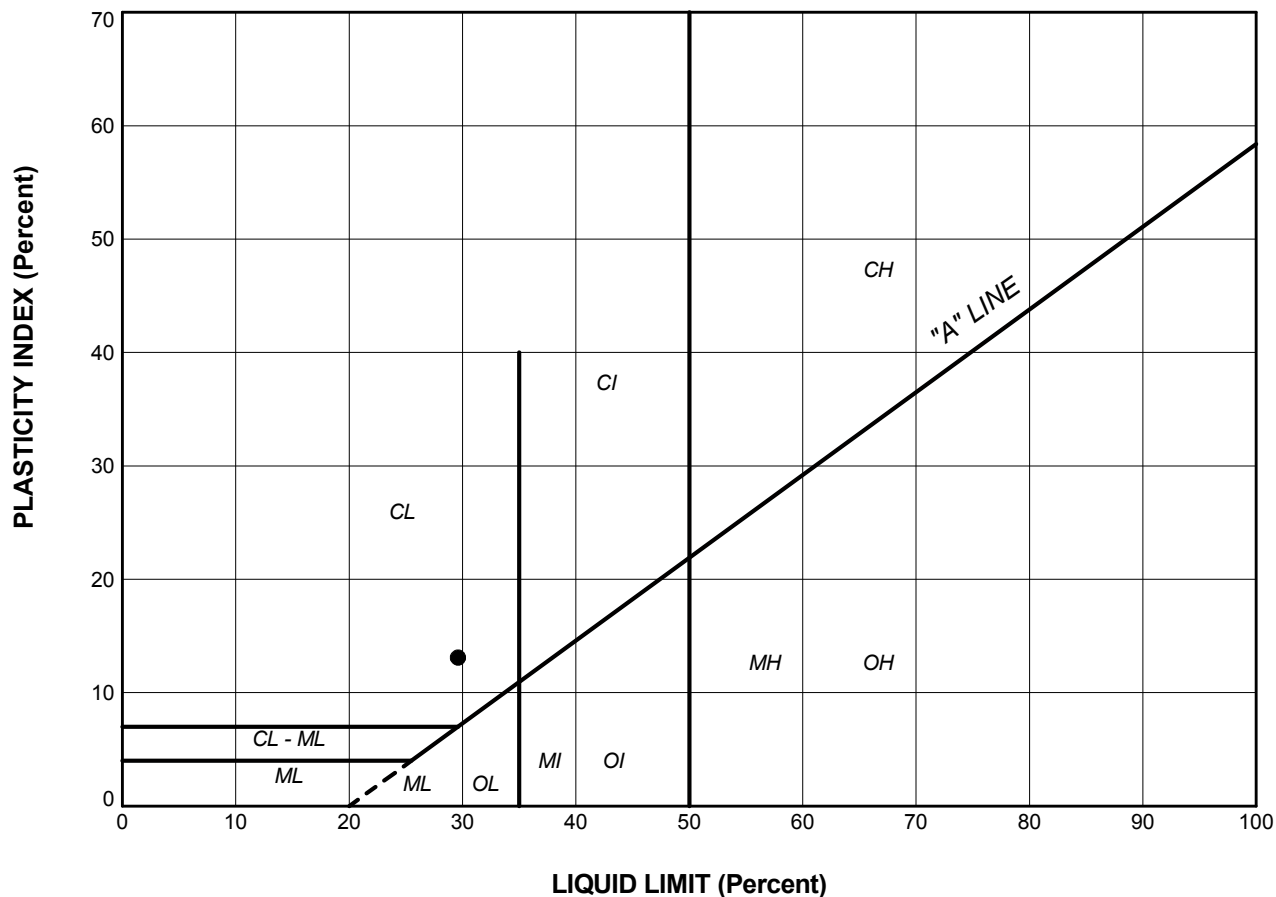


NO.	DATE	BY	REVISION		
Geocres No. 42F-28					
HWY. 11		PROJECT NO. 11-1191-000B		DIST.	
SUBM'D. AC	CHKD.	DATE: SEP 2014		SITE:	
DRAWN: TB	CHKD. AB	APPD. JMAG		DWG. E1	

PROJECT 11-1191-0008			RECORD OF BOREHOLE No 1			1 OF 1 METRIC											
G.W.P. 5307-04-00			LOCATION N 5515505.1; E 265379.6			ORIGINATED BY EHS											
DIST _____ HWY 11			BOREHOLE TYPE HW Casing, NW Casing and NQ Coring			COMPILED BY AC											
DATUM GEODETIC			DATE April 3, 2014			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60			
233.2	GROUND SURFACE																
0.0	ASPHALT						233										
0.3	Sand and gravel to gravelly sand, trace silt (FILL)																
232.4	Brown Frozen																
0.9	STYROFOAM (75 mm)		1	SS	23		232										
	Gravelly sand to sand, trace to some silt (FILL)																
	Compact Brown Moist		2	SS	24												
230.8							231										
2.4	CLAYEY SILT, trace sand		3	SS	7												1 10 54 35
	Stiff Brown Moist																
229.8							230										
3.4	SILT and SAND, trace to some clay, trace gravel (TILL)																
	Very dense Brown to grey at 4.7 m depth. Moist		4	SS	74/0.1												
							229										
			5	SS	112/0.2												
							228										
			6	SS	50/0.1		227										2 37 48 13
226.5																	
6.7	END OF BOREHOLE																
	Note: 1. Borehole dry upon completion of drilling.																

PROJECT 11-1191-0008				RECORD OF BOREHOLE No 2				1 OF 1 METRIC									
G.W.P. 5307-04-00				LOCATION N 5515513.4; E 265379.2				ORIGINATED BY EHS									
DIST _____ HWY 11				BOREHOLE TYPE NW Casing and Wash Boring				COMPILED BY AC									
DATUM GEODETIC				DATE April 7, 2014				CHECKED BY AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
230.7	GROUND SURFACE																
0.0	Sand, to some gravel, trace organics (FILL)		1	SS	6												
0.2	Grey Wet																
	Sandy SILT to SILT and SAND, trace to some gravel, trace clay (TILL)		2	SS	64												10 25 52 13
	Loose to very dense																
	Grey Moist		3	SS	45												
			4	SS	70/0.05												
			5	SS	80/0.05												3 35 47 15
		6	SS	40/0.02													
225.9	END OF BOREHOLE		7	SS	50/0.1												
4.8	Note: 1. Borehole dry upon completion of drilling.																

SUD-MTO 001 11-1191-0008 DETAIL GP J GAL-MISS.GDT 30/07/14 DATA INPUT:



SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

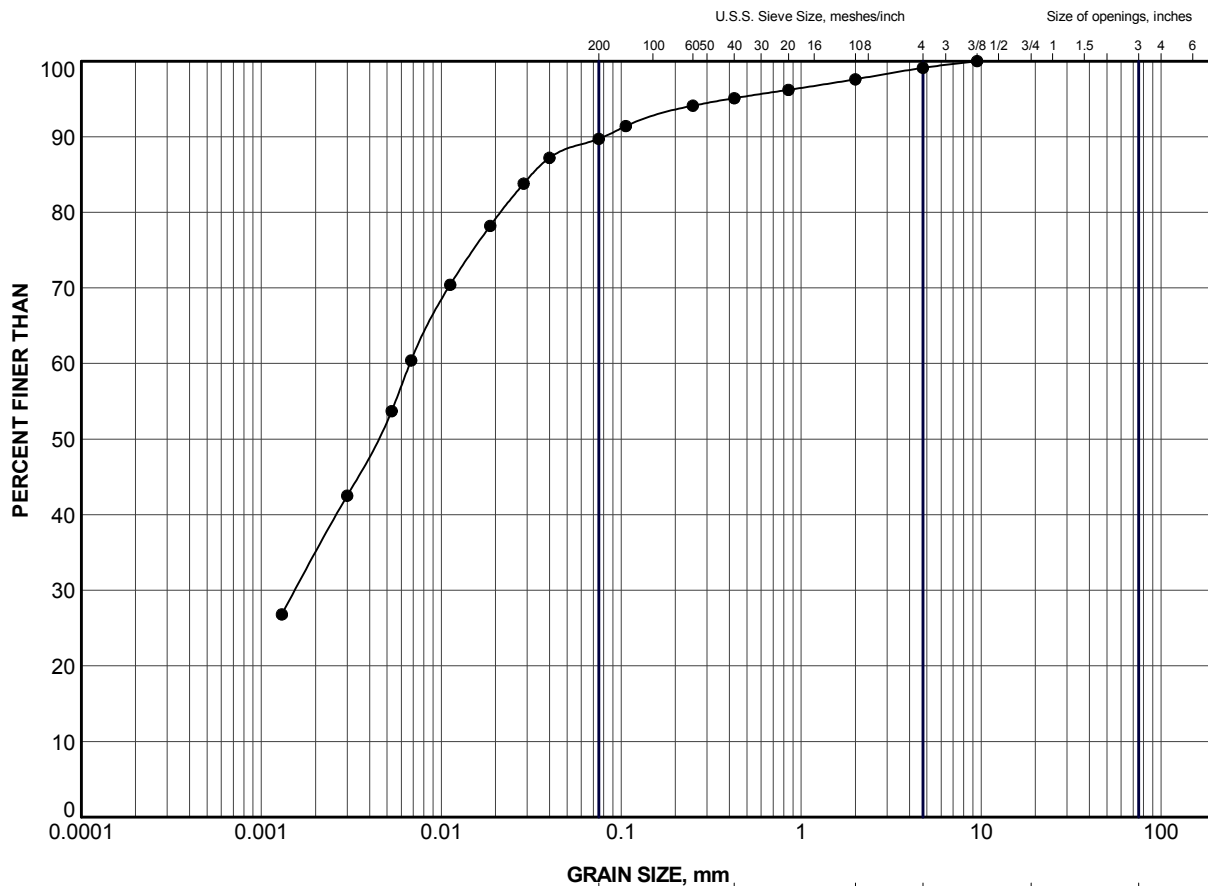
PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	1	3	29.6	16.5	13.1

PROJECT			HIGHWAY 11 NAGAGAMI RIVER BRIDGE CULVERT EXTENSION - STA 20+070		
TITLE			PLASTICITY CHART CLAYEY SILT		
PROJECT No.		11-1191-0008	FILE No. 11-1191-0008 DETAIL.GPJ		
DRAWN	TB	Apr 2014	SCALE	N/A	REV.
CHECK	AB	Apr 2014	FIGURE E1		
APPR	JMAC	Apr 2014			




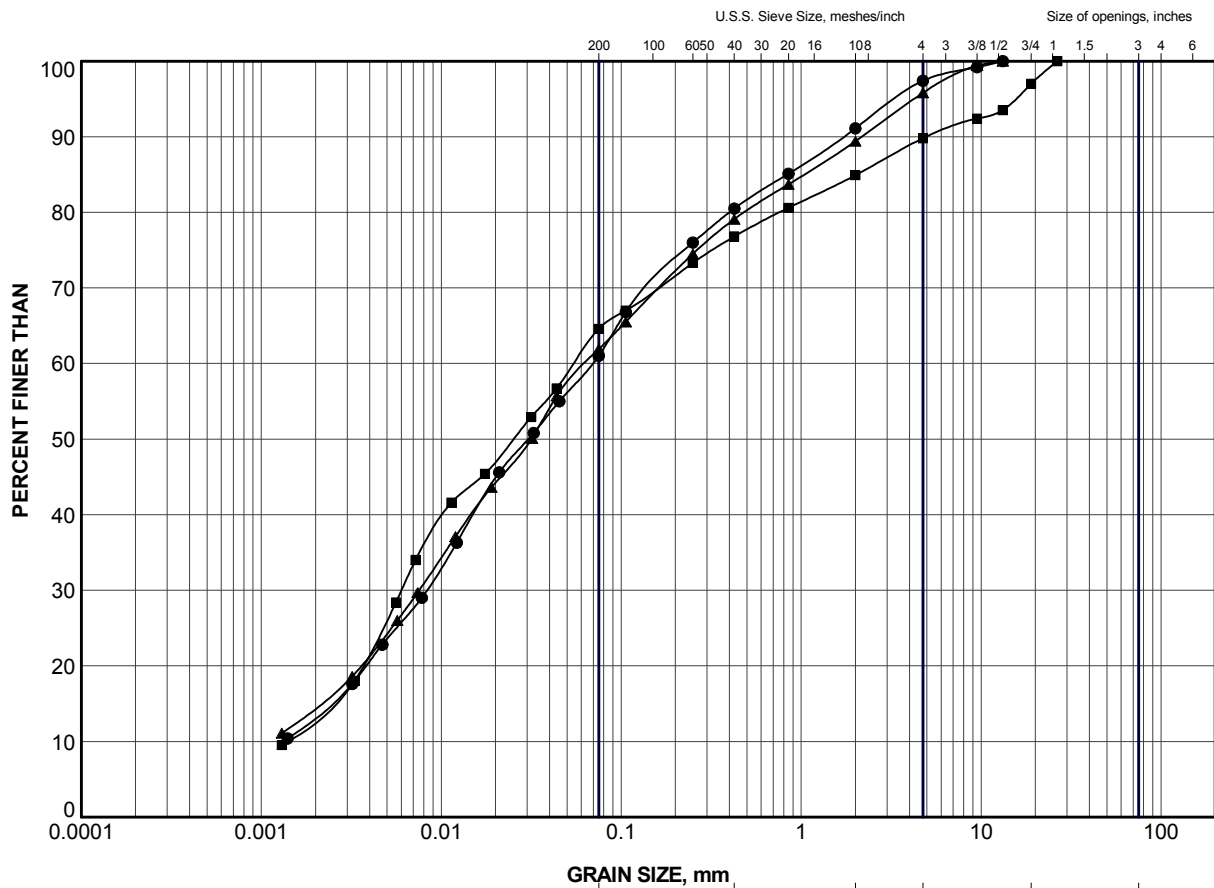


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	3	230.6


PROJECT	HIGHWAY 11 NAGAGAMI RIVER BRIDGE CULVERT EXTENSION - STA 20+070				
TITLE	GRAIN SIZE DISTRIBUTION CLAYEY SILT				
	PROJECT No.	11-1191-0008	FILE No. 1191-0008 DETAIL.GPJ		
	DRAWN	TB	Apr 2014	SCALE	N/A
	CHECK	AB	Apr 2014	REV.	
	APPR	JMAC	Apr 2014		
FIGURE E2					

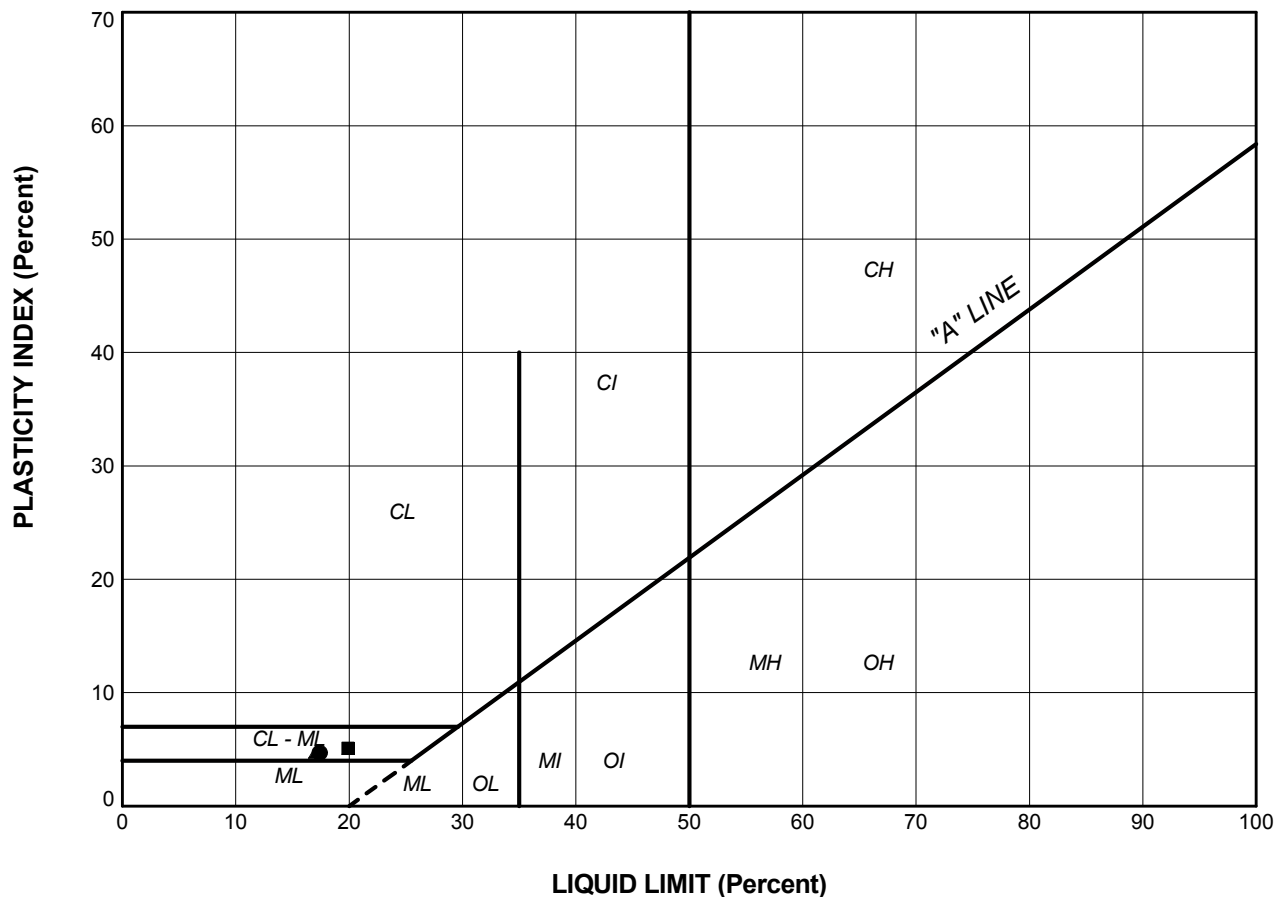


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	6	226.9
■	2	2	229.7
▲	2	5	227.5

PROJECT						HIGHWAY 11 NAGAGAMI RIVER BRIDGE CULVERT EXTENSION - STA 20+070											
TITLE						GRAIN SIZE DISTRIBUTION SANDY SILT to SILT and SAND (TILL)											
PROJECT No.						11-1191-0008						FILE No-1191-0008 DETAIL.GPJ					
DRAWN		JJL		Jul 2014		SCALE		N/A		REV.							
CHECK		AB		Jul 2014													
APPR		JMAC		Jul 2014								FIGURE E3					
 Golder Associates SUDBURY, ONTARIO																	



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	1	6	17.4	12.7	4.7
■	2	2	19.9	14.8	5.1
▲	2	5	17.1	12.3	4.8

PROJECT			HIGHWAY 11 NAGAGAMI RIVER BRIDGE CULVERT EXTENSION - STA 20+070		
TITLE			PLASTICITY CHART SANDY SILT to SILT and SAND (TILL)		
PROJECT No.		11-1191-0008	FILE No. 11-1191-0008 DETAIL.GPJ		
DRAWN	JJL	Jul 2014	SCALE	N/A	REV.
CHECK	AB	Jul 2014	FIGURE E4		
APPR	JMAC	Jul 2014			





APPENDIX F

Non-Standard Special Provisions

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:

<u>Criteria</u>	<u>Tolerance</u>
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.

H-PILES - Item No.

Non-Standard Special Provision

903.07.02 Driven Piles

903.07.02.01 Pile Driving Requirements and Restrictions

Section 903.07.02.01 of OPSS 903 is amended by the addition of the following:

The Contractor shall commence assessment of the ultimate axial resistance of the pile by the Hiley Formula (Standard Drawing SS103-11) once the pile reaches a depth of 3.0 m above the design pile tip elevation shown in the Contract Drawings and at subsequent 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate axial resistance as determined by the Hiley Formula is not achieved within the 3.0 m interval down to the design pile tip elevation the Contractor shall stop pile driving and notify the Contract Administrator. At this depth the pile should be allowed to rest for 48 hours, and the Hiley Formula shall then be applied immediately upon re-striking of the pile. If the ultimate axial resistance is still not achieved after the 48 hour wait period, the Contract Administrator shall be notified and authorization given prior to driving the pile below the design pile tip elevation.

The contractor shall have materials and equipment available on site to deal with varying pile lengths as the pile tip elevation (and hence length of pile) will depend on achieving the required geotechnical axial resistance as specified in the contract.

WORKING SLAB – Item No.

Non-Standard Special Provision

Scope of Work

This Special Provision covers the requirements for the supply and placement of a concrete working slab on a soil subgrade under the structure foundations for the Nagagami River Bridge. The purpose of the working slab is to protect the subgrade from disturbance and loosening due to construction traffic and ponded water and also to provide a level working surface.

Construction

Protection of Founding Soil

- Following inspection and approval of the prepared soil subgrade by the Quality Verification Engineer, a working slab, with a minimum thickness of 100 mm shall be placed on the foundation subgrade as per the contract drawings and documents. The concrete shall have a minimum 28 day compressive strength of 20 MPa.

Unwatering carried out for the footing or pile cap excavation shall be done in such a manner as to prevent any disturbance to the surrounding original soil.

Basis of Payment

Payment at the Contract Price for the above tender item includes full compensation for all labour, equipment and material to do the required work.

GROUNDWATER CONTROL - Item No.

Non-Standard Special Provision

Construction of foundation footings and/or pile caps of the abutments and pier at the Nagagami River Bridge site will require excavations to extend below the groundwater level. The compact to very dense cohesionless soils that are present below the groundwater table will slough, run, boil or cave into the excavation unless appropriate groundwater controls are in place. The Contractor is to design and install an appropriate unwatering system to enable construction in dry conditions, to prevent disturbance to the founding soils.

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

OBSTRUCTIONS

Non-Standard Special Provision

The Contactor is hereby notified that cobbles and boulders are present on the bottom of the Nagagami River and present within the fill material at the site. Further, the native soils at the site of the Nagagami River 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

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

For more information, visit golder.com

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Ltd.
1010 Lorne Street
Sudbury, Ontario, P3C 4R9
Canada
T: +1 (705) 524 6861

