

**FOUNDATION INVESTIGATION AND DESIGN REPORT
LITTLE WABIGOON RIVER BRIDGE REPLACEMENT
HIGHWAY 17
BORUPS CORNERS, ONTARIO
UNORGANIZED KENORA DISTRICT**

G.W.P. 470-00-00, SITE No. 41S-67

Geocres Number: 52F-39

Report to

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TABLE OF CONTENTS

PART 1 FACTUAL INFORMATION

1	INTRODUCTION	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION AND FIELD TESTING	2
4	LABORATORY TESTING	4
5	DESCRIPTION OF SUBSURFACE CONDITIONS	5
5.1	Pavement structure.....	5
5.2	Fill.....	5
5.3	Sand and Silt	7
5.4	Clayey Silt to Silty Clay	8
5.5	Silt.....	9
5.6	Bedrock.....	10
5.7	Water Levels	11
6	MISCELLANEOUS	12
7	GENERAL.....	14
8	STRUCTURE FOUNDATIONS.....	15
8.1	Spread Footings on Native Soils.....	15
8.2	Augered Caissons (drilled shafts)	16
8.3	Steel H-Piles driven to Bedrock.....	16
8.3.1	Pile Tips	18
8.3.2	Pile Installation	18
8.3.3	Pile Driving to Bedrock	18
8.3.4	Downdrag.....	18
8.3.5	Lateral Resistance	19
8.4	Recommended Foundation	21
8.5	Depth of Frost Penetration	21
9	SHEET PILE WALLS.....	21
10	EXCAVATION AND GROUNDWATER CONTROL	23
11	APPROACH EMBANKMENTS	24
11.1	Slope stability.....	24
11.2	Settlement	25

12	EROSION PROTECTION	26
13	BACKFILL TO ABUTMENTS	26
14	ROADWAY PROTECTION.....	26
15	SEISMIC CONSIDERATIONS.....	27
15.1	Seismic Design Parameters.....	27
16	CONSTRUCTION CONCERNS	28
17	CLOSURE	29

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Site Photographs
Appendix D	Foundation Comparison
Appendix E	Slope Stability Output
Appendix F	List of SPs and OPSS, and Suggested Text for NSSP
Appendix G	Borehole Locations and Soil Strata Drawings

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of a bridge at the Highway 17 crossing of the Little Wabigoon River southeast of the Borups Corners, Ontario, in the Unorganized Kenora District.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, a stratigraphic profile, cross sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to Hatch Mott MacDonald, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0010.

2 SITE DESCRIPTION

The Little Wabigoon River Bridge is located on Highway 17, approximately 6.5 km southeast of the intersection of Highway 17 and Highway 603, known as Borups Corners, in the Unorganized Kenora District in Ontario. Dryden is located approximately 48 km west of the site and Ignace is located approximately 45 km east of the site.

The existing Little Wabigoon River Bridge is a five span structure supported on timber piles. The bridge is 12.0 m wide and 32.0 m long. The existing embankments are about 4.5 m to 5.0 m high.

The surrounding lands are undeveloped and heavily treed. Bedrock outcroppings and small creeks/water bodies are visible on both sides along the existing Highway 17. The Little Wabigoon River flows from south to north.

Selected photographs in Appendix C show the general nature of the surrounding lands and the existing bridge structure. Photos of the site show the presence of rock fill, cobbles and boulders on the forward and side slopes below the existing abutments. It is not confirmed if this rockfill, cobbles and boulders is for erosion control purposes or whether the embankments contain rockfill.

The region is characterized by massive to foliated granodiorite to granite intruded by later stage mafic dykes. The bedrock is mantled by sand and silt layers and extensive deposits of silty clay to clayey silt and silt.

3 SITE INVESTIGATION AND FIELD TESTING

The field investigation program was designed to cover both alternatives, a single span and a three span structure.

The site investigation and field testing for this project was carried out on July 13, August 10 to 13, September 15 to 23 and October 22, 2011 and consisted of drilling and sampling eleven boreholes (numbered LWR-01 to LWR-10 and LWR-6B) in the area of the existing west and east approaches, potential abutment and pier locations. Boreholes LWR-01 and LWR-10 were drilled near the west and east approaches, respectively. Boreholes LWR-2 to LWR-5 were drilled near the west abutment and west pier and Boreholes LWR-06 to LWR-09 and LWR-06B were drilled near the east abutment and east pier. Boreholes LWR-01 to LWR-03 and LWR-08 to LWR-10 were drilled Highway 17 surface, through the existing embankments. Boreholes LWR-04 to LWR-07 were drilled below the bridge deck near the abutments, at the toe of the highway embankment.

Boreholes were advanced to depths ranging from 8.1 m to 22.5 m (elevations 370.5 to 385.0). Borehole LWR-06 was terminated at 3.7 m depth (elevation 385.1). Borehole LWR-05 was terminated at 11.6 m depth (elevation 376.6) and a Dynamic Cone Penetration Test (DCPT) was extended below this depth to 14.2 m (elevation 373.9). The boreholes were generally terminated upon refusal on bedrock or boulders.

Boreholes LWR-03, LWR-04, LWR-07, LWR-07B, LWR-08 were advanced 2.9 m to 3.6 m into bedrock by NQ size diamond coring.

The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

Drilling on the highway shoulders was carried out using a truck-mounted CME 75 drill rig and the boreholes were advanced with hollow-stem augers and NQ coring techniques. The drilling at the toe of the embankment was carried out using wash-boring methods with casing and tripod. Portable split spoon sampling equipment driven with a Standard SPT hammer was used for penetration testing in the boreholes at the toe of the embankment. In general, samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the fill and native soils.

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

Rock cores were logged, and the Total Core Recovery (TCR), Fracture Index (FI) and Rock Quality Designation (RQD) were determined.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers consisting of 19 mm PVC pipe with slotted screen were installed in Boreholes LWR-02, LWR-05 and LWR-09 and enclosed in filter sand to permit longer term groundwater level monitoring. The boreholes were backfilled with bentonite holeplug in general accordance with O.Reg. 903 upon completion. The location and completion details of the piezometer and boreholes are shown in Table 3.1.

Table 3.1 – Borehole Abandonment Details

Foundation Unit	Borehole	Piezometer Tip Depth/ Elevation (m)	Abandonment Details
West approach	LWR-01	None installed	Borehole backfilled with holeplug to 2.5 m, auger cuttings to 0.3m, then concrete to surface.
West abutment	LWR-02	15.4/377.7	Sand from 15.4 m to 13.2 m, holeplug from 13.2 m to 2.2 m, auger cuttings from 2.2 m to 0.6 m, sand and gravel from 0.6 m to 0.1 m, then asphalt to surface.
	LWR-03	None installed	Borehole backfilled with holeplug to 2.1 m, sand to 0.3m, then concrete to surface.
	LWR-04	None installed	Borehole backfilled with holeplug to surface.
	LWR-05	9.8/378.3	Sand from 9.8 m to 7.3 m, then holeplug to surface.
East abutment	LWR-06	None installed	Borehole backfilled with holeplug to surface.
	LWR-06B	None installed	Borehole backfilled with holeplug from 14.1 m to 4.3m, then concrete from 0.3 m to surface.
	LWR-07	None installed	Borehole backfilled with holeplug from 13.5 m to 4.2 m, auger cuttings, from 4.2 m to 0.3 m, and concrete from 0.3 to surface.
	LWR-08	None installed	Borehole backfilled with holeplug to 1.5 m, sand to 0.3 m, then concrete to surface.
	LWR-09	12.7/380.3	Sand from 12.7 m to 9.4 m, holeplug from 9.4 m to 1.8 m, auger cuttings from 1.8 m to 0.6 m, sand from 0.6 m to 0.3 m, then concrete to surface.
East approach	LWR-10	None Installed	Borehole caved to 7.1 m, then backfilled with holeplug to 1.5 m, sand from 1.5 m to 0.3 m, then concrete to surface.

The piezometers will be decommissioned in accordance with O. Reg. 903 prior to the end of 2012.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and natural moisture content determination and rock samples to geological logging. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer) and Atterberg Limits testing

where appropriate. The results of this testing program are summarized on the Record of Borehole sheets included in Appendix A and on the figures presented in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the rock core samples are included in Appendix B and on the Record of Borehole sheets in Appendix A (as average unconfined compressive strength per run).

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing included in Appendix G. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

In general terms, the overburden stratigraphy at this site consists of pavement structure overlying various layers of embankment fill (sand, silt, sand and gravel with boulders and sandy silt to silty sand). Cobbles and boulders and rock protection were observed at the toe of the embankments, below the abutments. A layer of native compact to loose sand and silt was encountered below the fill. Layers of soft to very stiff clayey silt to silty clay and loose to dense silt were encountered below the sand and silt. Below the native soils, slightly weathered to fresh grey granodiorite to granite bedrock was contacted. More detailed descriptions of the individual strata are presented below.

5.1 Pavement structure

Pavement structure was encountered in the boreholes drilled through the existing Highway 17 shoulders. The pavement structure consists of approximately 75 mm to 150 mm of asphalt overlying granular fill. A layer of approach slab concrete, 325 mm thick, was encountered below the asphalt in boreholes LWR-02, LWR-03, LWR-08 and LWR-09. In Borehole LWR-06B, the concrete was 400 mm thick.

5.2 Fill

Fill was encountered below the pavement structure in the boreholes drilled on Highway 17 shoulders and surficially in boreholes drilled at the toe of the embankments, below the abutments.

Based on soil composition, the fill comprising the existing highway embankment, consisted of the following various soil types:

- Brown sand fill containing trace to some gravel, some silt to silty, trace clay and occasional asphalt fragments and roots, immediately below the asphalt.

- Brown silt fill containing some clay to clayey, trace sand and trace gravel.
- A layer of sandy silt to silty sand fill with organics and wood fragments was encountered below the silt fill at 2.8 m and 2.7 m depth (elevations 390.3 and 390.4) in Boreholes LWR-01 and LWR-03, respectively. A thin layer of clayey silt fill was also encountered within this sandy silt/silty sand fill.
- Brown to grey sand and gravel fill with boulders and cobbles were encountered from the surface in Boreholes LWR-04 to LWR-07, drilled below the abutments and this fill is visible at the toe of the forward and side slopes, below the existing abutments, as shown in photographs in Appendix C. The cobbles, boulders and possible rockfill on the embankment surface are for erosion protection purposes.

In general, the thickness of the fill layers forming the highway embankment (boreholes drilled from the top of the highway) ranged from 1.2 m to 3.8 m. The depth to the base of the fill ranged from 1.4 m to 3.9 m (elevations 389.2 to 391.7).

Fill encountered in the boreholes drilled at the toe of the embankment (below the abutments) ranged in thickness from 0.6 m to 2.0 m. The lower boundary of Boreholes LWR-04 to LWR-07 ranged from 0.6 m to 2.0 m depth (elevations 386.8 to 387.5).

SPT N-values recorded in the fill ranged from 4 to 32 blows per 0.3 m of penetration, indicating generally a loose to compact relative density. An SPT N-value of 40 blows per 0.3 m of penetration was measured in Borehole LWR-08 near elevation 391.4, indicating a dense relative density.

The moisture content of the fill samples ranged from 4% to 28%.

Grain size distribution curves for samples of fill tested are presented on the Record of Borehole sheet and on Figures B1 to B3 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Silt Fill (%)	Sand Fill (%)	Clayey Silt Fill (%)
Gravel	0	9	0
Sand	2	65	7
Silt	83	23	71
Clay	15	3	22

5.3 Sand and Silt

In Boreholes LWR-01 to LWR-03, drilled from the top of the embankment, a layer of native brown to grey sand and silt containing some gravel and trace clay was contacted below the fill at depths ranging from 3.4 m to 3.9 m (elevations 389.2 to 389.7). In Borehole LWR-10, also drilled from the top of the highway embankment, this layer was contacted below the clayey silt at 6.1 m depth (elevation 387.0). The thickness of the sand and silt layer ranged from 2.0 m to 4.5 m. The depth to the base of the sand and silt varied from 7.2 m to 8.1 m (elevations 385.9 to 385.0).

In Boreholes LWR-04 to LWR-07, drilled at the toes of the embankments slope, the sand and silt layer was contacted at depth ranging from 0.6 m to 2.0 m (elevations 386.8 to 387.5). In Borehole LWR-06B, also drilled at the toe of the embankment, the sand and silt layer was contacted at 5.5 m depth (elevation 387.5). The thickness of the sand and silt ranged from 1.0 m to 3.1 m. The depths to the base of the sand and silt layer were 1.8 m and 3.7 m in Boreholes LWR-04 and LWR-05 (elevations 386.1 to 384.5), respectively. The depth to the base of the silt and sand was at 8.1 m depth (elevation 384.9) in Borehole LWR-6B. Borehole LWR-06 was terminated within the sand and silt at 3.7 m depth (elevation 385.1).

SPT N-values recorded in the sand and silt ranged from 5 to 20 blows per 0.3 m of penetration, indicating a loose to compact relative density. An SPT N-value of 32, indicating a dense relative density was measured in Borehole LWR-06 near elevation 385.5. An SPT N-value of 70 blows per 0.25 m of penetration, indicating a very dense relative density, was measured in Borehole LWR-10 near borehole termination depth at elevation 385.3.

The moisture content of the sand and silt samples ranged from 20% to 30%. A moisture content of 82% was measured in Borehole LWR-04.

Grain size distribution curves for samples of the sand and silt layer tested are presented on the Record of Borehole sheet and on Figure B4 and B5 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	Sand and Silt
Gravel	0 to 12
Sand	23 to 56
Silt	38 to 69
Clay	3 to 12

5.4 Clayey Silt to Silty Clay

Native brown to grey clayey silt to silty clay containing trace sand and gravel was encountered at various depths in most of the boreholes. The depths, elevations and thicknesses corresponding to the silty clay/clayey silt layers encountered in the boreholes are listed in Table 5.1.

Table 5.1 – Depths, Elevations and Thicknesses of Clayey Silt to Silty Clay Layers

Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
LWR-01	7.2 to 11.3	386.0 to 381.9	4.1
LWR-02	7.8 to 12.8	385.3 to 380.3	5.0
LWR-03	7.9 to 12.7	385.2 to 380.4	4.8
LWR-04	7.3 to 9.6	380.6 to 378.3	2.3
LWR-06B	12.6 to 13.8	380.3 to 379.2	1.2
LWR-07	3.0 to 5.5	385.7 to 383.3	2.5
	9.1 to 9.9	379.7 to 378.9	0.8
LWR-09	3.7 to 9.9	389.3 to 383.1	6.2
LWR-10	1.4 to 6.1	391.7 to 387.0	4.7

Standard Penetration tests performed in the clayey silt/silty clay layers gave SPT N-values ranging from 2 to 26 blows per 0.3 m of penetration, indicating a very soft to very stiff consistency. Higher SPT N-values ranging from 23 to 31 blows for 0.3 m penetration were recorded in Borehole LWR-04, indicating a very stiff to hard consistency.

In-situ vane shear tests were carried out to assess the undrained shear strength of the soft to firm cohesive deposits. Shear strengths results were 35 kPa, 38 kPa and 80kPa. Based on remoulded shear vane tests, the clayey silt/silty clay had a Sensitivity Value of 2 to 3.

The moisture contents of samples of the clayey silt/silty clay layers typically ranged from 19% to 38%. Moisture contents of 79% and 54% were obtained from samples taken near elevations 382.3 to 382.7 in Boreholes LWR-02 and LWR-03. In Borehole LWR-07, a moisture content of 47% was encountered near elevation 384.4.

Selected samples of the clayey silt/silty clay underwent laboratory grain size analysis testing and Atterberg Limits tests. The grain size distribution curves for tested samples of clayey silt/silty clay are presented in Appendix B, Figures B6 and B7. The results of the Atterberg Limits tests are presented in Figure B11, Appendix B. The results are also summarized on the Record of Borehole sheets included in Appendix A. The results of the laboratory tests are summarized as follows:

Soil Particles	Clayey silt to silty clay (%)
Gravel	0 to 1
Sand	0 to 4
Silt	46 to 79
Clay	20 to 54

Index Property	Clayey silt (%)	Silty clay (%)
Liquid Limit	29	68
Plastic Limit	20	22

The above results indicate that the clayey silt is of low plasticity with a group symbol of CL and the silty clay is of high plasticity with a group symbol of CH.

5.5 Silt

Native reddish brown to grey silt was encountered at various depths in most of the boreholes, except in Boreholes LWR-01, LWR-06 and LWR-10. The depths, elevations and thicknesses corresponding to the silt layers encountered in each borehole are list in Table 5.2.

Table 5.2 – Depths, Elevations and Thicknesses of Silt

Borehole	Depth below existing ground surface (m)	Elevation (m)	Thickness (m)
LWR-02	12.7 to 17.4	380.4 to 375.6	4.7
LWR-03	12.8 to 19.4	380.3 to 373.6	6.6
LWR-04	1.8 to 7.3	386.1 to 380.6	5.5
LWR-05	3.7 to 11.6	384.5 to 376.6	7.9
LWR-06B	8.1 to 12.6	384.9 to 380.3	4.5
LWR-07	5.5 to 9.1	383.3 to 379.7	3.6
LWR-08	3.7 to 10.2	389.3 to 382.8	6.5
LWR-09	9.9 to 12.7	383.1 to 380.3	2.8

Standard Penetration tests performed in the silt layers gave SPT N-values ranging from 6 to 41 blows per 0.3 m of penetration, indicating a loose to dense relative density. A low SPT N-value of 2 blows per 0.3 m of penetration, indicating very loose relative density, was measured in Borehole LWR-08, near elevation 388.7

The moisture contents of samples of the silt layers typically ranged from 7% to 41%. In Borehole LWR-04 moisture contents of 59% and 52% were noted near elevations 384.2 to 384.8 and 65% in Borehole LWR-05 near elevation 383.1.

Selected samples of the silt underwent laboratory grain size analysis testing and one sample was subjected to Atterberg Limits test. The grain size distribution curves for tested samples of silt are presented in Appendix B, Figures B8 to B10. The results of the Atterberg Limits test are presented in Figure B12, Appendix B. The results are also summarized on the Record of Borehole sheets included in Appendix A. The results of the laboratory tests are summarized as follows:

Soil Particles	Clayey silt to silty clay (%)
Gravel	0
Sand	0 to 17
Silt	70 to 96
Clay	4 to 13

Index Property	Clayey silt (%)
Liquid Limit	23
Plastic Limit	16

The above results indicate that the silt is of low plasticity with a group symbol of CL.

5.6 Bedrock

The native soils described above are underlain by granodiorite to granite bedrock. The bedrock is slightly weathered to fresh and grey in colour with white bands. Occasional mechanical breaks and sub-vertical fractures were observed in the rock cores. The borehole data indicates that the bedrock slopes down across the site from southeast to northwest.

Table 5.3 summarizes depths and elevations to the top of bedrock or auger refusal in the boreholes and DCPT.

Table 5.3 – Depths and Elevations of Top of Bedrock or Auger Refusal

Foundation Unit	Borehole/DCPT	Top of Bedrock or Auger Refusal	
		Depth (m)	Elevation (m)
West abutment	LWR-02 ⁽²⁾	17.4	375.7
	LWR-03 ^(1,2)	19.4	373.7
	LWR-04 ^(1,3)	9.6	378.3
	LWR-05/DCPT ⁽³⁾	14.2	373.9
East abutment	LWR-06B ⁽³⁾	9.5	379.2
	LWR-07 ^(1,3)	9.9	378.9
	LWR-08 ^(1,2)	10.2	382.8
	LWR-09 ⁽²⁾	12.7	380.2
East approach	LWR-10 ⁽²⁾	8.1	385.0

⁽¹⁾Bedrock proved by coring

⁽²⁾Borehole drilled from the top of highway embankment

⁽³⁾Borehole drilled from the toe of highway embankment (below the abutments)

The Total Core Recovery (TCR) in the bedrock ranged from 92% to 100%. The RQD value ranged from 72% to 97% indicating a fair to excellent rock quality.

The estimated unconfined compressive strength of the rock core ranged from 169 MPa to 389 MPa, indicating a very strong to extremely strong rock. An estimated compressive strength of 68 MPa was measured in Borehole LWR-03 Run 1, indicating a strong rock. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from one borehole. A summary of the Point Load Test Results is presented in Appendix B.

5.7 Water Levels

Water levels were observed in the boreholes during and upon completion of drilling. Three standpipe piezometers were installed in Boreholes LWR-02, LWR-05 and LWR-09 to monitor water levels after completion of drilling. The water levels measured in the piezometer are summarized in Table 5.4, along with the measurements in the boreholes upon completion of drilling.

Table 5.4 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
LWR-02	August 17, 2011	5.0	388.1	Piezometer
	September, 15, 2011	5.5	387.6	
	October 11, 2011	5.0	388.1	
LWR-05	September 23, 2011	2.0	386.1	Piezometer
LWR-06B	September 10, 2011	0.5	388.2	Open Borehole
LWR-07	October 10, 2011	4.7	384.1	Open borehole
LWR-09	August 17, 2011	4.2	388.8	Piezometer
	September, 15, 2011	4.4	388.6	
	October 11, 2011	4.1	389.0	

The piezometric reading of the current investigation indicates that the groundwater level is at 2.0 m to 4.4 m depth (elevations 386.1 to 389.0).

The GA drawing indicates that water level in the Little Wabigoon River was at elevation 387.7 on May 12, 2011.

The above values are short-term readings and of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. Surveyors retained by Hatch Mott MacDonald provided the co-ordinates and the ground surface elevations for the boreholes.

Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied a truck mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations.

OGS Drilling Inc. of Almonte, Ontario supplied the portable drilling/coring equipment to drill and core boreholes that were not accessible using a truck mounted rig.

Overall supervision of the field program was conducted by Mr. Mark Farrant, P. Eng. Interpretation of the data and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

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Review Principal



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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for design of a new bridge to replace the existing bridge that carries Highway 17 over Little Wabigoon River. The site is approximately 6.5 km southeast of the intersection of Highway 17 and Highway 603, known as Borups Corners in the Unorganized Kenora District in Ontario.

The existing Little Wabigoon River Bridge is a five span structure supported on timber piles. The bridge is 32.0 m long and 12.0 m wide. The existing embankments are approximately 4.5 m to 5.0 m high.

Based on the General Arrangement (GA) drawing provided by Hatch Mott MacDonald, the proposed bridge consists of a two lane, single span structure with a deck of precast prestressed voided concrete girders supported on a single row of driven steel H-piles. A sheet pile wall will be driven just behind the H-piles to retain the approach fill. The proposed length of the bridge is 25.0 m with a width of 12.9 m. It is anticipated that the replacement structure will be constructed along the existing horizontal alignment. Hatch Mott MacDonald have indicated that highway grade raise of 0.8 m and 0.5 m is required at the east and west abutments, respectively.

The discussions and recommendations presented in this report are based on the factual data obtained during the course of the investigation. The plans and profiles used for preparation of this report were provided by Hatch Mott MacDonald.

8 STRUCTURE FOUNDATIONS

The stratigraphy encountered in the boreholes drilled at the east and west approaches and abutments revealed surficial pavement structure overlying various layers of fill (loose to compact sand, silt, sand and gravel with cobbles/boulders and sandy silt/silty sand). Cobbles, boulders and rock protection were observed on the toes of the forward and side slopes below the existing abutments. A layer of native compact to loose sand and silt was encountered below the fill. Layers of native soft to very stiff clayey silt to silty clay and loose to dense silt were also encountered at various depths at this site. Slightly weathered to fresh grey granodiorite to granite bedrock and auger refusal on probable bedrock or boulders were encountered at depths ranging from 8.1 m to 19.4 m (373.7 to 385.0). Borehole data indicates that, at this site, the bedrock slopes from southeast to northwest.

The piezometric readings from the current investigation indicate that the groundwater level is at 2.0 m to 4.4 m depth (elevations 386.1 to 389.0).

The GA drawing indicates that water level in the Little Wabigoon River was at elevation 387.7 on May 12, 2011.

Based on existing site conditions, consideration was given to the following foundation types:

- Spread footings on native soils
- Augered Caissons (drilled shafts)
- Steel H-piles driven to bedrock

These foundation alternatives are discussed in the following sections.

A comparison of the technical advantages and disadvantages of alternate foundation schemes is presented in Appendix D. A foundation scheme preferred from a foundations perspective is recommended.

8.1 Spread Footings on Native Soils

Consideration was given to supporting the structure on spread footings founded on native soils, however this option is not recommended due to the following reasons:

- Low geotechnical capacities are present at this site in the overburden soils (native sand and silt) below the existing fill.

- Settlements under footing loads will occur if footings are placed on the native soils.
- The depth of excavation required to place the footings on competent native soils is not practical.
- Spread footings could be subject to erosion or undermining/scour during high river flows.
- Temporary footing excavations have environmental impact on the river.

In light of the above factors, the spread footings option was not further developed.

8.2 Augered Caissons (drilled shafts)

Augered caisson foundations were also considered for supporting the structure at this site.

However, the caissons would have to be founded on bedrock which was contacted at depths ranging from 9.5 m to 19.4 m at both approaches. The extensive depth to reach bedrock through cohesionless deposits under water table makes this a difficult and expensive option.

Additionally, the permeable nature of the overburden soils (sand and silt) above the bedrock would make it difficult to seal the bottom of the caisson liner into the founding stratum to exclude groundwater. Unwatering of the caissons would be required and attempts to do so might result in continued flow of fines into the caisson excavation. Sealing a liner into sloping bedrock to exclude flow of groundwater and soil fines is anticipated to be difficult and impractical.

Due to the above issues, the use of augered caissons founded on bedrock is not recommended at this site.

8.3 Steel H-Piles driven to Bedrock

The subsurface conditions at the west and east abutments are considered suitable for the design of foundations supported on steel H-piles driven to achieve resistance on bedrock.

The elevations at which bedrock was contacted and the piles are expected to develop the required resistance are given in Table 8.1.

Table 8.1– Estimated Pile Tip Elevation

Foundation Unit	Location	Borehole	Top of Bedrock or Refusal	Anticipated Pile Tip Elevation To Develop Required Resistance
West abutment	North side	LWR-03 ^(1,2)	373.6	373.8
		LWR-05/DCPT ⁽³⁾	373.9	
	South side	LWR-02 ⁽²⁾	375.6	377.0
		LWR-04 ^(1,3)	378.3	
East abutment	North side	LWR-07 ^(1,3)	378.9	379.6
		LWR-09 ⁽²⁾	380.2	
	South side	LWR-06B ⁽³⁾	379.2	381.0
		LWR-08 ^(1,2)	382.8	

⁽¹⁾Bedrock proved by coring

⁽²⁾Borehole drilled from the top of highway embankment

⁽³⁾Borehole drilled from the toe of highway embankment

* GA indicates that underside of pile cap is at elevation 392.0

The pile tip elevations shown in Table 8.1 should be used for estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.3.3 Pile Driving.

Based on the borehole data, most of the piles are expected to reach bedrock or a layer of refusal. However, it must be noted that cobbles, boulders and rock protection are exposed on the side and forward slopes below the existing abutment. It is not confirmed whether rockfill is present in some parts of the approach embankment. It must be recognized that embankments fills are heterogeneous in nature and may contain obstructions such as boulders or rockfill. If such obstructions are encountered at the proposed location of the new abutments, they will have to be removed, drilled through or penetrated to facilitate pile installation.

The axial, factored geotechnical resistance at Ultimate Limit States (ULS_f) for a steel H-Pile section 310x110 driven to refusal on bedrock is 2,000 kN.

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

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

8.3.1 Pile Tips

Since these piles will be founded on sloping bedrock condition, all piles must be fitted with Titus Rock Injector points.

8.3.2 Pile Installation

Pile installation should be in accordance with OPSS 903.

The Contract Documents should contain a NSSP alerting the Bidders to the possibility of piles within a group achieving the specified resistance at different elevations due to the variable depth to the top of bedrock.

8.3.3 Pile Driving to Bedrock

It should be noted that the piles at the abutments will be founded on sloping bedrock conditions.

Furthermore, we understand that the proposed bridge design may require that the deviation at the top of the pile be limited to 12 mm. Use of a driving template or other means may be required to achieve the specified maximum deviation.

For piles installed for the tolerances shown in Clause 903.07.05.01 of the Specification, the foundation drawing should include the note “Piles to be driven to bedrock”.

To reduce the potential for piles slipping on sloping bedrock or pile misalignment resulting from hard driving to confirm bedrock, it is recommended that the pile driving note on the foundation drawing be modified as follows:

“Piles to be driven to bedrock”. Upon initial contact with the bedrock:

1. Apply 10 blows at 10% of the hammer energy. Record the penetration.
2. Apply 10 blows at 50% of the hammer energy. If the penetration under 10 blows is less than 12.5 mm, the pile is set.
3. If the penetration under 10 blows is greater than 12.5 mm, refer the issue to the design team for resolution.”

The wording for an NSSP addressing this issue is included in Appendix F. This NSSP must be included in the tender documents.

8.3.4 Downdrag

Based on the GA drawing, new fill will be placed behind the sheet pile walls and behind the concrete box girder. The thickness of the new fill will be approximately 2.5 m. Hatch Mott MacDonald have indicated that highway grade raise of 0.8 m and 0.5 m is required at the east and west abutments, respectively.

At both abutments, downdrag forces will develop along the length of the pile embedded in the soft to firm clayey silt/silty clay layer.

For design purposes, an unfactored downdrag load of 250 kN per pile is recommended to evaluate the impact of downdrag for the abutment piles.

This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C8.6.4 to obtain a factored downdrag load.

In accordance with Section 6.8.4 of the CHBDC and clause C6.8.4 of the Commentary to CHBDC, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag.

In geotechnical analysis of downdrag, live load effects should not be considered. The location of the neutral plane for a pile or groups of piles should be determined by using unfactored loads and unfactored geotechnical parameters.

Factored dead and downdrag load should not exceed the factored structural resistance of a pile.

8.3.5 Lateral Resistance

For cohesionless soils, the lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

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

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

where z = depth of embedment of pile in metres

D = pile width in metres

n_h = value from Table 8.2

γ = unit weight (Table 8.2)

K_p = passive earth pressure coefficient (Table 8.2)

For cohesive soils, the lateral resistance of the piles may be calculated as follows:

$$k_s = 67 \cdot S_u / D \quad (\text{kN/m}^3)$$

$p_{ult} = 9 \cdot S_u$ (kPa) at and below a depth of $3 \cdot D$ (m) reduced to zero at the ground surface

where

$$\begin{aligned} D &= \text{pile width in metres} \\ S_u &= \text{undrained shear strength (kPa)} \end{aligned}$$

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K , for analysis may be obtained by the expression, $K = k_s * L * D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} * L * D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in one pile be limited to no more than 110 kN at ULS and 40 kN at SLS.

Table 8.2 – Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m ³)	S_u kPa	K_p	Unit Weight (kN/m ³)	Soil Conditions
West Abutment	OGL to 388.0	2,500	-	3.0	21	Sand, silt, loose to compact (FILL)
	388.0 to 385.0	2,500	-	3.0	11*	Sand and silt, loose to compact
	385.0 to 380.0	-	35	2.7	10*	Silty clay, very soft to firm
	380.0 to 374.0	4,500	-	3.0	11*	Silt, dense to compact
East Abutment	OGL to 388.0	4,000	-	3.0	21	Sand, silt, compact to dense (FILL)
	388.0 to 380.0	3,500	-	3.0	11*	Silt, compact

*Buoyant unit weight below the water table.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on pile spacing.

Where a pile group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented *parallel* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

8.4 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions steel H-pile foundations driven to refusal on bedrock are considered the most cost effective foundation option for supporting the bridge at this site.

8.5 Depth of Frost Penetration

The design depth of frost penetration at this site is 2.5 m.

9 SHEET PILE WALLS

Steel sheet pile walls will be driven adjacent to the H-pile foundations at each abutment. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill.

Driving of the sheet piles through the existing approach fill (loose to compact sand, silt, sand and gravel and sandy silt/silty sand) and native soils is considered feasible based on the borehole data. However, cobbles, boulders and rock protection are exposed in the side and forward slopes below the existing abutment. If such obstructions are present at the locations of the sheet pile walls, they will have to be removed in order to drive the sheet piles.

Sheet piles should be provided with sheet pile tip protector to minimize any tip damage. Design of the permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause corrosion and reduce the service life of the structure.

Based on GA drawing received on May 2, 2012, the top of the sheet pile abutment wall will be near elevation 393.0. The river valley in front of the sheet pile walls is at an approximate slope of 2H:1V. The maximum thickness of backfill behind the sheet pile wall is about 2.0 m. Granular A or Granular B Type III backfill should be used behind the sheet pile wall and the box girder.

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

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

$$P_h = K(\gamma h + q)$$

Where:

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

K = earth pressure coefficient (see Table 9.1)

γ = unit weight of retained soil (see Table 9.1)

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

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 9.1.

Table 9.1 – Earth Pressure Coefficient (K)

Condition	Earth Pressure Coefficient (K)							
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$, $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$		Existing Sand Fill, Silt Fill, Native Sand/Silt $\phi = 30^\circ$, $\gamma = 20 \text{ kN/m}^3$		Native Silty Clay/Clayey Silt $\phi = 27^\circ$, $\gamma = 20 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.33	0.57*	0.38	0.75*
At rest (Restrained Wall)	0.43	-	0.47	-	0.5	-	0.55	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	3.0	-	2.7	-

* For wing walls.

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

The factors in Table 9.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or Type III or 1.7 m for Granular A or Granular B Type II.

10 EXCAVATION AND GROUNDWATER CONTROL

If any earth excavation is required, it must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils within the probable depth of excavation at this site may be classed as Type 3 soils above the water table and Type 4 soils below the water table.

The excavation must be carried out in accordance with OPSS 902.

The piezometric reading of the current investigation indicates that the groundwater level is at varies from 2.0 m to 4.4 m depth (elevations 386.1 to 389.0). The GA drawing indicates that water level in the Little Wabigoon River was at elevation 387.7 on May 12, 2011.

Based on the preliminary GA for the bridge structure and the use of pile foundations, it is not expected that work at the abutments will require excavation below the river/groundwater level.

It is recommended that excavation for removal of existing structures be maintained above the water level in the river. Any excavation below the groundwater level/river level without dewatering is not recommended since the inflow of groundwater will make it difficult to maintain a dry, sound base on which to work.

The Contract Documents should contain a NSSP alerting the Contractor to the risks associated with excavation of soils submerged below the groundwater level without prior dewatering.

In general, the design of the dewatering system should be the responsibility of the Contractor and the Contract Documents should alert him to this responsibility.

11 APPROACH EMBANKMENTS

Based on site observations and GA drawing provided by Hatch Mott MacDonald, it is estimated that the existing approach embankment are 4.0 m to 4.5 m high with forward slopes near inclinations of 2H:1V. The foundation soils governing stability of the approach embankments consist generally of existing layers of native loose to compact sand and silt and stiff to very stiff silty clay/clayey silt.

Communication with Hatch Mott MacDonald indicates that the existing Highway 17 grade will be raised approximately 0.8 m and 0.5 m at the east and west abutments, respectively.

GA indicates that additional fill will be required between the new sheet pile wall and the new abutment. This new fill is expected to have a maximum thickness of about 2.5 m and extend for a length of up to 3.1 m behind the new sheet pile abutment wall. The sides of the new approach fill will be contained by sheet pile walls installed along each edge of the road.

Embankment construction should be carried out in accordance with OPSS 206. If new fill is placed against the existing fill, existing sloped embankment surfaces should be appropriately benched as per OPSD 208.010, after stripping of vegetation/organics, soft soils or otherwise unsuitable materials.

Comments regarding stability of embankment slopes and settlement of the foundations soils are provided in the following sections.

11.1 Slope stability

The existing embankments bearing on the foundation soils at this site appear to be performing satisfactorily under the existing conditions. For a grade raise of 0.5 m to 0.8 m, the stability of the existing embankment should continue to be satisfactory.

The additional approach fill (approximately 2.5 m) to be placed behind the new abutment will be supported within a sheet pile enclosure and therefore the stability of the new

approach will be governed by the sheet pile wall design. A global slope stability analysis was conducted to assess the embedment requirements for a sheet pile supporting the new 2.5 m high fill behind the sheet pile walls and the 0.8 m of new fill for grade raise. The analyses were carried out using the Morgenstern-Price method of slope stability analysis.

The results of the analyses indicate that an adequate factor of safety for the long term conditions of 1.5 is achieved if the sheet pile is driven to elevations 385.5 and 386.1 at the west and east abutments, respectively.

The slope stability computation outputs are included in Appendix E.

The stability of the embankments was not checked under seismic loading as the zonal acceleration at this site is 0.0g.

11.2 Settlement

The placement of approximately 2.5 m of new fill behind the sheet pile abutments and approximately 0.8 and 0.5 m of granular fill at the east and west abutments, respectively to raise the existing highway grade will induce immediate (elastic) settlement in the existing non-cohesive fill and silt layers as well as time dependent (consolidation) settlement in the underlying silty clay.

The total immediate and consolidation settlements were assessed using elastic methods and one-dimensional consolidation theory. Based on these analyses, the settlements at the bridge approaches are in the order of 25 mm to 30 mm, at the east and west abutments, respectively.

Inspection of the roadway surface and placing of additional granular base or asphalt padding to re-establish grades as necessary should be implemented during and after construction.

The settlement in the sand and silt layers will largely be completed by the end of construction. Settlement in the foundation clay is expected to be completed within 1 to 2 months following construction.

If post construction settlement and maintenance is not acceptable, consideration should be given to using lightweight fill or EPS as backfill behind the sheet pile wall in order to reduce the settlements.

12 EROSION PROTECTION

Erosion protection should be provided along the toe of any slopes that may be in contact with the river flow.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

13 BACKFILL TO ABUTMENTS

Backfill to the abutment if required, must consist of granular material.

Backfill to the abutments should consist of Granular A or Granular B Type III material meeting the requirements of Special Provision 110S13 “Amendment to OPSS 1010, April 2004”. The backfill must be in accordance with OPSS 902, and placed to the extents shown in OPSD 3101.150.

All new embankment earth fill should be placed in uniform lifts and be compacted in accordance with OPSS 501. Also, compaction equipment to be used adjacent to retaining structures must be restricted in accordance OPSS 501.

14 ROADWAY PROTECTION

During the new bridge construction, temporary excavation of existing embankments will be required. The bridge construction will be done in stages in order to keep at least one highway lane operational. Roadway protection will be required to facilitate staging of removals and support the existing Highway 17 adjacent to the excavation.

Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

Continuous sheet pile walls or conventional steel soldier pile and timber lagging wall are available options to provide temporary support to the roadway during excavation. Timber lagging boards should be installed as soon as the soil face is exposed and properly prepared.

The following parameters apply for design of the temporary shoring system:

γ	=	21 kN/m ³	(bulk unit weight)
γ_w	=	11 kN/m ³	(submerged unit weight under groundwater table)
K_a	=	0.33	(Active pressure coefficient for: road embankment sand and silt fill and native sand/silt)
K_p	=	3.0	(Passive pressure coefficient for: road embankment sand and silt fill and native sand/silt)
h_w	=	387.7	(Groundwater level)

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system.

Temporary groundwater and surface water control measures may be required during construction.

The design of roadway protection should be the responsibility of the Contractor. All shoring systems should be designed by a Professional Engineer experienced in such designs.

15 SEISMIC CONSIDERATIONS

15.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.02

The soil profile type at this site has been classified as Type III. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.5 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 15.1 may be used:

Table 15.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$ $\gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I or Type III $\phi = 32^\circ$ $\gamma = 21.2 \text{ kN/m}^3$	Existing Sand Fill, Native Sand/Silt $\phi = 30^\circ$ $\gamma = 20 \text{ kN/m}^3$	Native Silty Clay/Clayey Silt $\phi = 27^\circ$ $\gamma = 20 \text{ kN/m}^3$
Active (K_{AE})*	0.28	0.32	0.34	0.39
Passive (K_{PE})	3.7	3.2	2.9	2.7
At Rest (K_{OE})**	0.45	0.50	0.53	0.57

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

** After Woods

The site overlies loose to compact silts with high water table. A cursory review of the subsurface conditions indicates that a potential for liquefaction exists under the current conditions at this site.

Since the site is in a velocity related seismic zone of zero, this is not considered a significant risk. Localized liquefaction during a seismic event may result in local toe failure or a minimal embankment settlement which is expected to be readily repairable.

16 CONSTRUCTION CONCERNS

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

- The surface of the bedrock has been shown to be sloping from southeast to northwest across the site. Since the elevation of the bedrock surface was only established at discrete points, it is possible that higher or lower bedrock elevations will be encountered during construction. Furthermore, care must be employed while seating the piles on sloping bedrock not to cause pile slippage on bedrock.
- The potential variability of pile lengths driven to bedrock. Bedrock and auger refusal on probable bedrock were contacted at depth varying from 8.1 m to 19.4 m (373.6 to 385.0).
- Based on the borehole data, driving sheet piles near the location of the existing abutment is feasible. Visual site inspection indicates presence of rockfill on the side and forward slopes below the existing abutments. It is not confirmed whether the approach embankment contains rockfill in some areas. If the sheet piles or driven steel H-piles encounter these conditions, the obstructions will have to be removed for driving the piles.
- Roadway protection must be provided to maintain traffic during construction. Temporary shoring systems should be properly designed by a Professional Engineer experienced in such designs.

- The embankment side slopes should be inspected after construction for surficial disturbance. Where necessary, erosion control measures must be implemented.

17 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.



Rocío Palomeque Reyna, P.Eng., M.Eng.
Geotechnical Engineer

Report reviewed by:
P.K. Chatterji, P.Eng., Ph.D.
Review Principal



Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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


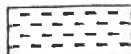



 Water Level
 Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
		Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

RECORD OF BOREHOLE No LWR-01

1 OF 2

METRIC

W.P. 470-00-00 LOCATION N 5 492 520.8 E 359 232.7 Little Wabigoon River Bridge ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.07.13 - 2011.07.13 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
393.1												
0.0	ASPHALT: (125mm)											
0.1	SAND, some gravel						393					
392.5	Brown											
0.6	Moist (FILL)											
	SILT, some clay to clayey, trace sand		1	SS	30		392					
	Compact											
	Brown		2	SS	13							
	Moist (FILL)											
							391					
390.4			3	SS	12							
2.8	Sandy SILT to Silty SAND, occasional organics and wood fragments											
	Compact											
	Brown		4	SS	15		390					
	Moist (FILL)											
389.2												
3.9	SAND and SILT						389					
	Compact to Loose											
	Brown		5	S	14							
	Moist to Wet						388					
			6	SS	6		387					
386.0												
7.2	Clayey SILT						386					
	Soft											
	Grey		7	SS	4							
							385					
	Some sand											
			8	SS	4		384					

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LWR-01

2 OF 2

METRIC

W.P. 470-00-00 LOCATION N 5 492 520.8 E 359 232.7 Little Wabigoon River Bridge ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.07.13 - 2011.07.13 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE						
								PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)						
381.9	Clayey SILT, silty sand seams Stiff Grey		9	SS	11		383							0 0 72 28
11.3	END OF BORHOLE AT 11.3m. NO WATER OBSERVED UPON COMPLETION OF BOREHOLE. BOREHOLE BACKFILLED WITH HOLEPLUG TO 2.5m, THEN CUTTINGS TO 0.3m, THEN CONCRETE TO SURFACE.						382							

METRIC

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	20 40 60 80 100	w _p	w		
393.1													
0.0	ASPHALT: (75mm)												
0.1													
392.7	CONCRETE: (325mm)												
0.4	SAND, some gravel												
392.4	Brown Moist (FILL)												
0.7	SILT, some clay to clayey, trace sand Compact Brown (FILL)		1	SS	20								
			2	SS	12								
			3	SS	12								
389.7													
3.4	SAND and SILT, trace clay Loose Grey Wet		4	SS	5								
			5	SS	9								
			6	SS	7								
385.2													
7.9	Clayey SILT, sandy silt seams Firm Grey		7	SS	7								

+³, ×³: Numbers refer to Sensitivity

ONTMT4S 5121 GPJ 5/29/12

RECORD OF BOREHOLE No LWR-02

2 OF 2

METRIC

W.P. 470-00-00 LOCATION N 5 492 509.1 E 359 235.7 Little Wabigoon River Bridge ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.08.13 - 2011.08.13 CHECKED BY RPR

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	20 40 60 80 100	20 40 60	w _p w w _L	GR		SA	SI	CL		
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE																		
	Continued From Previous Page																	
	Clayey SILT Soft to Firm Grey		8	SS	3		383											
							382	+ ²										
			9	SS	7		381							0 0 77 23				
380.4																		
12.7	SILT , trace clay Compact Grey Moist		10	SS	21		380											
							379											
			11	SS	22		378							0 0 92 8				
							377											
			12	SS	17		376											
375.6																		
17.4	END OF BOREHOLE AT 17.4m UPON REFUSAL ON PROBABLE BEDROCK. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Aug.17/11 5.0 388.1 Sep.15/11 5.5 387.6 Oct.11/11 5.0 388.1																	

+³, X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LWR-03

1 OF 3

METRIC

W.P. 470-00-00 LOCATION N 5 492 515 3 E 359 240 2 Little Wabigoon River Bridge ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.11 - 2011.08.12 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
393.1	ASPHALT: (75mm)						393							
0.0														
0.1	CONCRETE: (325mm)													
392.7														
0.4	SAND, some gravel													
392.3	Brown													
0.7	Moist													
	(FILL)													
	SILT, some clay to clayey, trace sand		1	SS	8		392							
	Loose													
	Brown													
	(FILL)		2	SS	9									
							391							
390.4														
2.7	Sandy SILT to Silty SAND, with		3	SS	10		390							
	organics													
	Stiff													
	Brown													
	Moist to Wet													
	(FILL)													
389.2	Layer of clayey silt fill													
3.8	SAND and SILT		4	SS	9		389							
	Loose													
	Brown													
	Moist to Wet													
							388							
			5	SS	10									
							387							
			6	SS	6		386							
385.3	No sample retrieved													
7.8	Clayey SILT						385							
	Stiff													
	Grey		7	SS	8		384							

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LWR-03

2 OF 3

METRIC

W.P. 6936-10-00 LOCATION N 5 492 515.3 E 359 240.2 Little Wabigoon River Bridge ORIGINATED BY SLL
HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.08.11 - 2011.08.12 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
	Continued From Previous Page							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%) 20 40 60			GR SA SI CL
380.3	Clayey SILT to Silty CLAY Soft Grey		8	SS	2		383					0 0 46 54
							382	3				
			9	SS	2		381			○		
								+2				
12.8	SILT, trace to some clay Compact Grey Moist		10	SS	18		380			○		
							379					
			11	SS	27		378			○		0 0 91 9
							377			○		
			12	SS	28		376			○		
							375			○		0 0 88 12
	Dense		13	SS	31		374					
373.6			14	SS	50/ 0.075							
19.4	BEDROCK, granodiorite to granite, slightly weathered, grey, with white bands, occasional mechanical and sub-vertical breaks										FI 0	

Continued Next Page

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

RECORD OF BOREHOLE No LWR-03

3 OF 3

METRIC

W.P. 6936-10-00 LOCATION N 5 492 515.3 E 359 240.2 Little Wabigoon River Bridge ORIGINATED BY SLL
HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.08.11 - 2011.08.12 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE			w _P w w _L				
								20 40 60 80 100	20 40 60						
	Continued From Previous Page														
	Coring started at 19.4m BEDROCK , granodiorite to granite, slightly weathered, grey, with white bands, occasional and sub-vertical breaks Sub-vertical breaks: 125mm at 19.8m 175mm at 20.1m 50mm at 20.8m Quartzile seams: 75mm at 20.0m 63mm at 20.3m		1	RUN			373						3	RUN #1 TCR=100% SCR=100% RQD=76% UCS=241MPa (Average)	
													1		
													0		
													2		
													1		
													3	RUN #2 TCR=100% SCR=100% RQD=82% UCS=257MPa (Average)	
													4		
													1		
370.5			2	RUN			371						2		
22.5	END OF BOREHOLE AT 22.5m. BOREHOLE BACKFILLED WITH HOLEPLUG BENTONITE TO 2.1m, SAND TO 0.3m, THEN CONCRETE TO SURFACE.														

RECORD OF BOREHOLE No LWR-04

1 OF 2

METRIC


W.P. 6936-10-00 LOCATION N 5 492 502.9 E 359 244.7 Little Wabigoon River Bridge ORIGINATED BY MAT
HWY 17 BOREHOLE TYPE Wash Boring/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.09.15 - 2011.09.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	
387.9												
0.0	SAND and GRAVEL, cobbles and boulders at surface Loose to Compact Brown		1	SS	12		388					
387.3	Moist to Wet (FILL)		2	SS	5							
0.6	SAND and SILT, trace clay, occasional wood fragments Loose Grey Moist to Wet		3	SS	5		387					0 43 51 6
386.1	SILT, trace gravel, trace sand, trace to some clay Compact Grey Wet		4	SS	27		386					0 4 84 12
1.8			5	SS	16							
			6	SS	10		385					
			7	SS	13							
			8	SS	22		384					
			9	SS	23		383					0 1 94 5
							382					
			10	SS	20							
			11	SS	21		381					0 0 93 7
380.6	Clayey SILT Very Stiff to Hard		12	SS	23							
7.3			13	SS	31		380					
			14	SS	23							0 0 72 28
			15	SS	100/		379					
378.3	Sand layer (100mm)		16	SS	0.250							
9.6	BEDROCK, granodiorite to granite, slightly weathered, grey		1	RUN	57							RUN #1 TCR=100% SCR=100%

Continued Next Page

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

METRIC

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	SHEAR STRENGTH kPa					WATER CONTENT (%)			UNIT WEIGHT								
								20	40	60	80	100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			γ _p	γ _w							
								○ UNCONFINED + FIELD VANE									kN/m ³							
								● QUICK TRIAXIAL × LAB VANE																
	Continued From Previous Page																							
	BEDROCK, granodiorite to granite, slightly weathered, occasional mechanical and vertical breaks, grey, with white bands Coring started at 9.6m		2	RUN			378										1	RQD=75% UCS=310MPa (Average)						
	Moderately to slightly weathered, some mechanical breaks		3	RUN			377										4	TCCR=100% SCR=96% RQD=88% UCS=240MPa (Average)						
	Horizontal breaks at 10.7m, 10.8m, 11.0m, 11.1m, 11.3m and 11.9m		4	RUN													2	RUN #3 TCCR=100% SCR=85% RQD=77% UCS=240MPa (Average)						
	Horizontal breaks at 12.2m and 12.4m		5	RUN			376										6	RUN #4 TCCR=100% SCR=87% RQD=87% UCS=328MPa (Average)						
																	0	RUN #5 TCCR=100% SCR=94% RQD=94% UCS=290MPa (Average)						
375.1	END OF BOREHOLE AT 12.9m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.																1							
12.9																	1							

RECORD OF BOREHOLE No LWR-05

1 OF 2

METRIC

W.P. 6936-10-00 LOCATION N 5 492 507.3 E 359 248.6 Little Wabigoon River Bridge ORIGINATED BY MAT
HWY 17 BOREHOLE TYPE Wash Boring COMPILED BY AN
DATUM Geodetic DATE 2011.09.20 - 2011.09.20 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L		
388.1 0.0	SAND and GRAVEL, cobbles and boulders at surface, occasional roots Loose Brown		1	SS	7		388						
387.5 0.6	Damp (FILL)		2	SS	5								
	SAND and SILT, trace clay, trace gravel Loose to Compact Brown to Grey Moist to Wet		3	SS	11		387						
			4	SS	15								
			5	SS	16		386						0 56 41 3
			6	SS	20		385						
384.5 3.7	SILT, some sand, some clay Loose to Compact Grey Wet		7	SS	16		384						0 17 70 13
			8	SS	12								
			9	SS	11		383						
			10	SS	6								
			11	SS	7		382						
			12	SS	11								
			13	SS	41		381						0 0 93 7
	Dense to Compact		14	SS	30		380						
			15	SS	35								
			16	SS	34		379						

Continued Next Page

+ ³, × ³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LWR-05

2 OF 2

METRIC

W.P. 6936-10-00 LOCATION N 5 492 507.3 E 359 248.6 Little Wabigoon River Bridge ORIGINATED BY MAT
HWY 17 BOREHOLE TYPE Wash Boring COMPILED BY AN
DATUM Geodetic DATE 2011.09.20 - 2011.09.20 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
	Continued From Previous Page							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					
								20 40 60 80 100	20 40 60 80 100	20 40 60			
376.6	SILT, trace clay Dense to Compact Grey Wet		17	SS	36		378						
			18	SS	21		377						
11.6	End of sampling at 11.5m and start DCPT						376						
373.9							375						
14.2	END OF BOREHOLE AT 14.2m. WATER LEVEL OBSERVED AT 1.2m UPON COMPLETION OF DRILLING. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep.23/11 2.0 386.1												




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

RECORD OF BOREHOLE No LWR-06

1 OF 1

METRIC

W.P. 470-00-00 LOCATION N 5 492 494.1 E 359 256.6 Little Wabigoon River Bridge ORIGINATED BY MAT
HWY 17 BOREHOLE TYPE Wash Boring COMPILED BY AN
DATUM Geodetic DATE 2011.09.23 - 2011.09.23 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)					
388.8	SAND and GRAVEL, with cobbles and boulders Brown to Grey Moist to Wet (FILL)													GR SA SI CL	
0.0															
386.8	SAND and SILT, some clay, occasional roots and wood fragments Loose to Compact Grey Wet													0 34 54 12	
2.0															
	Dense		3	SS	32										
385.1	END OF BOREHOLE AT 3.6m. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.														
3.7															

RECORD OF BOREHOLE No LWR-06B

1 OF 2

METRIC

W.P. 6936-10-00 LOCATION N 5 492 494.1 E 359 256.6 Little Wabigoon River Bridge ORIGINATED BY JM
HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.10.22 - 2011.10.22 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L		
393.0													
0.0	ASPHALT: (100mm)						393						
0.1													
392.5	CONCRETE (bridge deck): (400mm)												
0.5	Gap between underside of deck and ground surface												
							392						
							391						
							390						
							389						
388.7													
4.3	SAND and GRAVEL Loose Brown Moist (FILL)		1	SS	4		388						
387.5													
5.5	SAND and SILT, trace gravel, trace clay Loose Grey Moist to Wet		2	SS	8		387						4 23 69 4
			3	SS	8								
			4	SS	8		386						
384.9							385						
8.1	SILT, trace clay Compact Grey Wet		5	SS	29		384						0 0 96 4

Continued Next Page

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

RECORD OF BOREHOLE No LWR-06B

2 OF 2

METRIC

W.P. 6936-10-00 LOCATION N 5 492 494.1 E 359 256.6 Little Wabigoon River Bridge ORIGINATED BY JM
HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.10.22 - 2011.10.22 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	W _P	W		
	Continued From Previous Page						20	40	60	80	100				
	SILT , trace sand, trace clay Compact to Dense Grey Wet Boulder from 10.7m to 11.0m		6	SS	31								○		
			7	SS	41								○		
380.3															
12.6	Clayey SILT , trace gravel and sand Grey Stiff		8	SS	12								○		1 3 76 20
379.2															
13.8	BEDROCK , granodiorite to granite, grey														
378.9															
14.1	END OF BOREHOLE AT 14.1m UPON REFUSAL ON BEDROCK. BOREHOLE OPEN TO 14.1m AND WATER LEVEL AT 4.8m BELOW DECK UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 14.1m TO 4.3m THEN CONCRETE FROM 0.3m TO SURFACE.														

METRIC

[illegible]

+ 3, × 3: Numbers refer to Sensitivity

ONTMT4S 5121.GPJ 5/29/12

RECORD OF BOREHOLE No LWR-07

2 OF 2

METRIC

W.P. 6936-10-00 LOCATION N 5 492 502.6 E 359 263.8 Little Wabigoon River Bridge ORIGINATED BY MAT
HWY 17 BOREHOLE TYPE Wash Boring/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2011.09.22 - 2011.09.22 CHECKED BY RPR

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								

RECORD OF BOREHOLE No LWR-08

1 OF 2

METRIC

W.P. 470-00-00 LOCATION N 5 492 487.2 E 359 264.4 Little Wabigoon River Bridge ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.10 - 2011.08.10 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
393.0	ASPHALT: (75mm)						393					
0.0												
0.1												
392.6	CONCRETE: (325mm)											
0.4												
	SAND, trace gravel, some silt to silty, trace clay, occasional asphalt fragments Compact to Dense Brown Moist (FILL)		1	SS	24		392					
			2	SS	40		391					
390.7												
2.3	SILT, some sand, trace gravel, trace clay Compact Brown Moist (FILL)		3	SS	16		390					
389.3												
3.7	SILT, trace to some clay, trace sand, occasional rootlets Very Loose to Loose Grey Moist to Wet		4	SS	2		389					0 7 82 11
							388					
			5	SS	7		387					
							386					
	Compact		6	SS	13		385					
			7	SS	14		384					0 0 91 9

Continued Next Page

+ ³ . X ³ . Numbers refer to
Sensitivity 20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LWR-08

2 OF 2

METRIC

W.P. 6936-10-00 LOCATION N 5 492 487.2 E 359 264.4 Little Wabigoon River Bridge ORIGINATED BY SLL
HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2011.08.10 - 2011.08.10 CHECKED BY RPR

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE			WATER CONTENT (%) w _P w w _L				
	Continued From Previous Page							20	40	60	80	100			
382.8	Refusal at 10.1m and start coring		8	SS	100/		383							FI	
10.2	BEDROCK , granodiorite to granite, slightly weathered to fresh, light grey, occasional vertical and mechanical breaks				0.100									5	
														1	
														0	
							382							0	
														0	
			1	RUN										0	
														0	
	Mechanical break at 12.1m						381							1	RUN #1 TCR=100% SCR=100% RQD=91% UCS=247MPa (Average)
														0	
														0	
														0	
379.9							380							0	
13.1	END OF BOREHOLE AT 13.1m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 1.5m, SAND TO 0.3m THEN CONCRETE TO SURFACE.														

RECORD OF BOREHOLE No LWR-09

1 OF 2

METRIC

W.P. 6936-10-00 LOCATION N 5 492 493.0 E 359 268.8 Little Wabigoon River Bridge ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.11 - 2011.08.11 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
393.0												
0.0	ASPHALT: (75mm)						393					
0.1												
392.6	CONCRETE: (325mm)											
0.4												
	SAND, some gravel, some silt to silty, trace clay Dense to Compact Brown Moist (FILL)		1	SS	32		392					
			2	SS	10		391					9 65 23 3
390.7												
2.2	SILT, some sand, trace gravel, trace clay Compact Brown Moist (FILL)		3	SS	16		390					
389.3							389					
3.7	Clayey SILT, trace sand Firm to Very Stiff Brown to Grey Moist		4	SS	5							
							388					
			5	SS	6		387					0 4 70 26
			6	SS	16		386					
							385					
			7	SS	22		384					
383.1												

Continued Next Page

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

RECORD OF BOREHOLE No LWR-09

2 OF 2

METRIC

W.P. 470-00-00 LOCATION N 5 492 493 0 E 359 268 8 Little Wabigoon River Bridge ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.11 - 2011.08.11 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
	Continued From Previous Page											
9.9	SILT, trace clay Compact Grey Wet		8	SS	20		383					0 0 91 9
							382					
			9	SS	26		381					
380.2												
12.7	END OF BOREHOLE AT 12.7m UPON REFUSAL ON PROBABLE BEDROCK. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Aug.17/11 4.2 388.8 Sep.15/11 4.4 388.6 Oct.11/11 4.1 389.0											

+³, X³: Numbers refer to
Sensitivity 20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No LWR-10

1 OF 1

METRIC

W.P. 470-00-00 LOCATION N 5 492 480.4 E 359 272.8 Little Wabigoon River Bridge ORIGINATED BY SLL
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.08.11 - 2011.08.11 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	
393.1													
0.0	ASPHALT: (150mm)												
0.2	SAND, some gravel, occasional cobbles Dense Brown Moist (FILL)		1	GS			393						
			1	SS	30		392						
391.7													
1.4	Clayey SILT Stiff to Very Stiff Brown		2	SS	25								
							391						
			3	SS	16								
			4	SS	8		390						
							389						
			5	SS	8								
							388						
387.0							387						
6.1	SAND and SILT, some gravel, trace clay Compact to Dense Grey Wet		6	SS	14								
							386						
			7	SS	70/ 0.250								
385.0													
8.1	END OF BOREHOLE AT 8.1m UPON AUGER REFUSAL ON PROBABLE BEDROCK OR BOULDER. BOREHOLE CAVED TO 7.1m AND NO WATER WAS OBSERVED. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 1.5m, SAND TO 0.3m, THEN CONCRETE TO SURFACE.												

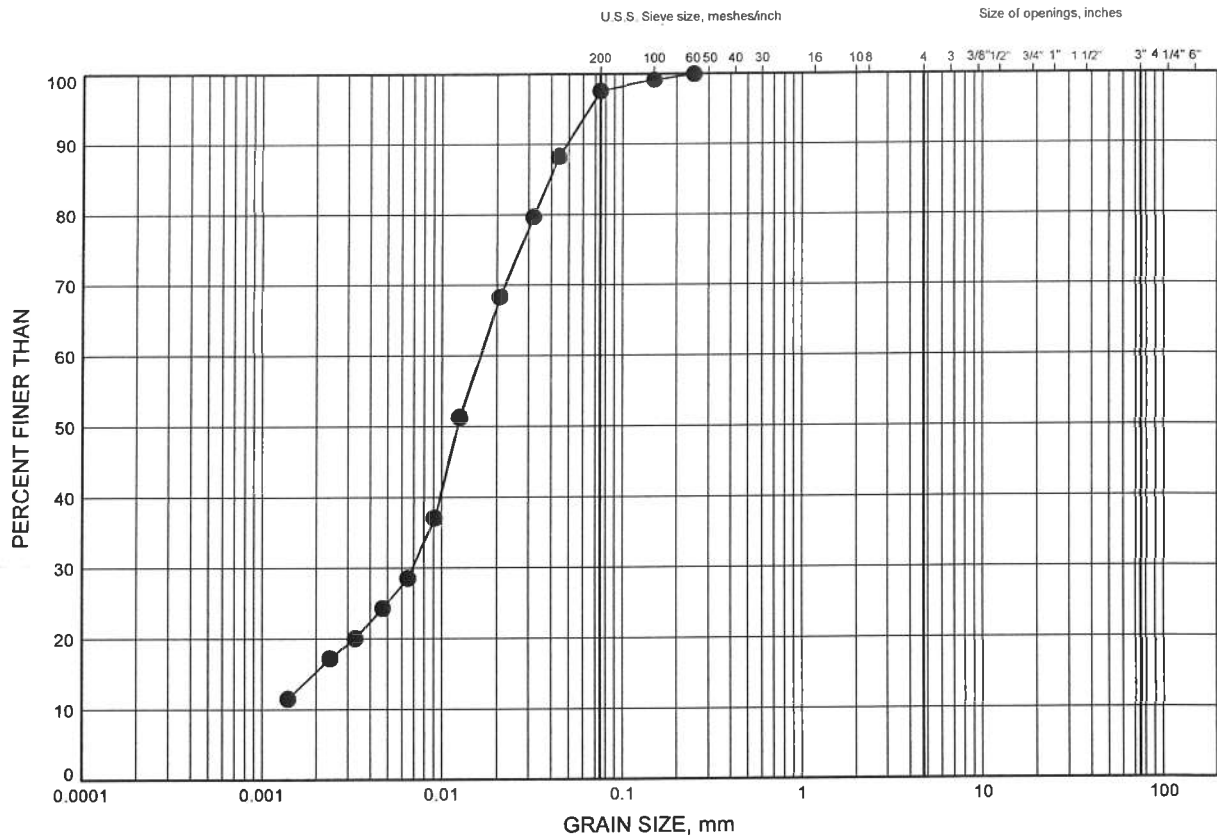
+³, ×³: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

Appendix B
Laboratory Test Results

Little Wabigoon River GRAIN SIZE DISTRIBUTION

FIGURE B1

SILT FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-01	1.83	391.31

Date May 2012
W.P.# 470-00-00

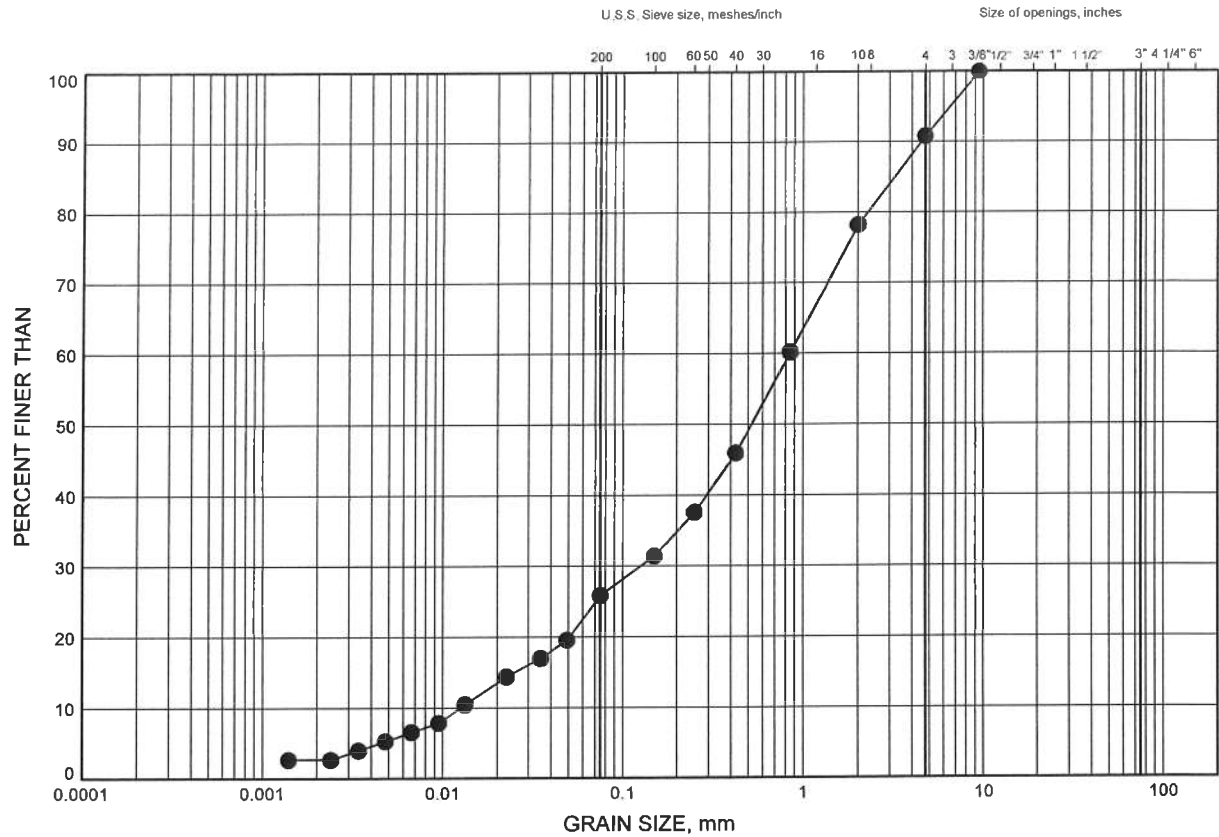


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Little Wabigoon River
GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-09	1.83	391.13

Date May 2012
W.P.# 470-00-00

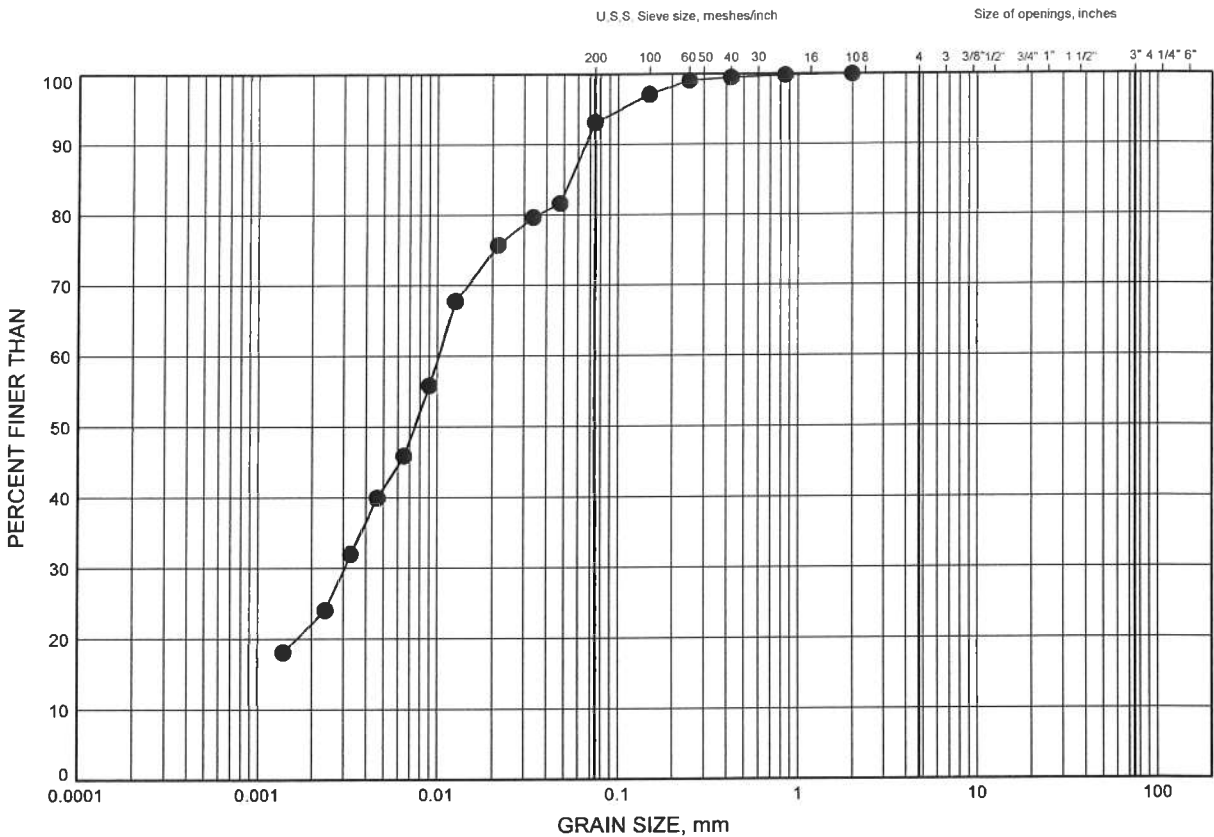


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Little Wabigoon River GRAIN SIZE DISTRIBUTION

FIGURE B3

CLAYEY SILT FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-03	3.60	389.46

Date May 2012
W.P.# 470-00-00

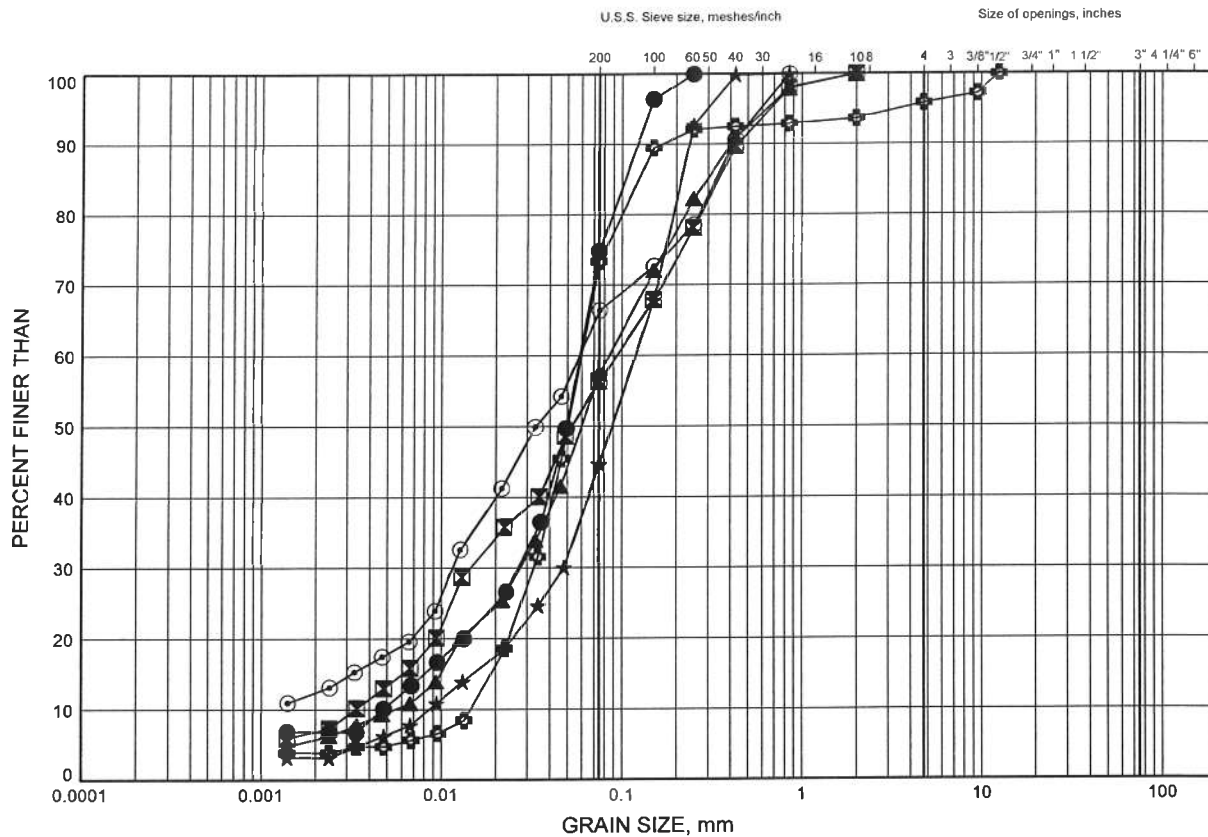


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Little Wabigoon River GRAIN SIZE DISTRIBUTION

FIGURE B4

SAND & SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-02	5.79	387.30
⊠	LWR-02	7.32	385.78
▲	LWR-04	1.52	386.42
★	LWR-05	2.13	386.00
⊙	LWR-06	3.35	385.44
⊕	LWR-06B	5.79	387.20

Date May 2012

W.P.# 470-00-00



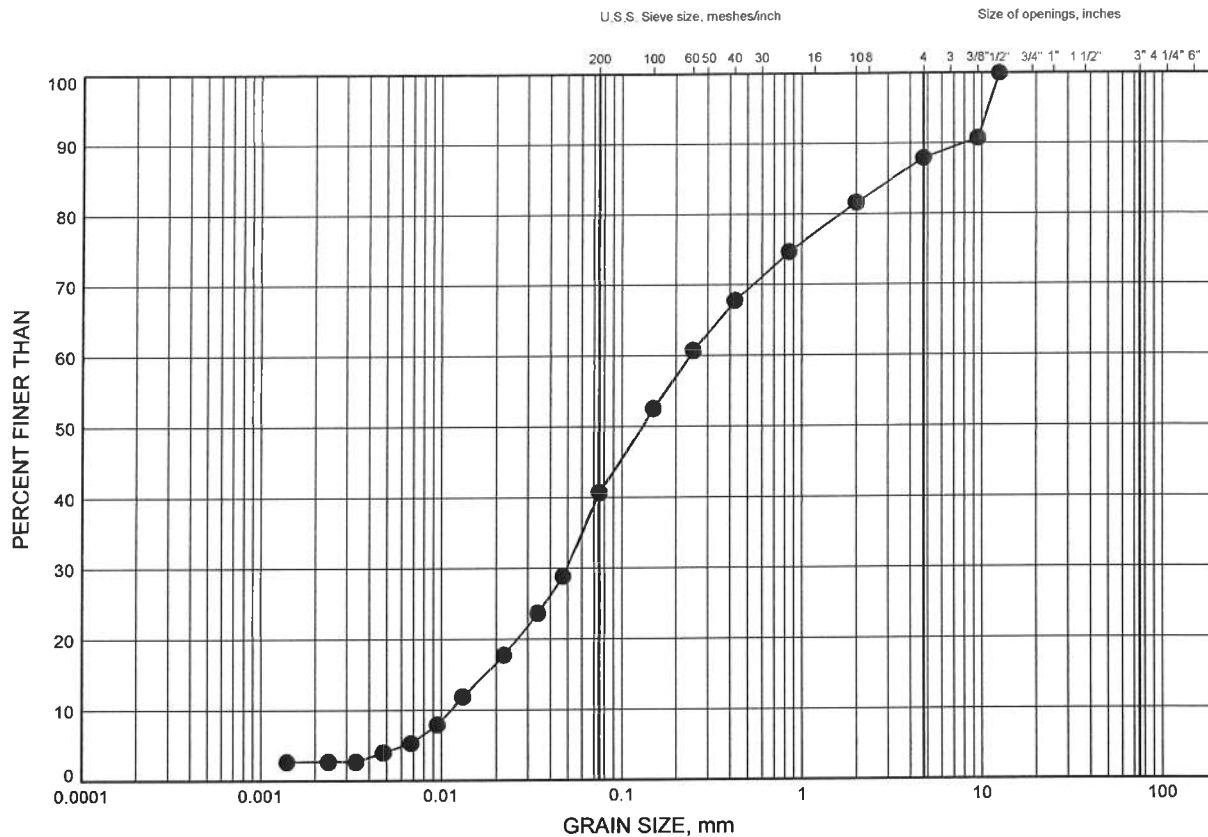
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Little Wabigoon River GRAIN SIZE DISTRIBUTION

FIGURE B5

SAND & SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-10	7.82	385.28

Date May 2012

W.P.# 470-00-00



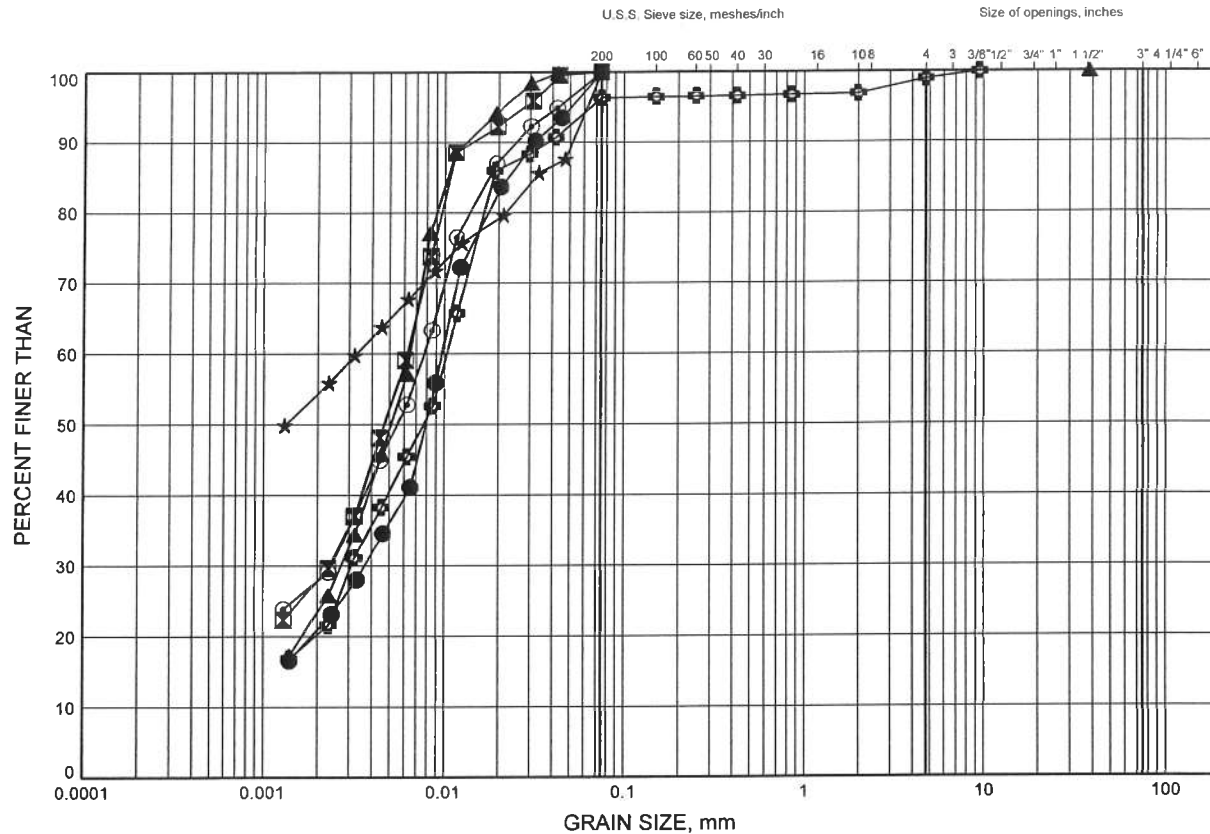
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Little Wabigoon River GRAIN SIZE DISTRIBUTION

FIGURE B6

CLAYEY SILT to SILTY CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-01	7.92	385.22
⊠	LWR-01	10.97	382.17
▲	LWR-02	11.89	381.21
★	LWR-03	10.36	382.69
⊙	LWR-04	8.84	379.10
⊕	LWR-06B	13.41	379.58

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W.P.# 470-00-00



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FIGURE B7

U.S.S. Sieve size, meshes/finch

Size of openings, inches

PERCENT FINER THAN

GRAIN SIZE, mm

Grain Size (mm)	U.S.S. Sieve Size	Opening Size (inches)	Percent Finer (Circles)	Percent Finer (Squares)	Percent Finer (Triangles)	Percent Finer (Stars)
0.0015			28	22	22	18
0.0025			32	28	28	23
0.00375			38	32	32	27
0.005			44	38	35	35
0.0075			50	42	40	42
0.01	20	0.004	63	67	48	54
0.015	10	0.0075	75	80	60	69
0.025	60	0.0125	88	90	68	81
0.0375	40	0.015	95	95	75	93
0.05	30	0.02	98	98	80	97
0.075	20	0.025	100	100	96	99
0.1	100	0.0375	100	100	99	100
0.25	60	0.0625	100	100	100	100
0.425	40	0.1	100	100	100	100
0.85	20	0.2	100	100	100	100
1.75	10	0.425	100	100	100	100
3.5	5	0.85	100	100	100	100
7.0	3	1.75	100	100	100	100
14.0	1 1/2	3.5	100	100	100	100
28.0	3/4	7.0	100	100	100	100
56.0	1/2	14.0	100	100	100	100
112.0	3/8	28.0	100	100	100	100
224.0	1/4	56.0	100	100	100	100
448.0	1/8	112.0	100	100	100	100
896.0	1/16	224.0	100	100	100	100
1792.0	1/32	448.0	100	100	100	100
3584.0	1/64	896.0	100	100	100	100
7168.0	1/128	1792.0	100	100	100	100
14336.0	1/256	3584.0	100	100	100	100
28672.0	1/512	7168.0	100	100	100	100
57344.0	1/1024	14336.0	100	100	100	100
114688.0	1/2048	28672.0	100	100	100	100
229376.0	1/4096	57344.0	100	100	100	100
458752.0	1/8192	114688.0	100	100	100	100
917504.0	1/16384	229376.0	100	100	100	100
1835008.0	1/32768	458752.0	100	100	100	100
3670016.0	1/65536	917504.0	100	100	100	100
7340032.0	1/131072	1835008.0	100	100	100	100
14680064.0	1/262144	3670016.0	100	100	100	100
29360128.0	1/524288	7340032.0	100	100	100	100
58720256.0	1/1048576	14680064.0	100	100	100	100
117440512.0	1/2097152	29360128.0	100	100	100	100
234881024.0	1/4194304	58720256.0	100	100	100	100
469762048.0	1/8388608	117440512.0	100	100	100	100
939524096.0	1/16777216	234881024.0	100	100	100	100
1879048192.0	1/33554432	469762048.0	100	100	100	100
3758096384.0	1/67108864	939524096.0	100	100	100	100
7516192768.0	1/134217728	1879048192.0	100	100	100	100
15032385536.0	1/268435456					

SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-07	4.57	384.22
☒	LWR-07	9.45	379.35
▲	LWR-09	5.79	387.17
★	LWR-10	2.59	390.52

Date May 2012

W.P.# 470-00-00

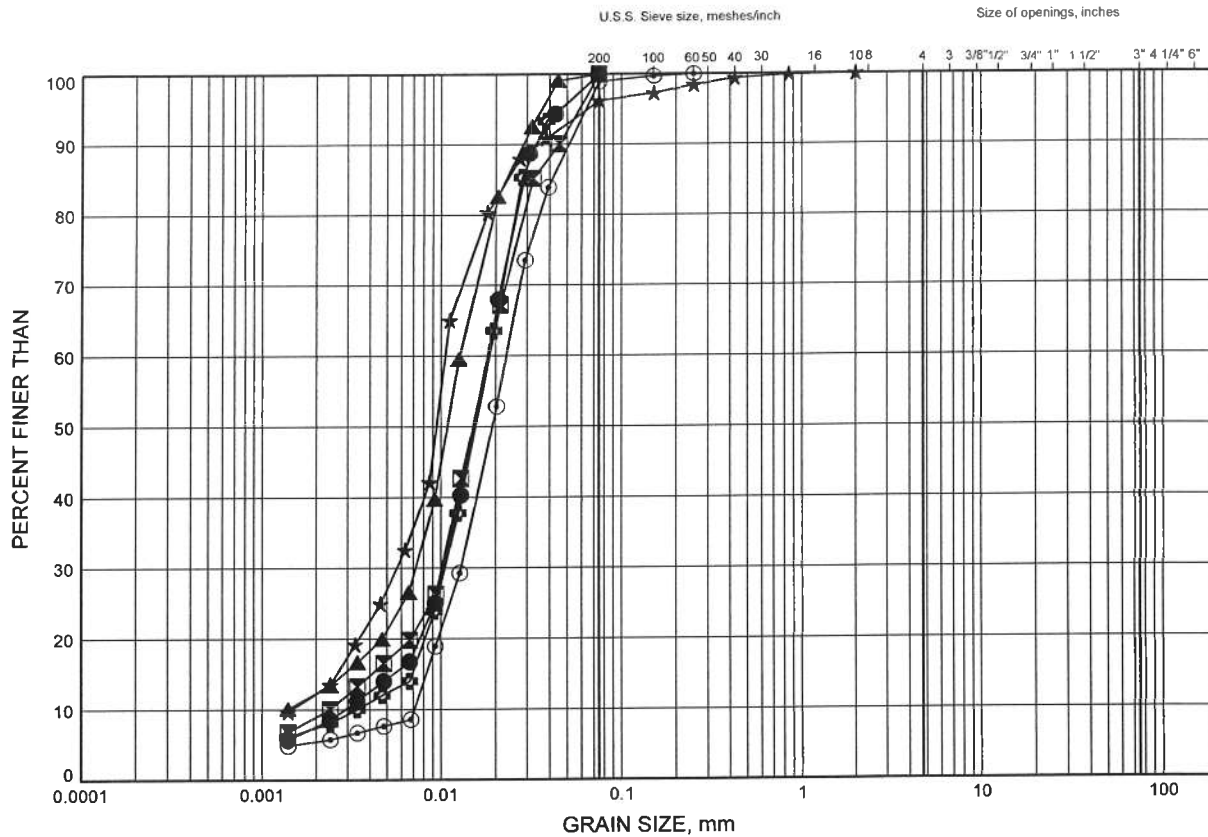


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Little Wabigoon River GRAIN SIZE DISTRIBUTION

FIGURE B8

SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-02	14.94	378.16
⊠	LWR-03	14.94	378.12
▲	LWR-03	17.98	375.07
★	LWR-04	2.13	385.81
⊙	LWR-04	5.18	382.76
⊕	LWR-04	7.01	380.93

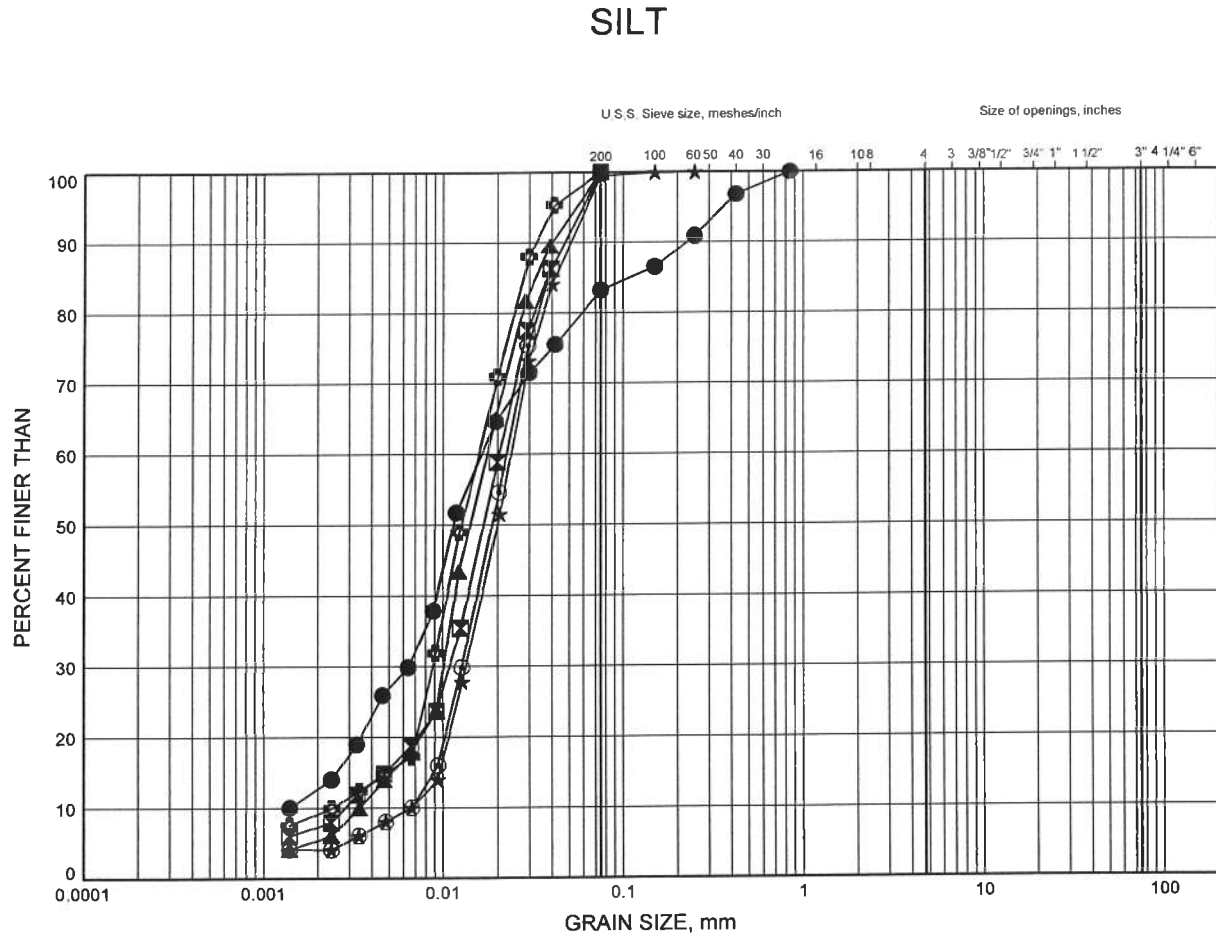
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Little Wabigoon River GRAIN SIZE DISTRIBUTION

FIGURE B9



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-05	3.96	384.17
⊠	LWR-05	7.62	380.52
▲	LWR-05	10.67	377.47
★	LWR-06B	8.84	384.16
⊙	LWR-07	6.40	382.39
⊕	LWR-07	8.23	380.57

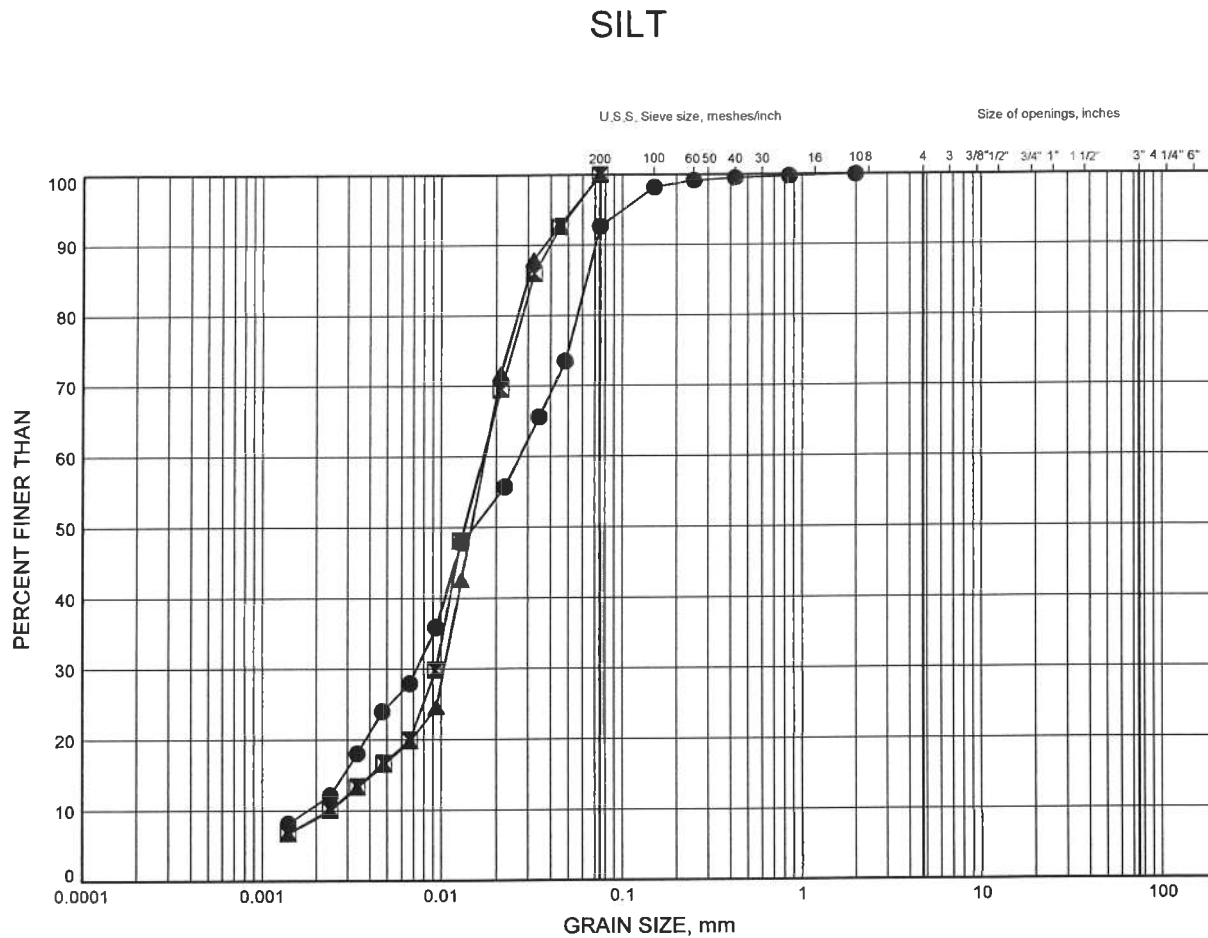
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Little Wabigoon River GRAIN SIZE DISTRIBUTION

FIGURE B10



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-08	4.27	388.72
■	LWR-08	8.84	384.15
▲	LWR-09	10.36	382.60

Date May 2012

W.P.# 470-00-00



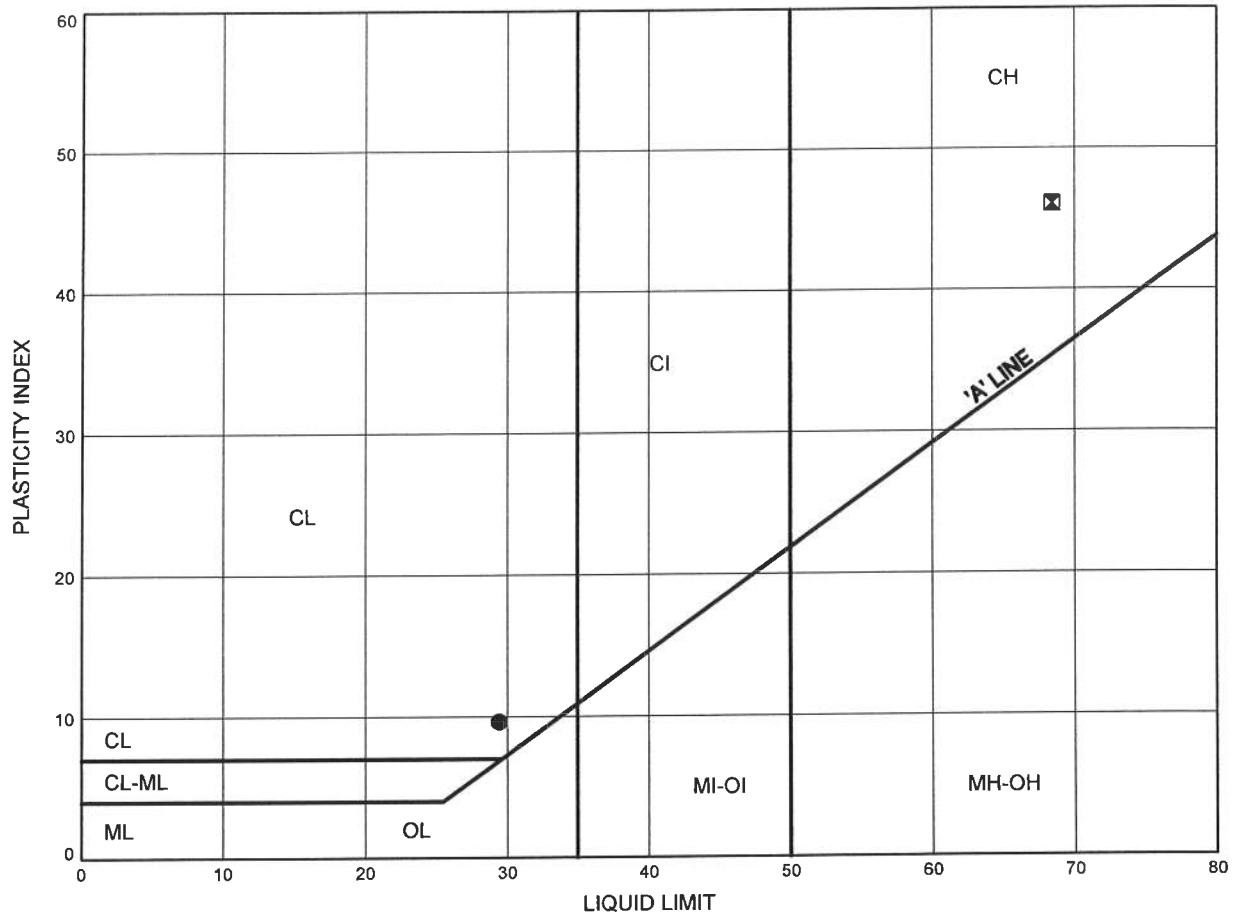
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Little Wabigoon River
ATTERBERG LIMITS TEST RESULTS

FIGURE B11

CLAYEY SILT to SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-01	10.97	382.17
⊠	LWR-03	10.36	382.69

Date May 2012
 W.P.# 470-00-00

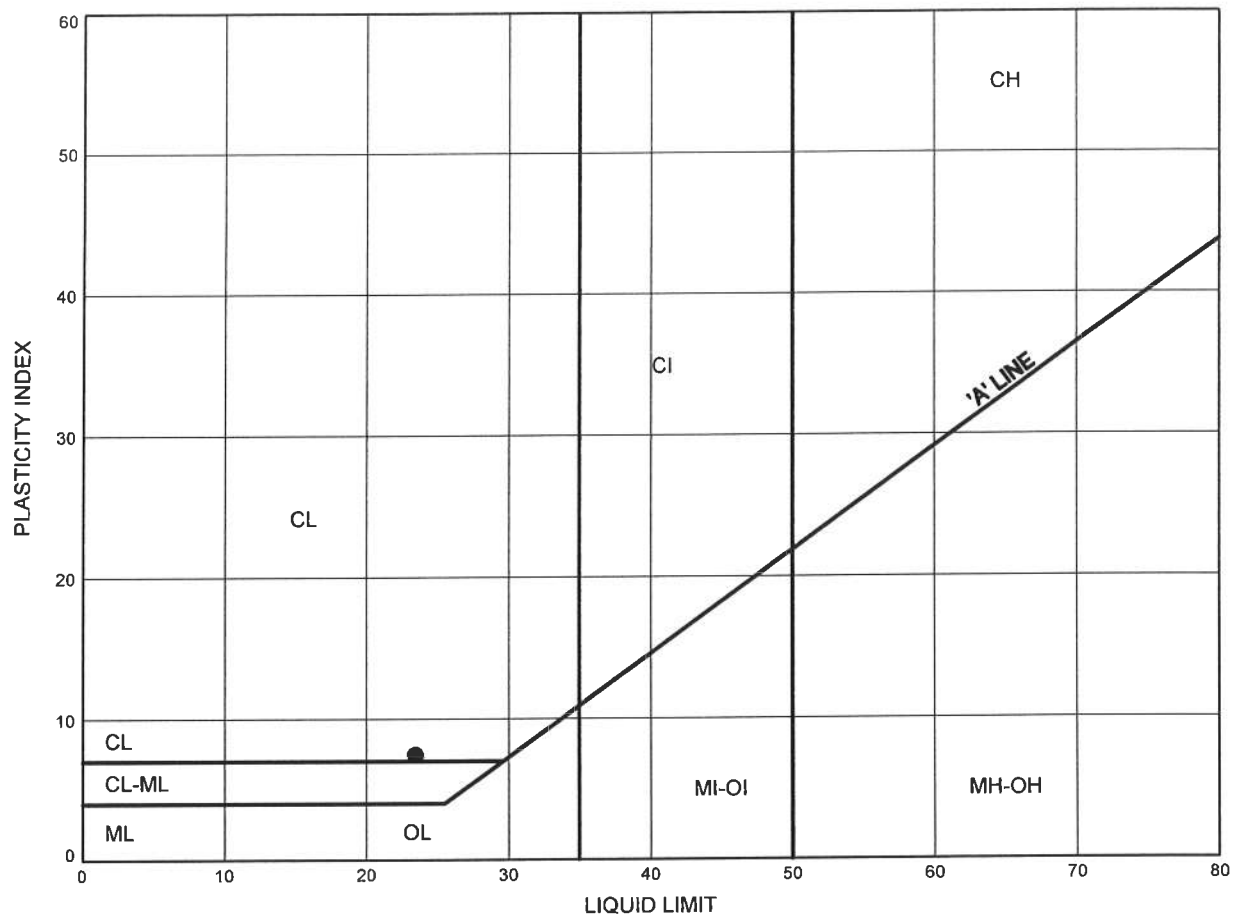


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Little Wabigoon River
ATTERBERG LIMITS TEST RESULTS

FIGURE B12

SILT



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	LWR-08	4.27	388.72

Date May 2012

W.P.# 470-00-00



Prep'd AN

Chkd. RPR



THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No : 19-1605-121 Client : HMM
Date Drilled : August 10,2011
Project Name : Little Wabigoon River Bridge Date Tested : September 06,2011
Core Size : NQ BH No : LWR-08 Tester : DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	10.4	D	22.3	47.3	66.4	233.6	Granodiorite to granite	Very Strong
2	1	10.9	D	19.0	47.3	63.8	199.3	Granodiorite to granite	Very Strong
3	1	11.5	D	26.5	47.2	57.9	278.5	Granodiorite to granite	Extremely Strong
4	1	12.5	D	23.2	47.3	60.1	242.7	Granodiorite to granite	Very Strong
5	1	13.0	D	26.8	47.2	59.3	281.0	Granodiorite to granite	Extremely Strong
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



POINT LOAD TEST SHEET

Job No : 19-1605-121 Client : HMM
Date Drilled : August 12,2011
Project Name : Little Wabigoon River Bridge Date Tested : September 06,2011
Core Size : NQ BH No : LWR-04 Tester : DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	9.8	D	30.3	48.1	245.0	309.8	Granodiorite to granite	Extremely Strong
2	2	10.4	D	28.6	49.6	150.0	278.5	Granodiorite to granite	Extremely Strong
3	2	10.4	A	26.4	49.6	53.0	202.6	Granodiorite to granite	Very Strong
4	3	10.9	D	22.8	50.8	210.0	213.4	Granodiorite to granite	Very Strong
5	3	11.2	D	28.4	50.8	140.0	266.2	Granodiorite to granite	Extremely Strong
6	4	11.9	D	35.7	51.4	330.0	328.3	Granodiorite to granite	Extremely Strong
7	5	12.6	D	33.6	51.0	215.0	312.4	Granodiorite to granite	Extremely Strong
8	5	12.6	A	34.6	51.0	63.1	226.8	Granodiorite to granite	Very Strong
9	5	12.8	D	35.6	51.0	420.0	331.6	Granodiorite to granite	Extremely Strong
10									
11									
12									
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28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



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POINT LOAD TEST SHEET

Job No : 19-1605-121 Client : HMM
Date Drilled : August 12, 2011
Project Name : Little Wabigoon River Bridge Date Tested : September 06, 2011
Core Size : NQ BH No : LWR-03 Tester : DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	19.6	D	30.5	47.5	60.7	317.5	Granodiorite to granite	Extremely Strong
2	1	20.1	D	37.4	47.5	64.1	389.4	Granodiorite to granite	Extremely Strong
3	1	20.7	D	18.7	47.5	61.4	194.3	Granodiorite to granite	Very Strong
4	1	21.2	D	22.7	47.5	56.9	235.9	Granodiorite to granite	Very Strong
5	1	21.6	D	6.6	47.4	58.7	68.4	Granodiorite to granite	Strong
6	2	21.8	D	16.3	47.5	60.0	169.1	Granodiorite to granite	Very Strong
7	2	22.4	D	33.2	47.5	63.1	345.1	Granodiorite to granite	Extremely Strong
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9									
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30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.

Appendix C
Site Photographs



Photograph 1 – Highway 17 and Little Wabigoon River Bridge crossing (east side of bridge)



Photograph 2 – Highway 17 and Little Wabigoon River Bridge crossing (west side of bridge)



Photograph 3 – Existing conditions of the Little Wabigoon River Bridge structure



Photograph 4 – Existing conditions of the Little Wabigoon River Bridge structure



Photographs 5 and 6 – Existing conditions of the Little Wabigoon River Bridge structure

Appendix D
Foundation Comparison

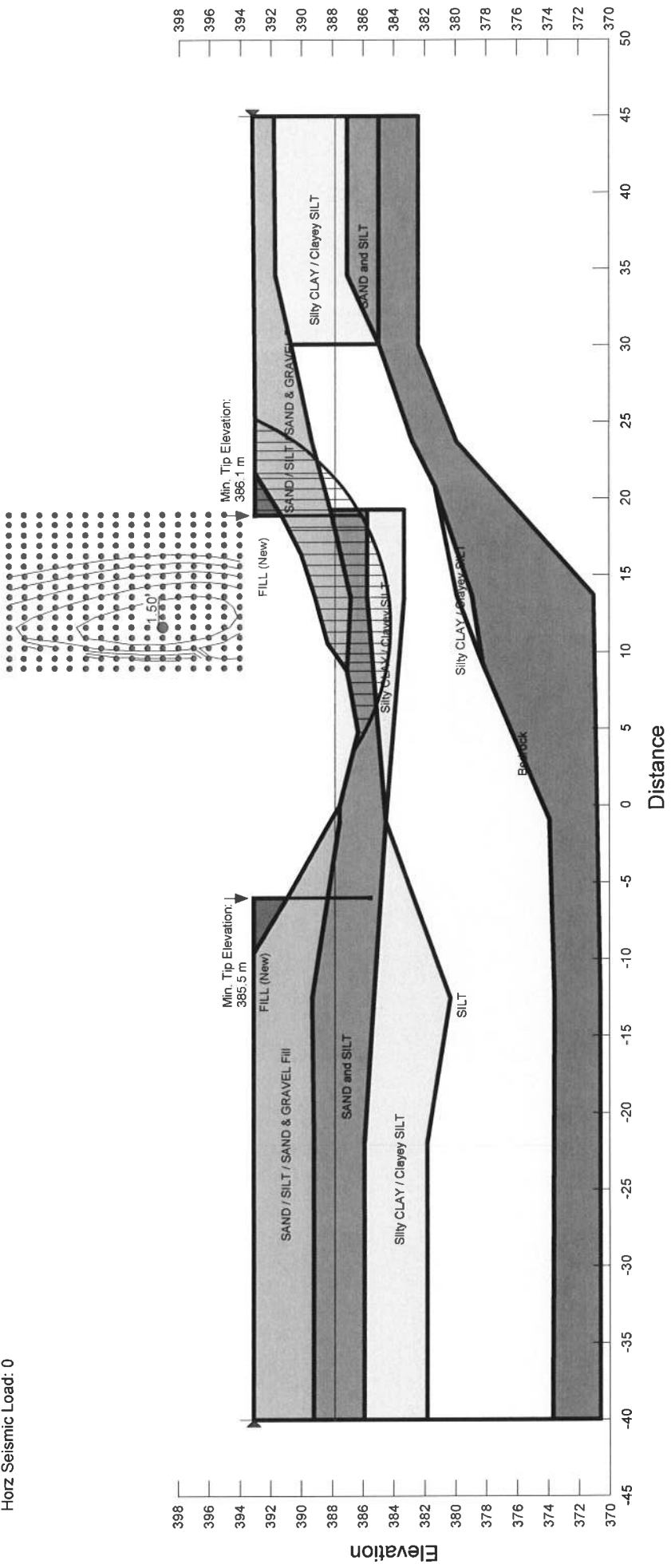
COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

H-Piles Driven to Bedrock	Footings on Native Soil	Caissons
<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by driving piles to achieve resistance on the bedrock or refusal. ii. Installation of piles could continue in freezing weather. iii. Foundation construction may require less volume of excavation than footings. iv. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Pile lengths required to achieve design resistance may vary. iii. Pre-augering will be required if boulders and/rockfill are encountered during pile driving. <p style="text-align: center;">RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Low available geotechnical resistance in native cohesionless deposits. ii. Potential for settlements. iii. Dewatering will be required due to the high groundwater levels. iv. Potential disturbance of river during excavation. <p style="text-align: center;">NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Construction of caissons could continue in freezing weather. ii. High geotechnical resistance available for units founded on bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher cost than spread footings ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table. iii. Potential difficulty in cleaning and inspecting bases. <p style="text-align: center;">NOT RECOMMENDED</p>

Appendix E
Slope Stability Output

Title: Little Wabigoon River Bridge Replacement
Name: East Abutment
Description: Long Term Analysis
Comments: HWY 17 - Bridge and Culvert Rehabs NWR
Last Solved Date: 10/16/2012, 1:57:18 PM
Method: Morgenstern-Price
Interslice force function option: Half-Sine
Minimum Slip Surface Depth: 1 m
Horz Seismic Load: 0

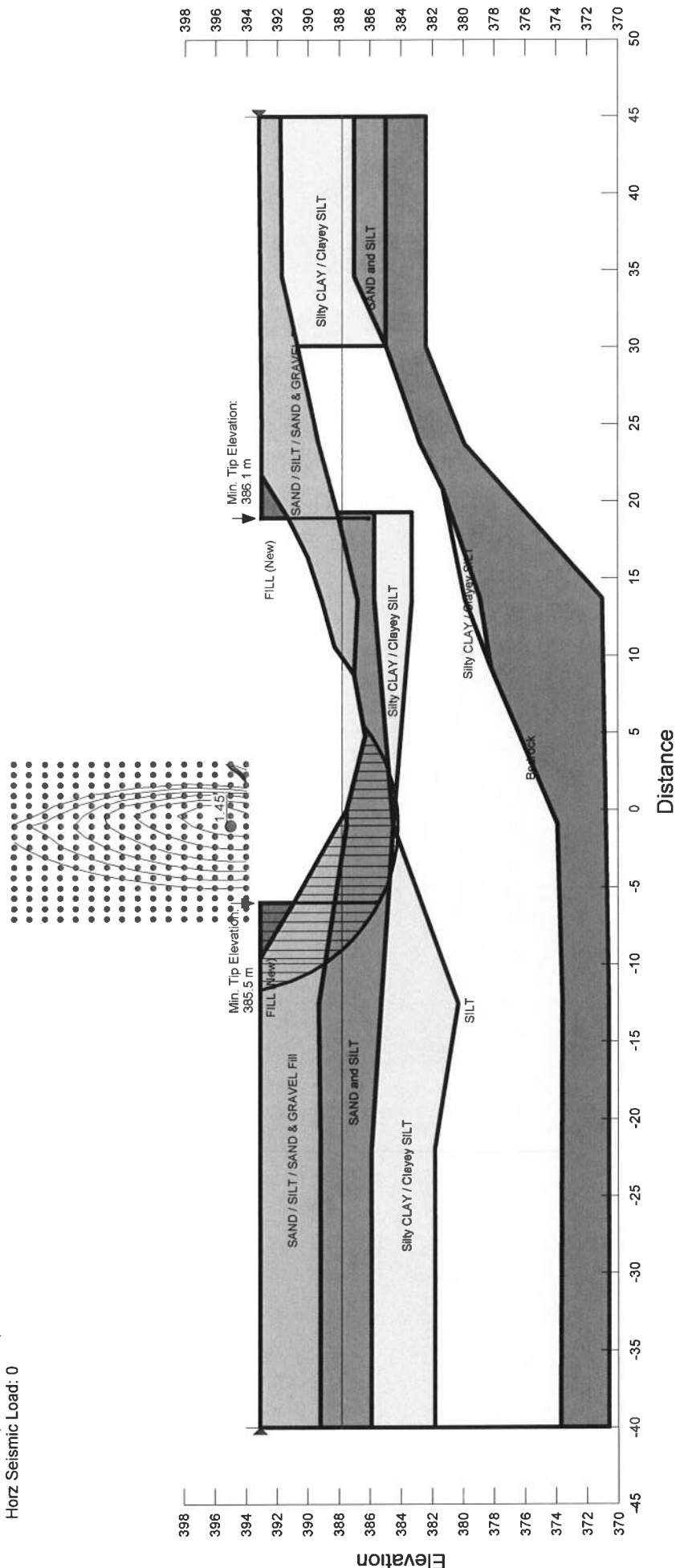
FILL (New)	21 kN/m³	0 kPa	32 °	1
SAND / SILT / SAND & GRAVEL Fill	20 kN/m³	0 kPa	32 °	1
SAND and SILT	20 kN/m³	0 kPa	32 °	1
Silty CLAY / Clayey SILT	18 kN/m³	0 kPa	28 °	1
SILT	20 kN/m³	0 kPa	28 °	1
Bedrock				



Title: Little Wabigoon River Bridge Replacement
Name: West Abutment
Description: Long Term Analysis
Comments: HWY 17 - Bridge and Culvert Rehabs NWR
Last Solved Date: 10/16/2012, 2:17:40 PM

Method: Morgenstern-Price
Interslice force function option: Half-Sine
Minimum Slip Surface Depth: 1 m
Horz Seismic Load: 0

FILL (New)	21 kN/m³	0 kPa	32 °	1
SAND / SILT / SAND & GRAVEL Fill	20 kN/m³	0 kPa	32 °	1
SAND and SILT	20 kN/m³	0 kPa	32 °	1
Silty CLAY / Clayey SILT	18 kN/m³	0 kPa	28 °	1
SILT	20 kN/m³	0 kPa	28 °	1
Bedrock				



Appendix F

List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

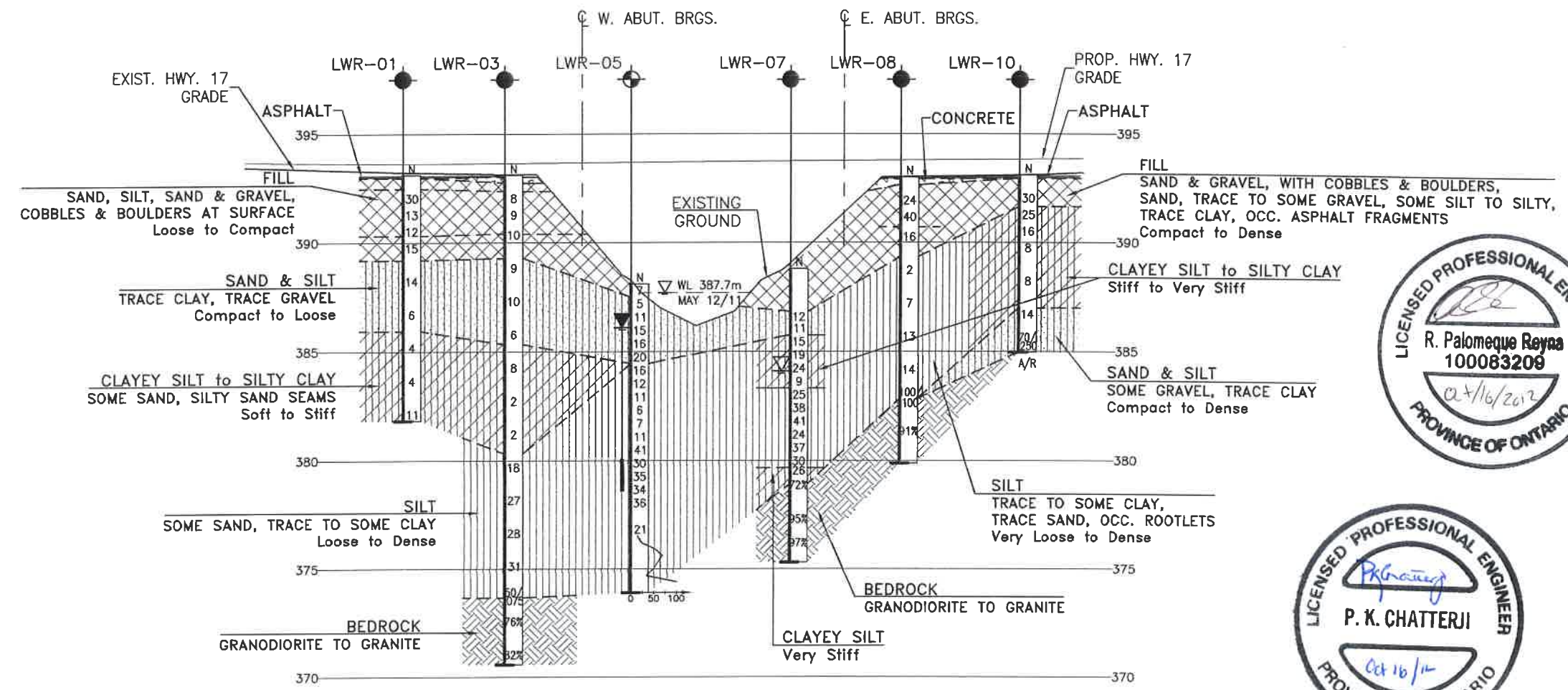
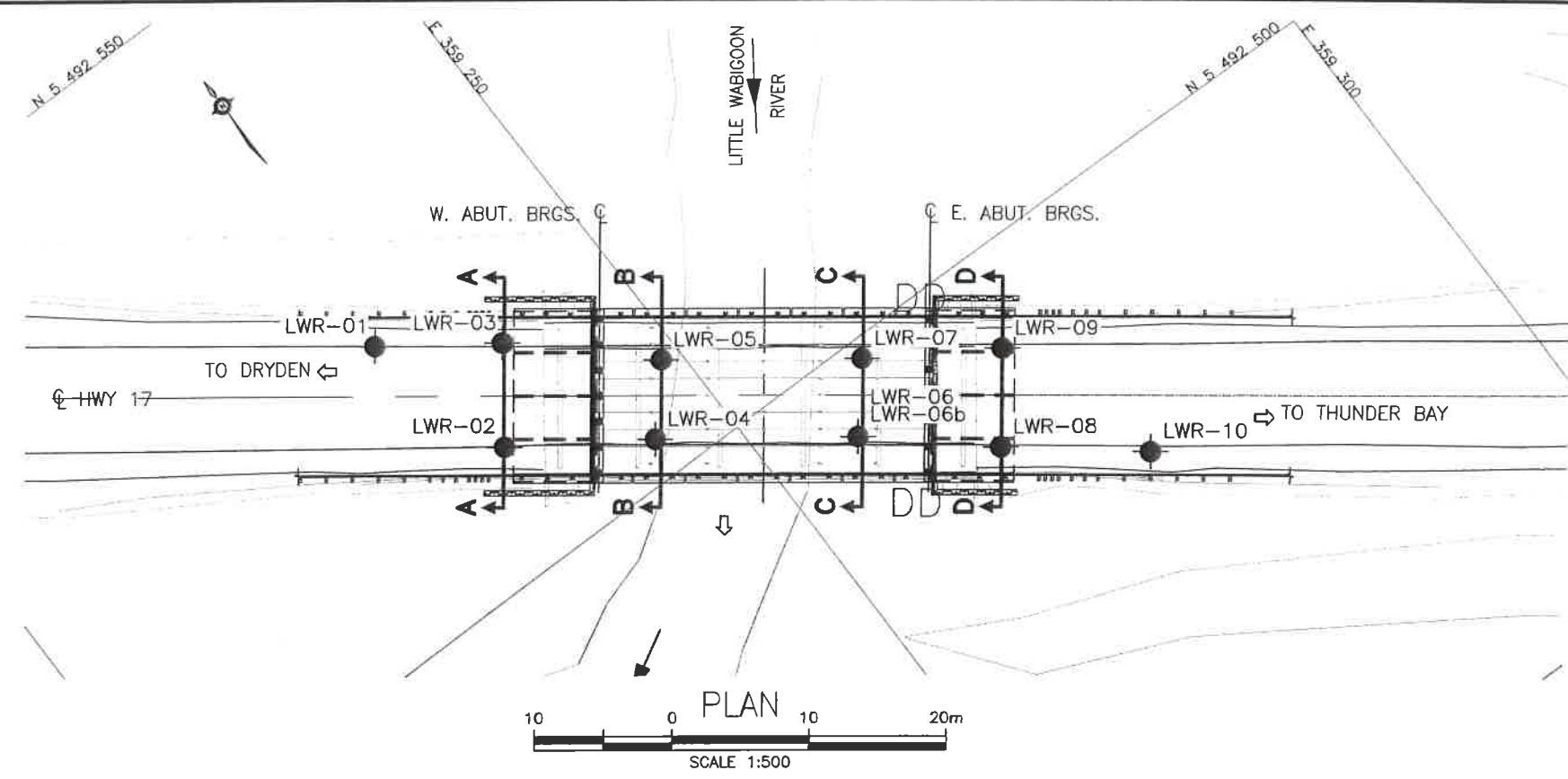
- OPSS 903
- OPSD 803.010
- OPSS 501
- OPSS 804
- OPSS 902
- OPSS 539

2. H-Piles founded on sloping bedrock

- All piles must be fitted with Titus Rock Injector points.
- “Piles to be driven to bedrock”.
- Upon initial contact with the bedrock:
 1. Apply 10 blows at 10% of the hammer energy. Record the penetration.
 2. Apply 10 blows at 50% of the hammer energy. If the penetration under 10 blows is less than 12.5 mm, the pile is set.
 3. If the penetration under 10 blows is greater than 12.5 mm, refer the issue to the design team for resolution.”
- The QVE must terminate driving before the pile is damaged by overdriving.

Appendix G

Drawing Borehole Locations and Soil Strata



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

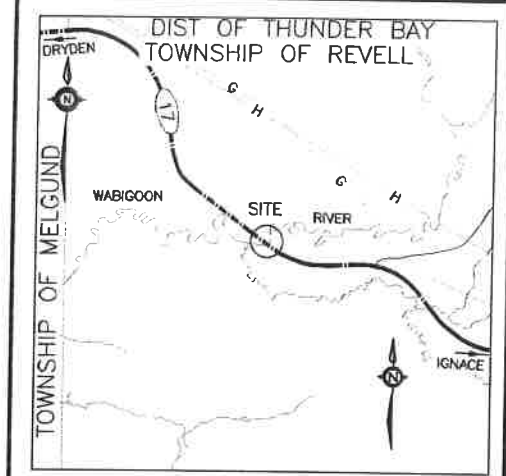
CONT No 2012-6013
WP No 470-00-01

LITTLE WABIGOON
RIVER BRIDGE
STRUCTURAL REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
33

**Hatch Mott
MacDonald**

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LEGEND

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

NO	ELEVATION	NORTHING	EASTING
LWR-01	393.1	5 492 520.8	359 232.7
LWR-02	393.1	5 492 509.1	359 235.7
LWR-03	393.1	5 492 515.3	359 240.2
LWR-04	387.9	5 492 502.9	359 244.7
LWR-05	388.1	5 492 507.3	359 248.6
LWR-06	388.8	5 492 494.1	359 256.6
LWR-06B	393.0	5 492 494.1	359 256.6
LWR-07	388.8	5 492 498.6	359 260.3
LWR-08	393.0	5 492 487.2	359 264.4
LWR-09	393.0	5 492 493.0	359 268.9
LWR-10	393.1	5 492 480.4	359 272.8

- NOTES-**
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
 - This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRE No. 52F-39



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	RRP	CHK RPR	CODE CAN/CSA S6-06 LOAD CI-625-001 DATE OCT. 2012
DRAWN	AN	CHK RPR	SITE 415-57 STRUCT DWG 2

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

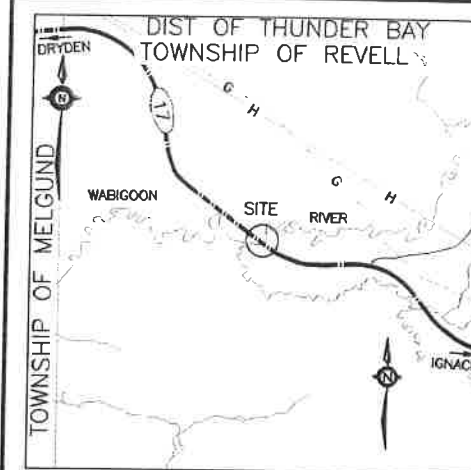
CONT No 2012-6013
WP No 470-00-01

LITTLE WABIGOON
RIVER BRIDGE
STRUCTURAL REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
34

**Hatch Mott
MacDonald**

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

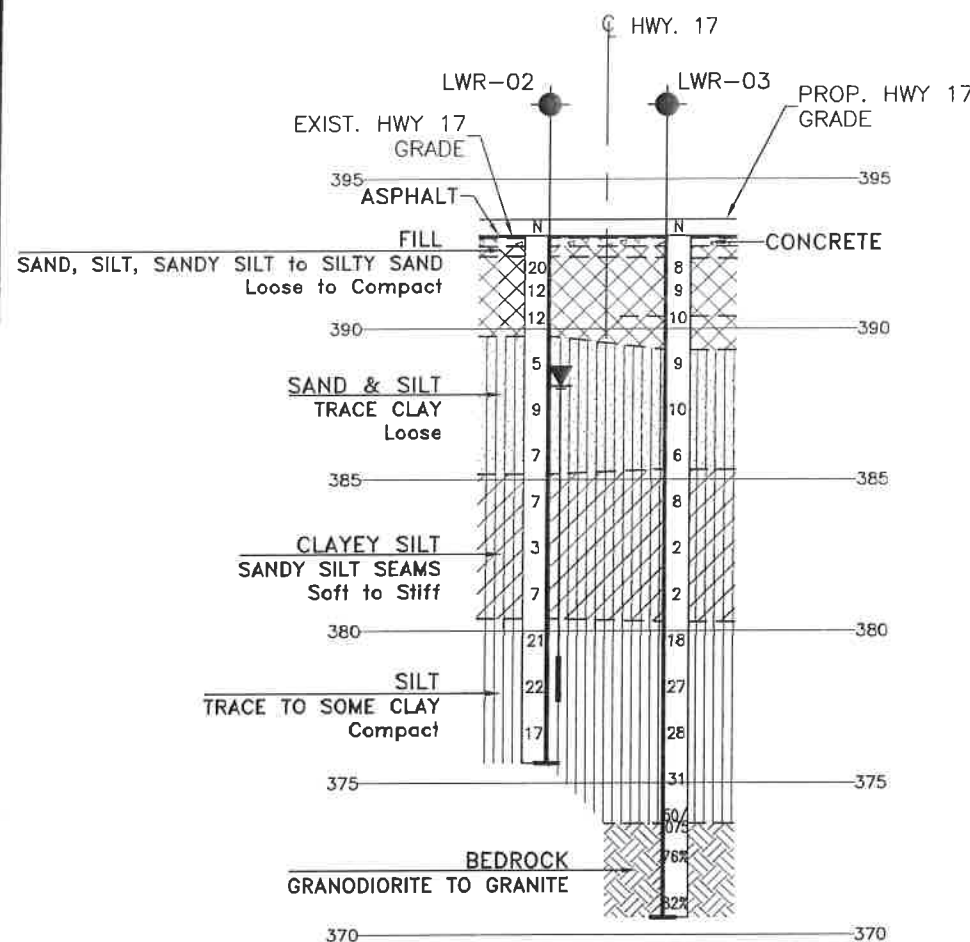
◆	Borehole
◆	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60' Cone, 475J/blow)
PH	Pressure, Hydraulic
W	Water Level
↑	Head Artesian Water
⊥	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
LWR-01	393.1	5 492 520.8	359 232.7
LWR-02	393.1	5 492 509.1	359 235.7
LWR-03	393.1	5 492 515.3	359 240.2
LWR-04	387.9	5 492 502.9	359 244.7
LWR-05	388.1	5 492 507.3	359 248.6
LWR-06	388.8	5 492 494.1	359 256.6
LWR-06B	393.0	5 492 494.1	359 256.6
LWR-07	388.8	5 492 498.6	359 260.3
LWR-08	393.0	5 492 487.2	359 264.4
LWR-09	393.0	5 492 493.0	359 268.9
LWR-10	393.1	5 492 480.4	359 272.8

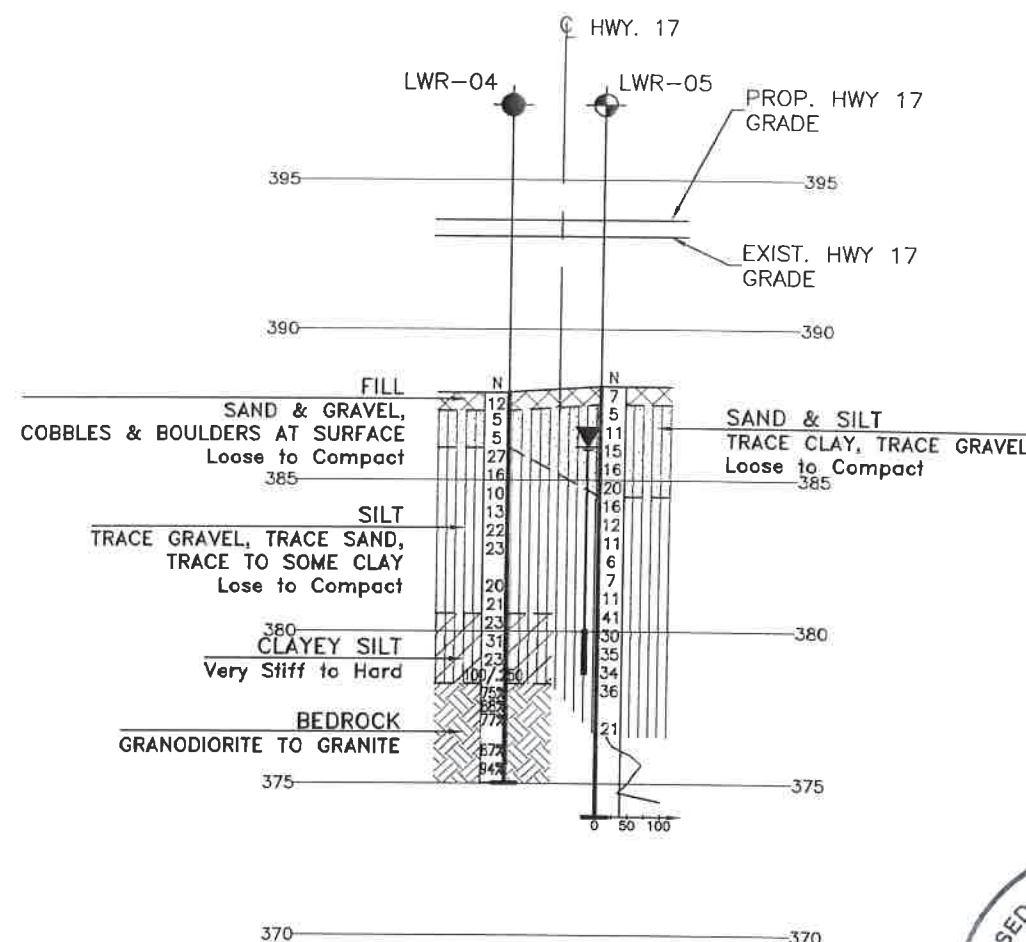
NOTES

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

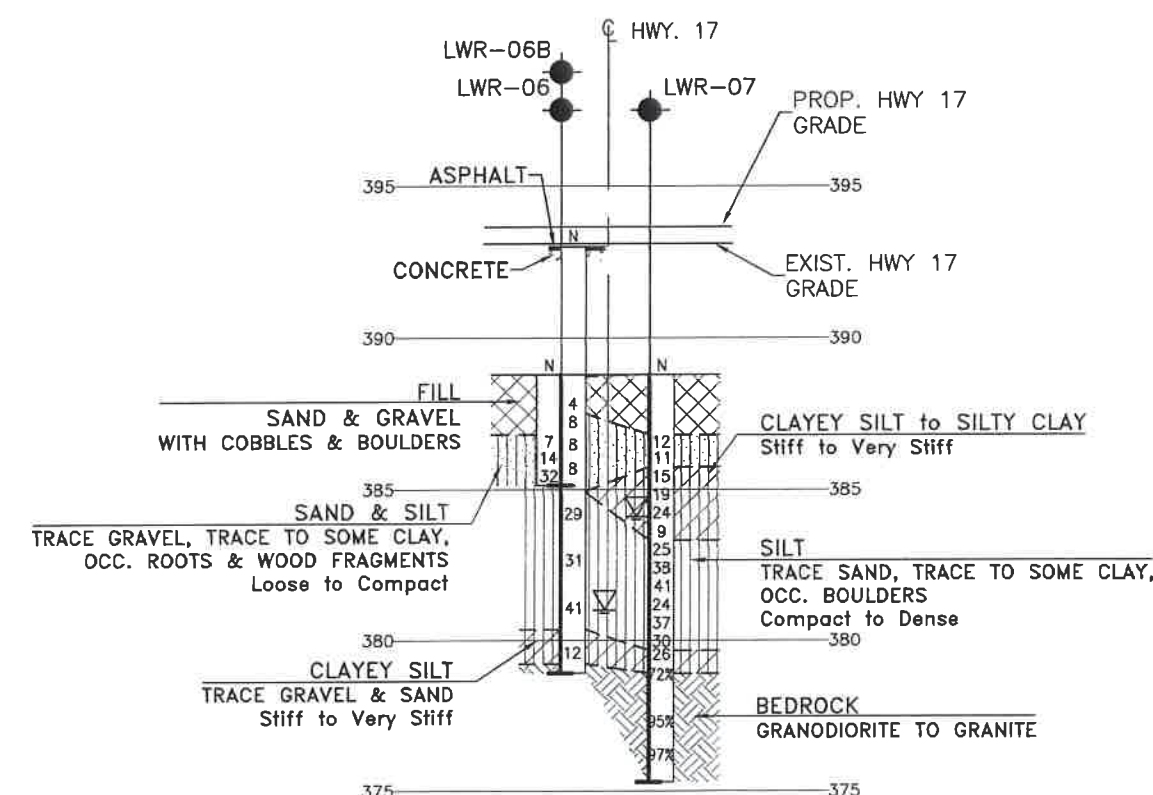
GEOCRES No. 52F-39



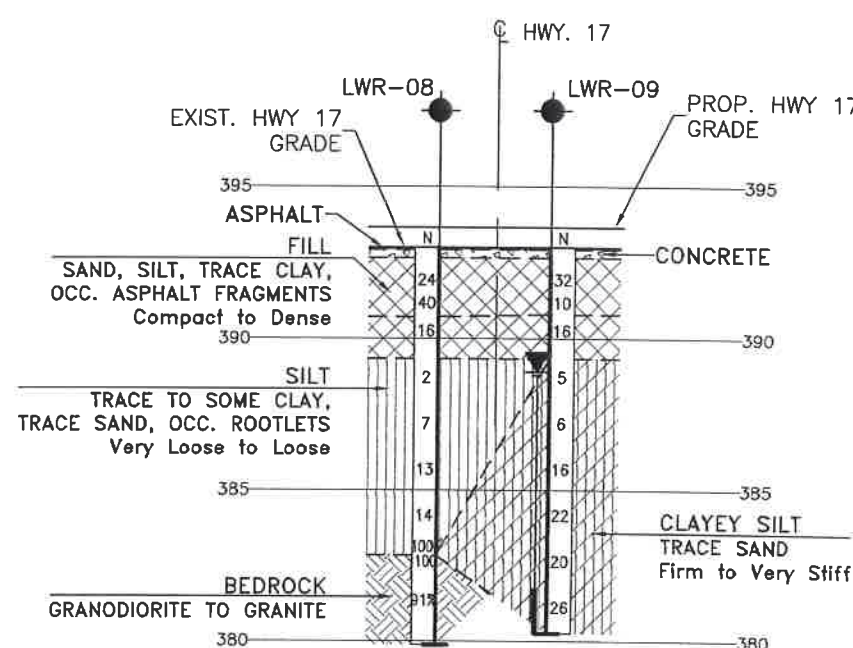
SECTION ALONG A-A



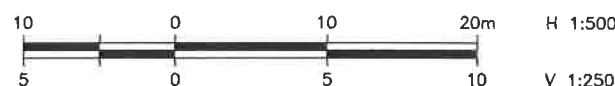
SECTION ALONG B-B



SECTION ALONG C-C



SECTION ALONG D-D



REVISIONS	DATE	BY	DESCRIPTION
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DRAWN	AN	CHK RPR	SITE 415-67 STRUCT DWG 3