FOUNDATION INVESTIGATION AND DESIGN REPORT
PINE RIVER BRIDGE REPLACEMENT
HIGHWAY 61, DISTRICT OF THUNDER BAY, ONTARIO
W.P. 6098-10-01, SITE #48W-105

Geocres Number: 52A-196

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted for the proposed replacement of the Pine River Bridge on Highway 61, in the Thunder Bay District, Ontario.

The purpose of the investigation was to explore the subsurface conditions at the site, and based on the data obtained, to provide a borehole location plan, record of borehole sheets, a stratigraphic profile, 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 (HMM), under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0010.

2 SITE DESCRIPTION

The bridge site is located on Highway 61 approximately 39 km south of Thunder Bay. The Pine River flows meandering easterly into the Lake Superior. The existing bridge is a single span structure with a span length of 24.4 m. A total length of 40.9 m between the ends of the wing walls and 11.1 m bridge deck width was indicated on the archive design drawings dated July 1949. The existing approach embankments are approximately 4.5 m in height.

The land surrounding the site is treed and undulating with low hills in the vicinity. Photographs of the bridge and surrounding area are presented in Appendix C.

The site lies within the physiographical region known as the Animikie Basin of the Southern Province, which is characterized by sedimentary rock of the Rove Formation. According to Ontario Geological Survey (OGS) data, the bedrock at this site generally consists of black shale, siltstone, greywacke and limestone. The bedrock is overlain by glaciolacustrine and quiet basin deposits of the Pleistocene age consisting of silts and clays with minor sands.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing at this site were carried between October 29 and November 3, 2014. A total of four boreholes, denoted as PINE-01, PINE-02, PINE-05, and PINE-06, were advanced to depths ranging from 9.8 m to 45.0 m below the existing highway embankment. Two dynamic cone penetration tests denoted PINE-03 and PINE-04 were advanced to 24.1 m each, to supplement the sampled borehole information. Details of the borehole locations, drilling depths and completion details are summarized in Table 3.1 below. It is noted that the investigation was programmed on the basis of a bridge rehabilitation anticipated at the time of drilling.

Table 3.1 – Field Work Summary

Location	Boreholes	Drilling and Coring Depth/ Base of Hole Elevation (m)	Completion Details
South Approach	PINE -01	9.8 / 205.3	Borehole backfilled with bentonite holeplug and cuttings to 0.3 m, concrete mix to 0.05 m then asphalt to surface.
South Abutment	PINE-02	45.0 / 170.0	Borehole backfilled with bentonite holeplug to 36.6 m, holeplug and cuttings to 0.3 m, concrete to 0.1 m, then asphalt patch to surface.
	PINE-03	24.1 / 190.9	Dynamic cone penetration test.
North Abutment	PINE-04	24.1 / 190.9	Dynamic cone penetration test.
	PINE-05	32.3 / 182.7	Borehole backfilled with bentonite holeplug to 4.6 m, cuttings to 0.3 m, concrete to 0.1 m then asphalt cold patch to surface. Moved 1.5 m north, augered to12.2 m (Elev. 202.8) and installed standpipe piezometer consisting of 19 mm diameter Schedule 40 PVC pipe with a 3 m slotted screen.
North Approach	PINE-06	10.2 / 204.9	Borehole backfilled with bentonite holeplug and cuttings to 0.3 m, dry cement to 0.1 m then asphalt cold patch to surface.

The locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawings included in Appendix G.

All boreholes were advanced using a CME55 truck-mounted drill rig in combination with hollow stem augers and NW casing/tri-cone methods to advance the boreholes in the overburden. Samples of the encountered soils were obtained from the boreholes at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). The field vane in the N-size was used to obtain in-situ undrained shear strength of the cohesive soils.

Borehole PINE-02 was advanced through a till layer with cobbles and boulders (above bedrock) by coring using rock coring equipment in NQ size. Coring was continued to a depth of 5.2 m into the underlying bedrock. All rock cores were logged, and the values of Total Core Recovery (TCR), Solid Core Recovery (SCR) and Rock Quality Designation (RQD) were determined.

A member of Thurber's technical staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing. The ground surface elevations at the boreholes and borehole locations were obtained from the drawings provided from HMM.

Groundwater conditions in the open boreholes were observed during the drilling operations. A standpipe piezometer consisting of 19 mm PVC pipe with a slotted screen was installed adjacent to Borehole PINE-05. Following the final water level reading, the piezometer was decommissioned in general accordance with MOE Regulation 903.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets included in Appendix A. Selected samples were also subjected to grain size analysis and Atterberg Limits testing, and the results of this testing program are summarized on the Record of Borehole sheets in Appendix A and shown on the figures included in Appendix B.

Point load tests (PLT) were performed on selected intact rock core samples. Unconfined compressive strengths (UCS) of the rock cores correlated from the PLT results are shown on the Record of Borehole sheets in Appendix A and the results of the testing are enclosed in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A presenting details of the encountered soils. The model of the soil stratigraphy is illustrated on the "Borehole Locations and Soil Strata" drawing 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.

The subsurface stratigraphy encountered below the existing embankment fill at the site generally consists of glaciolacustrine cohesive and cohesionless deposits underlain by a glacial till and bedrock. The cohesive deposits consist of silty clays and clayey silts extending to depths between 26.8 m and 27.7 m. A cohesionless deposit of silt to clayey silt extends to the clayey silt till encountered at 34.3 m depth on the south side and to sandy gravel encountered at 30.6 m depth on the north side. The limestone bedrock was encountered at approximately 39.8 m depth in Borehole PINE-02 located on the south side of the river. Descriptions of the individual strata are presented below.

5.1 Asphalt

Asphalt pavement was encountered in all sampled boreholes, i.e., Boreholes PINE-01, PINE-02, PINE-05 and PINE-06. The thickness of the asphalt ranged from 50 to 225 mm.

5.2 Embankment Fill

Embankment fill was encountered below the asphalt in all sampled boreholes. The thickness of the fill ranged from 2.0 m to 4.1 m, with the base of the fill between Elev. 213.0 and Elev. 210.7.

The fill was in general cohesionless. The proportions of sand and gravel varied within the fill, and the material is classified as sand and gravel to gravelly sand. The fill contains varying fine fractions (silt and clay) and occasional cobbles.

In Boreholes PINE-01, the cohesionless fill extended to 2.3 m depth (Elev. 212.8) and was underlain by 1.3 m of clayey silt fill to a depth of 3.6 m (Elev. 211.5). This cohesive fill is probably the native silty clay reworked during embankment construction.

SPT ‘N’ values recorded in the cohesionless fill ranged from 9 to 52 blows per 0.3 m penetration, indicating a loose to very dense relative density. SPT values of 4 and 8 blows per 0.3 m of penetration were obtained in the clayey silt fill indicating a firm consistency.

Moisture contents of the granular fill ranged from 3 to 11% with typical values between 3 and 6%. The values of moisture content in cohesive fill were 25 and 34%.

The results of grain size analyses conducted on fill samples are provided on the Record of Borehole sheets in Appendix A, and are illustrated in Figure B1 of Appendix B. The results are summarized as follows:

Gravel	30 to 42%
Sand	50 to 54%
Silt & Clay	7 to 18%.

5.3 Upper Silty Clay

A reddish brown silty clay was encountered directly below the fill in all boreholes drilled at this site. The thickness of the upper silty clay fully penetrated in Borehole PINE-02 and PINE-05 were 8.6 m and 10.5 m, respectively. The lower boundary of the layer was encountered at 12.6 m and 14.8 m (Elev. 202.4 and 200.2) in Borehole PINE-02 and PINE-05, respectively. Boreholes PINE-01 and PINE-06 were terminated in the silty clay deposit at depths of 9.8 m (Elev. 205.3) and 10.2 m (Elev. 204.9).

SPT 'N' values recorded in the silty clay varied between zero blows per 0.3 m penetration (Weight of Rod to Weight of Hammer) to 21 blows per 0.3 m of penetration, however, typically the N values ranged from 3 to 6 blows per 0.3 m of penetration. The value of 21 blows per 0.3 m of penetration in the lower zone of the silty clay in Borehole PINE-05 indicate the probable presence of silt/clayey silt interlayers within this deposit. Field vane shear tests (VST) measured undrained shear strengths ranging from 34 to 85 kPa. Based on the SPT and VST data, the consistency of the silty clay varied from very soft to stiff, typically being firm.

Sensitivity of the silty clay, calculated as a ratio of undisturbed strength to remoulded strength, ranged from 3 to 10, suggesting that the silty clay is of normal to high sensitivity.

The results of grain size analyses conducted on samples of the upper silty clay are provided on the Record of Borehole sheets in Appendix A, and illustrated in Figure B2 of Appendix B. The results are summarized as follows:

Gravel	0%
Sand	0% to 6%
Silt	51% to 65%
Clay	35% to 49%

The results of Atterberg Limits tests conducted on samples of the upper silty clay are provided on the Record of Borehole sheets in Appendix A and are illustrated in Figure B5 of Appendix B. The results indicated that the deposit has liquid limits ranging from 34 to 39% and plasticity indices ranging from 13 to 18%., suggesting low to medium plasticity of the deposit. Natural moisture contents of the silty clay ranged from 23 to 40%.

5.4 Clayey Silt

A layer of brown clayey silt underlies the silty clay on the south side of the river. Trace of stratification was noted in this deposit. The deposit was 3.0 m thick with the lower boundary at 15.6 m depth (Elev.199.4).

One SPT 'N' value recorded in the clayey silt was 2 blows per 0.3 m penetration. Field vane shear tests (VST) measured undrained shear strengths of 38 to 63 kPa. Based on the SPT and VST data, the consistency of the silty clay varied from very soft to stiff.

Natural moisture contents of 22 to 38% were measured on samples of this deposit.

5.5 Lower Silty Clay

A layer of brown silty clay underlies the clayey silt in Borehole PINE-02 and the upper silty clay in Borehole PINE-05. The thickness of the deposit was approximately 12 m, and the base of the lower silty clay was encountered between 27.7 m and 26.8 m depth (Elev. 187.3 and 188.2).

SPT 'N' values recorded in the silty clay varied from 1 blows per 0.3 m penetration to 13 blows per 0.3 m of penetration, typically ranging from 5 to 8 blow per 0.3 m of penetration. A blow count of 1 blows per 0.3 m of penetration was obtained at Elev. 190 m in Borehole PINE-05. Undrained shear strengths ranging from 66 kPa to in excess of 100 kPa were measured by field vane shear tests (VST). Based on the SPT and VST data, the consistency of the silty clay varied from firm to very stiff.

Sensitivity of the silty clay ranged from 2 to 6, suggesting that the silty clay is, generally, of normal sensitivity to sensitive.

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

Gravel	0%
Sand	0% to 5%
Silt	22% to 31%
Clay	69% to 76%.

The results of Atterberg Limits tests conducted on samples of the lower silty clay are provided on the Record of Borehole sheets in Appendix A and illustrated in Figure B6 of Appendix B. The liquid limits ranged from 62 to 64% and plasticity indices ranged from 36 to 38%., suggesting high plasticity of the deposit. Natural moisture contents of the silty clay ranged from 32 to 51%.

5.6 Silt to Clayey Silt

In Boreholes PINE-02 and PINE-05, a silt with some clay to clayey silt was encountered underlying the lower silty clay below the depth of 27.7 m and 26.8 m. The deposit was dark greyish brown and varied in thickness from 3.8 m and 6.6 m. The lower boundary of the deposit was between 34.3 m and 30.6 m depth (Elev. 180.7 and 184.4).

SPT 'N' values recorded in the silt layer ranged from 7 to 19 blows per 0.3 m penetration, indicating a loose to compact relative density. Natural moisture contents were measured to be between 31 and 57%.

5.7 Sandy Gravel

A layer of dark grey sandy gravel was encountered in Borehole PINE-05 below 30.6 m depth (Elev. 184.4). The layer contained trace of silt and occasional cobbles. The borehole was advanced into this deposit for 1.7 m and terminated at 32.3 m depth (Elev. 182.7) upon encountering artesian groundwater.

The sandy gravel was very dense as indicated by one SPT 'N' value of more than 100 blows per 0.3 m penetration recorded at the base of the deposit. A natural moisture content of 10% was measured on one sample of this deposit.

The results of a grain size analysis conducted on one sample of the sandy gravel are provided on the Record of Borehole sheets in Appendix A and are plotted in Figure B4 of Appendix B. The results are summarized as follows:

Gravel	65%
Sand	27%
Silt & Clay	8%.

5.8 Clayey Silt Till

A layer of clayey silt till was encountered below the silt to clayey silt in Borehole PINE-02. The clayey silt till was dark greyish brown in colour and contained cobbles and boulders. The frequency of occurrence of the cobbles and boulders in this deposit increased significantly below 37.5 m depth (Elev. 177.5), and the borehole was advanced further by coring to confirm anticipated bedrock. The till layer was fully penetrated in that borehole and was 5.5 m thick. The lower boundary of the till was at 39.8 m depth (Elev. 175.2).

One SPT ‘N’ value recorded in the deposit was 24 blows per 0.3 m penetration, indicating a very stiff consistency. A natural moisture content of 22% was obtained for one sample of this deposit.

5.9 Bedrock

Bedrock was encountered beneath the bouldery zone in the clayey silt till at 39.8 m depth (Elev.175.2).

The bedrock is described as limestone, moderately weathered to fresh with trace of calcite, grey with light grey specking. Highly fractured zones were encountered in the bedrock below 42.2 m depth. Clay infilling and possible sand and clay seams were inferred during coring operations near the end of the borehole (in Runs 3 and 4).

In the two upper rock cores, the measured Total Core Recovery (TCR) was 100% and 88%, and the Rock Quality Designation (RQD) was 100% and 77%, indicating good to excellent rock quality. In the two lower rock cores, the measured TCR was 50% and 25%, with an RQD of 0% for both cores, indicating a very poor rock quality.

Borehole PINE-02 was terminated at 45.0 m depth (Elev. 170.0 m).

The unconfined compressive strength (UCS) of the rock, estimated from the results of point load tests conducted on the rock core samples, ranges from 67 to 119 MPa, indicating a strong to very strong intact rock. Average values of 71 and 94 MPa were obtained for core runs #1 and #2. The point load test results are included on the Record of Borehole sheets in Appendix A, and the point load test sheet with details of testing is enclosed in Appendix B.

5.10 Water Levels

The water levels in the boreholes were measured upon completion of drilling operations. Since water was used during the wash-boring and coring operations, the measured water levels may not reflect prevailing groundwater levels at the site.

An artesian water condition was noted upon completion of drilling in Boreholes PINE-02 and PINE-05. The water head of 2.1 m and 1.7 m above the ground level/embankment grade was measured at the commencement of removal of casings from those boreholes at approximately 45.0 m and 32 m depth. It is probable that the sandy gravel and bouldery zone of the till or fractured bedrock are water bearing strata in this area. The artesian water was sealed at the source, and the boreholes were decommissioned.

Borehole PINE-05P was drilled adjacent to Borehole PINE-05 to install a standpipe piezometer for monitoring of groundwater level after drilling. The water levels measured in the open boreholes upon completion of drilling and in the piezometer are summarized in Table 5.1.

Table 5.1: Water Level Measurements

Boreholes	Date	Water Level Depth/Elevation
PINE -01	Nov. 2, 2014	Borehole open to 6.1 m and dry.
PINE-02	Oct. 30, 2014	Upon completion of rock coring, artesian water condition observed in bedrock; water head at 2.1 m above the road level. Borehole sealed and decommissioned.
PINE-05	Oct. 31, 2014	Artesian condition observed in sandy gravel at 32.3 m depth (Elev. 182.7); water head at 1.7 m above the road level. Borehole sealed and decommissioned.
		Piezometer installed to 12.2 m depth:
	Nov. 1, 2014	2.0 / 213.0
	Nov. 2, 2014	2.3 / 212.7
	Nov. 3, 2014	2.3 / 212.7
PINE-06	Nov. 3, 2014	Borehole open to 3.7 m and dry.

The approximate water level in the river shown on the preliminary GA drawing is at Elev. 209.0 on July 6/2012. The water level in the river and groundwater levels are expected to fluctuate seasonally and are subject to precipitation patterns, and may vary from the levels presented above.

6 MISCELLANEOUS

Eastern Ontario Diamond Drilling supplied the drill rig and conducted the drilling, sampling and in-situ testing operations. A truck-mounted CME #55 drill rig was used for the duration of the investigation.

The drilling and sampling operations were supervised in the field by Mr. Matthew Whalen of Thurber. Mr. Mark Farrant, P.Eng. directed the field operations.

The report was prepared by Ms. Anna Piascik, P.Eng., and reviewed by Mr. Murray Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects.

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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 provides geotechnical recommendations to assist the design team in selecting and designing a suitable foundation system for the proposed replacement bridge.

At present, Highway 61 crosses the Pine River on a single span structure with a span length of 24.4 m (a total length of 40.9 between ends of wing walls) and with the deck width of 11.1 m. Based on the archive design drawings, each bridge abutment is supported on 39 timber piles 0.3 m in diameter and 12.2 m in length. A pile cut-off elevation of 208.0 is specified on the drawings, indicating a tip elevation of 195.8. The existing approach embankments are approximately 4.5 m in height with the top elevation of approximately 215.0 on each side of the bridge.

The preliminary General Arrangement drawing indicates that the replacement bridge will be a single span structure with a span of 31.0 m and width of 13.9 m supported on H-piles. The existing road grade will be raised by 630 mm at the south abutment and by 930 mm at the north abutment. Sheet pile walls are proposed at both abutments to retain the approach fill. The new abutments will be installed behind the footprint of the existing abutment foundations.

The discussion and recommendations presented in this report are based on information provided by Hatch Mott MacDonald (HMM) and on the factual data obtained in the course of this investigation. The existing MTO foundation reports (Geocres Nos. 52A-112 and 52A-120) prepared for the sites located in the general area, however remotely to the Pine River Bridge, were reviewed to aid in appreciation of the soil conditions at the Pine River Bridge.

8 STRUCTURE FOUNDATIONS

In general, the site is underlain by glaciolacustrine cohesive and cohesionless deposits extending to significant depths and overlying relatively thin glacial till and bedrock.

The cohesive deposits consisting predominantly of silty clay extend to as much as 27.7 m depth (Elev. 187.3). The upper silty clay extending to approximately 15 m depth (Elev. 200) is typically of low to medium plasticity; the lower deposit is highly plastic/compressible. The consistency of the deposit varies from very soft to very stiff. The silty clay is underlain by a cohesionless silt to clayey silt which in turn overlies the clayey silt till encountered at 34.3 m (Elev. 180.7) on the south side of the river and sandy gravel at 30.6 m depth (Elev. 184.4) on the north side of the river. The limestone bedrock was encountered beneath the till at 39.8 m depth (Elev. 175.2) in one borehole located in the area of the south abutment.

The river level was indicated on the General Arrangement drawing at Elev. 209.0 on July 6, 2012. Groundwater levels in the piezometers installed at Borehole PINE-05 were measured at 2.3 m depth (Elev. 212.7). Groundwater with artesian pressure (water head at 2.1 and 1.7 m above the embankment grade) was encountered in Boreholes PINE-02 and PINE-05, respectively, during drilling operations at the base of boreholes, i.e., below 32 m and 40 m depth.

Several foundation options were considered for this bridge, namely

- spread footings placed on native soil or engineered fill,
- driven steel H-piles, and
- augered caissons.

A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix D.

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

8.1 Spread Footings on Native Soil or Engineered Fill

The use of spread footings to support the abutments is not recommended given the relatively low geotechnical resistance available in the native soils and the potential for large consolidation settlement in the cohesive deposits.

8.2 Driven H-Pile Foundations

8.2.1 Axial Resistance

The ground conditions at the site are considered to be suitable for the use of steel H-piles driven to refusal on bedrock at the south abutment and very dense sandy gravel at the north abutment.

H-piles founded in the above strata should be designed using the recommended geotechnical capacities presented in Table 8.1. The ULS resistance for piles on bedrock has been reduced considering the possibility that some of the piles may encounter refusal above the bedrock surface in the hard clayey silt till with cobbles and boulders.

Table 8.1 – Recommended Geotechnical Resistance and Reaction for HP310x110

Foundation Element	Pile Tip Depth/Elevation (m)	Factored Geotechnical Resistance at ULS (kN) per pile	Geotechnical Reaction at SLS (kN) per pile
South Abutment	39.8 / 175.2	1600	N/A
North Abutment	32.3 / 182.7	1600	1400

The actual founding elevation for each pile may vary during installation.

Oversize materials (e.g. greater than 75 mm nominal diameter) must not be used for any new fill through which the piles will be driven.

8.2.2 Pile Tips

Pile tip protection is recommended for driven H-piles to prevent pile damage when setting the piles in the very dense/hard deposit or if cobbles or boulders are encountered. The tips of all driven H-piles at the abutments should be fitted with pile tip protection from an approved manufacturer such as Skyline Steel, Titus Steel (Standard H-point) or similar.

8.2.3 Pile Installation

Pile installation should be in accordance with OPSS 903.

Pile driving should be controlled in accordance with Standard Drawing SS103-11 (Hiley Formula) and an ultimate pile resistance should be specified by the designer. The Hiley formula need not be used until the piles are within 2.0 m of the design pile tip elevation. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile. "R" must have a minimum value of twice the design load at ULS.

If the proposed bridge design requires that the deviation at the top of the pile be limited to tight tolerance, a driving template or other means may be required to achieve the specified maximum deviation.

The new piles will be installed behind the existing bridge abutments and in close proximity to the pile caps. The cohesive soils extending from the river bottom to significant depths are soft and sensitive to disturbance; therefore it is important that the existing pile caps and timber piles are left in place.

Artesian groundwater conditions were encountered at approximately 32.3 m depth (Elev. 182.7) on the north side of the river and in bedrock on the south side of the river. Pile foundations will extend to the strata under artesian pressure. It is expected that adhesion and squeezing of the silty clay to the pile surfaces within the significant thickness of cohesive soils overlying the bearing strata will minimize upward water seepage along the soil-pile interface. Should any upward seepage continue around the pile after installation, it is recommended that a filter be installed around the top of the embedded portion of the piles to provide drainage and prevent loss of fines within the seepage flow. The filter should comprise compacted sand meeting the grading requirements for fine aggregates in OPSS 1002, be a minimum 600 mm thick, and be covered by rock protection.

8.2.4 Pile Lateral Resistance

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

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

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

Where z = depth of embedment of pile (m)

D = pile width or diameter (m)

n_h = coefficient of horizontal subgrade reaction (kN/m^3)

γ' = effective unit weight (kN/m^3)

K_p = passive earth pressure coefficient

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

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

$$p_{ult} = 9 \, S_u \qquad (\text{kPa})$$

Where S_u = undrained shear strength (kPa)

D = pile width or diameter in metres

The above equations and recommended parameters in Table 8.2 below may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance.

Table 8.2 – Soil Parameters for Lateral Pile Resistance

Soil Unit	Elevation (m)		γ' (kN/m³)	n_h (kN/m³)	K_p	S_u (kPa)
	Top	Bottom				
South Abutment						
Fill	GS	211.0	20	2,500	3.0	-
Upper Silty Clay	211.0	202.4	7	-	-	40
Clayey Silt	202.4	199.4	7	-	-	40
Lower Silty Clay	199.4	187.3	8	-	-	45
Silt/Clayey Silt	187.3	180.7	9	2,000	3.0	-
Clayey Silt Till	180.7	175.2*	11	-	-	150
North Abutment						
Fill	GS	210.7	20	2,500	3.0	-
Upper Silty Clay	210.7	200.2	7	-		40
Lower Silty Clay	200.2	188.2	8	-	-	45
Silt/Clayey Silt	188.2	184.4	9	2,000	3.0	-
Sandy Gravel	184.4	182.7*	11	6,000	3.5	-

* Pile tip elevations will vary at pile locations.

For analysis, the spring constant, K_s, may be obtained from the expression:

$$K_s = k_s \, L \, D \text{ (kN/m)},$$

where k_s = coefficient of horizontal subgrade reaction (kN/m³),
D = pile width (m), and
L = length (m) of the pile segment or element used in the analysis.

The ultimate lateral resistance, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} \, L \, D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

According to CHBDC Clause 6.8.7.1, Table C 6.4, the lateral resistance of an HP310x110 pile driven in those soil conditions should be limited to 120 kN at ULS, and 35 kN at SLS.

The modulus of subgrade reaction may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 8.3. Intermediate values may be obtained by linear interpolation.

Table 8.3 – Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre	Reduction Factor
Pile group oriented ***perpendicular*** to direction of loading	4D	1.0
	1D	0.5
Pile group oriented ***parallel*** to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

In the case of conventional abutments, i.e. not integral type, horizontal loads may be resisted by means of battered piles. Additional lateral resistance could also be provided by socketing the piles into bedrock. However, considering the depth to bedrock at this site, socketing of the piles is not expected to be an efficient means of developing lateral resistance.

8.3 Downdrag

The weight of the additional fill that is proposed to raise the grade of the approach embankments will induce consolidation of the underlying clay deposits. As a result, downdrag forces will develop along the length of abutment piles embedded in these deposits.

For design purposes, an unfactored downdrag load of 750 kN per pile should be used to evaluate the impact of downdrag load on the bearing capacities of the abutment piles.

This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C6.8.4 to obtain a factored downdrag load. In accordance with Section 6.8.4 of the CHBDC and Clause C6.8.4 of the Commentary, 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. The factored dead and downdrag load should not exceed the factored structural resistance of a pile.

In geotechnical analysis of downdrag load, the effect of live load should not be considered.

The location of the neutral plane for a pile or group of piles should be determined by using unfactored loads and unfactored geotechnical parameters.

To mitigate the downdrag loading on the abutment piles, the ultra-lightweight fill (EPS) could be used to compensate for the required earth fill thickness. The use of EPS fill is discussed further in Section 10 of this report.

8.4 Caissons / Drilled Shafts

In view of the presence of the relatively low strength compressible deposits extending to significant depth and the high water level, the use of caissons is not recommended and has not been developed herein.

8.5 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, steel H-piles driven into the hard till/sandy gravel is the preferred foundation option at this site.

8.6 Frost Cover

The depth of frost penetration at this site is approximately 2.2 m. The base of pile caps, if employed, must be provided with a minimum of 2.2 m of earth cover as protection against frost action.

8.7 Impact on Existing Foundations

Piles will be driven adjacent to the existing bridge for construction of the replacement bridge. The new foundation units should be positioned to avoid encountering the existing foundations during driving of the new piles.

Archive documents indicate that the timber piles supporting the existing bridge were driven into the underlying silty clay to approximate Elev. 195.8. Driving of the new piles may impact the performance of the existing pile foundations. Therefore, it is recommended that a monitoring program (including establishment of adequate benchmarks outside the zone of potential influence and acquirement of baseline readings in advance of construction) be implemented for the duration of foundation construction to identify any movement of the existing structure. Appropriate monitoring points and tolerable levels of movement should be specified by the structural designer. If movements exceed tolerable levels, the Contractor must be prepared to jack and/or shim the bridge structure. Suggested wording for an NSSP for monitoring of the existing structure during pile driving has been included in Appendix E.

9 SHEET PILE WALLS

The current design indicates that steel sheet pile walls will be installed adjacent to the pile foundations in lieu of conventional abutment walls. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill. The alignment of the sheet piles must be carefully selected to avoid the existing substructure as much as practical.

Lateral stability of the sheet pile walls should be checked by the wall designer using the parameters presented in Table 9.1. The coefficients of passive earth pressure (K_p) are provided for horizontal ground surface in front of the sheet pile wall. For sloping ground in front of the sheet pile wall, the recommended values for the coefficients of passive earth pressure (K_p) should be reduced.

Table 9.1 – Soil Parameters for Sheet Pile Analysis

Soil Unit	Elevation (m)		γ' (kN/m^3)	K_a	K_p	K_o
	Top	Bottom				
South Abutment						
Fill	GS	211.0	20	0.33	3.0	0.50
Upper Silty Clay	211.0	202.4	7	0.39	2.6	0.56
Clayey Silt	202.4	199.4	7	0.39	2.6	0.56
Lower Silty Clay	195.0	187.3	8	0.42	2.4	0.59
Silt/Clayey Silt	187.3	180.7	9	0.39	2.6	0.56
Clayey Silt Till	180.7	175.2	11	0.31	3.3	0.47
North Abutment						
Fill	GS	210.7	20	0.33	3.0	0.50
Upper Silty Clay	210.7	200.2	7	0.39	2.6	0.56
Lower Silty Clay	200.2	188.2	8	0.42	2.4	0.59
Silt/Clayey Silt	188.2	184.4	9	0.39	2.6	0.56
Sandy Gravel	184.4	182.7	11	0.28	3.5	0.44

Cobbles and boulders may be encountered during driving the sheet piles through the existing approach and embankment fill. Any rock fill material, if present at the bridge abutments, as well as any visible obstructions along the sides of the embankments should be removed prior to driving the sheet piles. Removal of the existing bridge wing walls will also be required, as indicated on the preliminary GA drawing. Tip protection is not considered necessary for these sheet piles.

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. The native soils in front of the sheet piles should be protected from river erosion so that the sheet piles do not lose lateral support.

Backfill to the sheet pile walls should be in accordance with OPSS 902 and should consist of Granular A, Granular B Type II or Granular B Type III material. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

10 APPROACH EMBANKMENTS

Based on the latest GA drawing and information from the designer, the existing road grade will be raised by 930 mm at the north abutment and by 630 mm at the south abutment.

10.1 Approach Embankment Stability

The new approach embankments will be retained by sheet pile walls to approximately 6 m behind the abutment walls. The foundation soils governing stability of the approach embankments consist of very soft to stiff (typically firm) silty clay.

Global stability analyses were carried out to assess the stability of the forward slopes with the proposed sheet pile wall configuration. The stability analyses were carried out using the commercially available slope stability program GEO-SLOPE, applying the Morgenstern-Price method. The geotechnical model and results of the analyses are shown on Figures 1 to 4 in Appendix F. The estimated factors of safety (FOS) are summarized in Table 10.1.

Table 10.1 - Estimated Factors of Safety for Approach Embankments

Abutment	**Condition**	**Sheet Piles**	**Factor of Safety**	**Figure (Appendix F)**
North	Total stress analysis - Short term	Yes	1.50	1
	Effective Stress Analysis - long term	Yes	1.52	2
South	Total stress analysis - Short term	Yes	1.51	3
	Effective Stress Analysis - long term	Yes	1.54	4

The estimated Factors of Safety generally meet or exceed the minimum value of 1.5 normally accepted for this type of analysis under short and long term conditions, respectively, provided the sheet piles are driven to the following tip elevations:

North Abutment Elev. 196.0

South Abutment Elev. 197.0

The above noted depths of penetration were derived based on the slope stability criteria. The depths of sheet pile penetration must be checked to ensure that requirements for the lateral stability are fulfilled.

Behind the sheet pile wall abutments, the side slopes of the approach embankments will be sloped at an inclination of 2 horizontal to 1 verical (2H:1V). A grade raise of 930 mm and 630 mm is proposed for the north and south embankments, respectively. Slope stability analyses were completed for the more critical north approach embankment with the proposed grade raise of 930 mm (Figures 5 and 6 in Appendix F). The following factors of safety have been obtained:

FOS = 1.80 - Total stress analysis - Short term

FOS = 1.47 - Effective Stress Analysis - Long term.

The above Factors of Safety generally meet the minimum value of 1.5 normally accepted for this type of analysis.

10.2 Approach Embankment Settlements

Placement of additional embankment fill to establish the proposed grade raise of up to 930 mm is expected to result in consolidation settlement of the thick silty clay deposits underlying the site. Based on computations using Terzaghi one-dimensional consolidation theory, the primary consolidation settlements were estimated to be in the order of 30 mm for a grade raise of 930 mm.

Observation of the performance of the existing approach embankments indicates that the underlying compressible soils are also subject to ongoing secondary consolidation as well as downslope creep of the embankment slopes. Movement/deformation of the existing approach embankments were noted during the field work, as shown on Photographs 7 to 10 in Appendix C. The evidence of movement included pavement cracking, downslope leaning of the guide rail, and settlement in the vicinity of the wing walls. The asphalt thickness of 225 mm measured in Borehole PINE-05 may also be indicative of ongoing settlement and maintenance at the bridge.

Based on the site observations, ongoing differential settlement between the approach embankments and the bridge structure supported on an unyielding founding stratum should be anticipated. The settlement can be remedied by maintenance (asphalt padding) as settlement occurs. Creep movement of the embankment slopes and pavement edges should also be reduced by employing flatter side slopes of 2.5H:1V and preventing river erosion at the toe of slope.

If the anticipated long-term settlements and need for maintenance is not acceptable, the use of ultra-lightweight fill material (expanded polystyrene - EPS blocks) should be considered to reduce the loading on the compressible foundation soils and mitigate the settlements along the approach embankments.

Embankment construction should be in accordance with OPSS.PROV 206. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501. The backfill to the abutment walls should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150.

11 SCOUR AND EROSION PROTECTION

Erosion protection should be provided along any soil surfaces that may be in contact with the river flow to the height of the high water level. In particular, erosion protection must be provided in front of the sheet pile walls to prevent undermining/scouring of the walls at the abutments.

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

12 LATERAL EARTH PRESSURES

Earth pressures acting on the structure may be assumed to be distributed triangularly and to be governed by the characteristics of the abutment backfill. 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 = coefficient of lateral earth pressure (see Table 12.1)

γ = unit weight of retained soil (see Table 12.1)

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 given in Table 12.1.

Table 12.1 – Coefficients of Lateral Earth Pressure (K)

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35°$, $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I or Type III $\phi = 32°$, $\gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At-rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	-	3.3	-

* For wing walls.

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

The factors in Table 12.1 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 (CHBDC).

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 III, or at a depth of 1.7 m for Granular A or Granular B Type II.

13 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.00
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.00
- Peak Ground Acceleration 0.02 g

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 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.

For the design of retaining walls under seismic loading, the coefficients of horizontal earth pressure in Table 13.1 may be used:

Table 13.1 – Earth Pressure Coefficient for Earthquake Loading

Loading Condition	Earth Pressure Coefficient (K) for Earthquake Loading			
	Granular A or Granular B Type II $\phi = 35°; \gamma = 22.8$ kN/m^3		OPSS Granular B Type I or Type III $\phi = 32°; \gamma = 21.2$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (K_{AE})*	0.28	0.42	0.32	0.51
Passive (K_{PE})	3.6	-	3.2	-
At Rest (K_{OE})**	0.47	-	0.52	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods (1973).

Based on soil types and field test data, the foundation soils at the site are assessed as not being prone to liquefaction.

14 EXCAVATION AND DEWATERING

The proposed abutments will be installed behind and in close proximity to the existing bridge abutments. Excavations for removal of the existing abutment and wing walls are expected to extend to slightly above the river water level (Elev. 209 on July 6, 2012), and approximately 5 to 6 m below the existing embankment grade/top of bridge deck. This excavations are expected to extend into the existing embankment and abutment fills, and may extend nominally into the native cohesive deposits.

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill above the water table may be classified as Type 3 soil. Flatter slopes may be required at locations where water seepage affects surficial stability.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902. The selection of the method of excavation and equipment is the responsibility of the Contractor. Provision must be made for handling of pavement materials, potential obstructions in the fill, and cobbles/boulders. Special equipment may be required for excavation of the existing abutments to the required levels.

Roadway protection will be required to facilitate staged construction at this site. The temporary excavation support system should be designed and constructed in accordance with OPSS 539, Level 2 requirements. Sheet piles or soldier pile and lagging walls are considered appropriate for roadway protection. The Contractor should select the wall type and design taking into account the soil conditions encountered in the boreholes.

15 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not limited to:

- Installation of H-piles for the replacement bridge may potentially cause settlement of the existing bridge during staged construction. Monitoring of the settlement of the existing bridge will be required for the duration of pile driving. The Contactor should be prepared with appropriate equipment on site to maintain the grade of the existing bridge within acceptable tolerance.
- The existing piles and pile caps should be left in place to minimize disturbance to the soft to stiff silty clay underlying the site.
- The sequence of H-pile and sheet pile installation should be carefully considered to avoid pile alignment problems.
- Installation of the sheet piles retaining approach embankments may encounter resistance in the fill due to the presence of cobble/boulders. The Contractor must allow for removal of any such obstructions.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the clay subgrade to support the proposed construction equipment and any temporary structures or fill (i.e. as a pad for crane support). Site conditions may limit the type of equipment suitable for use. This is of particular importance due to the presence of the silty clay underlying directly the embankment fill. The design and safety of any temporary works is the responsibility of the Contractor. Recommended wording for an NSSP addressing this issue is provided in Appendix E.

16 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Ms. Anna Piascik, P.Eng. The report was reviewed by Mr. Murray Anderson, P.Eng. and Dr. P. K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction
1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

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

▼ Water Level

C_{pen} 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

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

DISCONTINUITY SPACING

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

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	(psi)	Field Estimation of Hardness*
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
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 % 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.

Ministry of Transportation
Ontario

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RECORD OF BOREHOLE No PINE-01

METRIC

WP# 6098-10-01 LOCATION Pine River Bridge N 5 325 499.9 E 339 427.6 ORIGINATED BY MNW

HWY 61 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN

DATUM Geodetic DATE 2014.11.02 - 2014.11.02 CHECKED BY MEF

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE)	WATER CONTENT (%)	GR	SA	SI	CL
215.1	GROUND SURFACE										
0.0	**ASPHALT:** (50mm)				215						
	Gravelly **SAND**, some silt Compact Dark Brown Moist (FILL)	1	SS	30							
		2	SS	27	214						
		3	SS	18				30	52	18 (SI+CL)	
212.8 2.3	Clayey **SILT**, trace gravel, occasional sand lenses Firm Reddish Brown Moist (FILL)	4	SS	8	213						
		5	SS	4	212						
211.5 3.6	Silty **CLAY**, with silt lenses Firm to Stiff Reddish Brown Moist					6.2					
		6	SS	4	211			0	0	65	35
					210	4.2					
		7	SS	3	209						
					208	8.9					
	Trace silt seams	8	SS	3	207			0	0	57	43
						5.2					
	Brown	9	SS	3	206						
205.3 9.8	END OF BOREHOLE AT 9.8m.					5.6					

Continued Next Page

+ 3, × 3 : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No PINE-01

METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 499.9 E 339 427.6	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.11.02 - 2014.11.02	CHECKED BY	MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
	BOREHOLE OPEN TO 6.1m AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.3m, DRY CONCRETE MIX TO 0.05m, THEN ASPHALT TO SURFACE.										

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S 5121.GPJ 2012TEMPLATE(MTO).GDT 12/23/14

RECORD OF BOREHOLE No PINE-02

METRIC

WP# 6098-10-01
LOCATION Pine River Bridge N 5 325 318.3 E 339 426.0
ORIGINATED BY MNW

HWY 61
BOREHOLE TYPE Hollow Stem Augers/Casing
COMPILED BY AN

DATUM Geodetic
DATE 2014.10.29 - 2014.10.30
CHECKED BY MEF

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
215.0	GROUND SURFACE
0.0	**ASPHALT:** (125mm)
0.1	**SAND** and **GRAVEL**, trace silt, occasional cobbles Compact to Dense Brown Moist to Dry (FILL)
211.0 4.0	Silty **CLAY** Firm to Stiff Reddish Brown Moist
	Wet
	Trace gravel from 8.8m to 10.1m

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	37
2	SS	29
3	SS	16
4	SS	10
5	SS	14
6	SS	5
7	SS	3
8	SS	12
9	SS	18

GROUND WATER CONDITIONS

ELEVATION SCALE: 214, 213, 212, 211, 210, 209, 208, 207, 206, 205

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values: 4.1 (approx. elev. 207.8); 6.9 (approx. elev. 206.3)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Sample	GR	SA	SI	CL
1	36	54	10 (SI+CL)	

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No PINE-02 METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 318.3 E 339 426.0	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Hollow Stem Augers/Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.10.29 - 2014.10.30	CHECKED BY	MEF

Continued From Previous Page

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
	Soft to Firm Trace silt seams
202.4 12.6	Clayey **SILT**, trace stratifications Very Soft to Stiff Brown Moist to Wet
199.4 15.6	Silty **CLAY**, trace sand Stiff Brown Moist Occasional lenses of sand Firm Dark Greyish Brown

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
10	SS	4	0 0 65 35
11	SS	11	
12	SS	2	
13	SS	9	
14	SS	7	0 5 22 73
15	SS	5	

Elevation scale: 204, 203, 202, 201, 200, 199, 198, 197, 196, 195

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
Field vane sensitivity values: 2.9, 3.3, 4.3, 2.3

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

UNIT WEIGHT γ kN/m³

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ◐ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No PINE-02

METRIC

WP# 6098-10-01 | LOCATION Pine River Bridge N 5 325 318.3 E 339 426.0 | ORIGINATED BY MNW

HWY 61 | BOREHOLE TYPE Hollow Stem Augers/Casing | COMPILED BY AN

DATUM Geodetic | DATE 2014.10.29 - 2014.10.30 | CHECKED BY MEF

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (kPa)	WATER CONTENT (%)	GR	SA	SI	CL
	Occasional sand seams Dark Grey and Brown Stratified	16	SS	5							
					194						
					193						
					192						
		17	SS	8				0	0	26	74
					191	Field vane, sensitivity 2.7					
	Occasional silt seams Wet				190						
					189						
		18	SS	5							
					188	Field vane, sensitivity 1.7					
187.3 27.7	**SILT**, some clay to clayey, trace sand Loose Dark Greyish Brown Wet				187						
					186						
		19	SS	9							
					185						

Continued Next Page

+ 3, × 3 : Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PINE-02

METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 318.3 E 339 426.0	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Hollow Stem Augers/Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.10.29 - 2014.10.30	CHECKED BY	MEF

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page							+ (field vane, >100)			
	Clayey with stratifications, occasional sand seams Wet						184				
							183				
			20	SS	7				○ (~58)		
	Trace of gravel						182				
							181				
180.7 / 34.3	Clayey **SILT**, trace sand, trace gravel, occasional cobbles and boulders Very Stiff Dark Greyish Brown Moist (TILL)						180				
							179				
							178				
			21	SS	24				○ (~21)		
	Frequent cobbles and boulders, borehole cored below 37.5m depth						177				
							176				
175.2 / 39.8	**LIMESTONE**						175				

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DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%): 20 40 60

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No PINE-02

METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 318.3 E 339 426.0	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Hollow Stem Augers/Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.10.29 - 2014.10.30	CHECKED BY	MEF

Column headings: SOIL PROFILE (ELEV/DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS
	LIMESTONE, trace calcite, moderately weathered to fresh, grey: (BEDROCK)	1	RUN			RUN #1 TCR=100% SCR=100% RQD=100% UCS=70.7MPa (Average)
	Clay infilling at 40.6m	2	RUN		174	RUN #2 TCR=88% SCR=88% RQD=77% UCS=94.4MPa (Average)
	Highly fractured below 42.2m	3	RUN		173, 172	RUN #3 TCR=50% SCR=13% RQD=0%
	Possible seams of sand and clay below 43m depth	4	RUN		171	RUN #4 TCR=25% SCR=0% RQD=0%
170.0 45.0	END OF BOREHOLE AT 45.0m. WATER LEVEL AT 2.1m ABOVE ROAD SURFACE ON COMPLETION OF DRILLING AND REMOVAL OF NQ CASING. BOREHOLE BACKFILLED WITH HOLEPLUG TO 36.6m, HOLEPLUG AND CUTTINGS TO 0.3m, CONCRETE TO 0.1m, THEN ASPHALT PATCH TO SURFACE.				170	

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+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

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Ontario

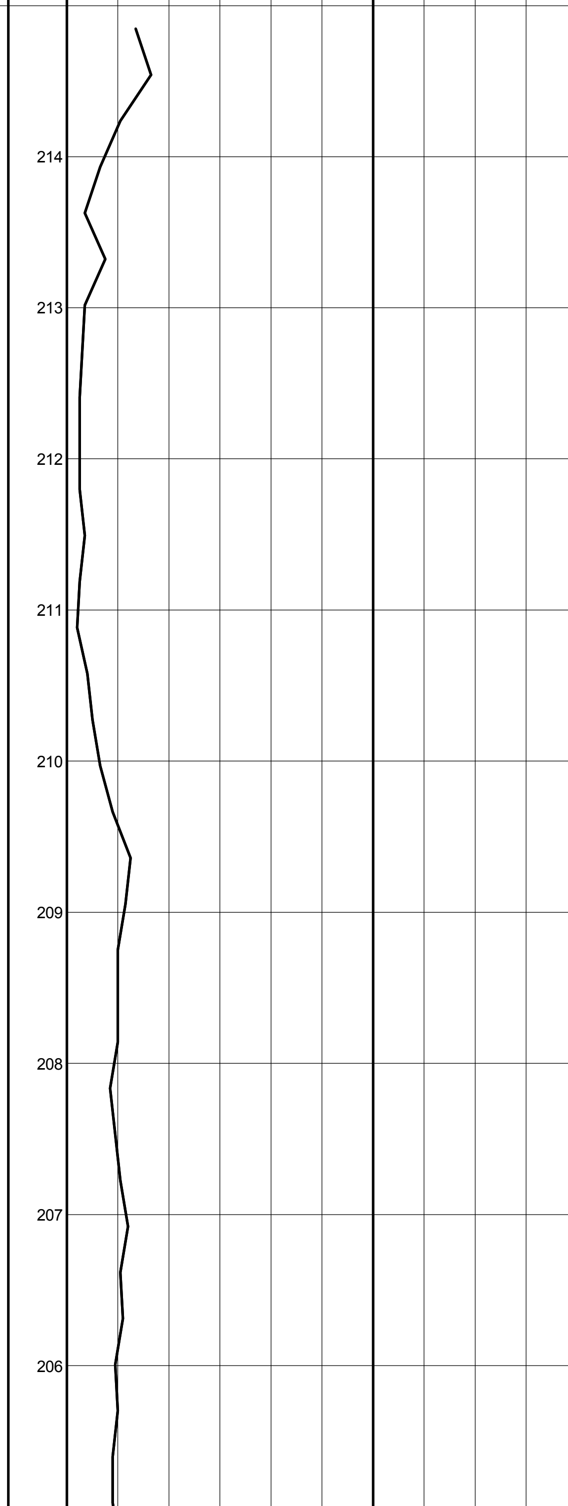
THURBER

RECORD OF BOREHOLE No PINE-03

METRIC

WP# 6098-10-01	LOCATION Pine River Bridge N 5 325 516.0 E 339 433.0		ORIGINATED BY MNW
HWY 61	BOREHOLE TYPE Dynamic Cone Penetration Test		COMPILED BY AN
DATUM Geodetic	DATE 2014.11.02 - 2014.11.02		CHECKED BY MEF

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.0	GROUND SURFACE										
0.0	**ASPHALT:** (125mm)										
0.1	Start DCPT at 0.1m.										



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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S 5121.GPJ 2012TEMPLATE(MTO).GDT 12/23/14

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No PINE-03

METRIC

WP# 6098-10-01	LOCATION Pine River Bridge N 5 325 516.0 E 339 433.0		ORIGINATED BY MNW
HWY 61	BOREHOLE TYPE Dynamic Cone Penetration Test		COMPILED BY AN
DATUM Geodetic	DATE 2014.11.02 - 2014.11.02		CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
							204				
							203				
							202				
							201				
							200				
							199				
							198				
							197				
							196				
							195				

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PINE-03

METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 516.0 E 339 433.0	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Dynamic Cone Penetration Test	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.11.02 - 2014.11.02	CHECKED BY	MEF

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL
	Continued From Previous Page										
							194				
							193				
							192				
190.9 / 24.1	END OF DCPT AT 24.1m.						191				

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
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RECORD OF BOREHOLE No PINE-04

METRIC

WP# 6098-10-01	LOCATION Pine River Bridge N 5 325 547.1 E 339 435.7	ORIGINATED BY MNW
HWY 61	BOREHOLE TYPE Dynamic Cone Penetration Test	COMPILED BY AN
DATUM Geodetic	DATE 2014.11.02 - 2014.11.02	CHECKED BY MEF

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
215.0	GROUND SURFACE										
0.0	Start DCPT from surface										
							214				
							213				
							212				
							211				
							210				
							209				
							208				
							207				
							206				
							205				

Continued Next Page

+ 3 , × 3 : Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No PINE-04

METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 547.1 E 339 435.7	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Dynamic Cone Penetration Test	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.11.02 - 2014.11.02	CHECKED BY	MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
							204				
							203				
							202				
							201				
							200				
							199				
							198				
							197				
							196				
							195				

Continued Next Page

+^{3}, ×^{3}: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

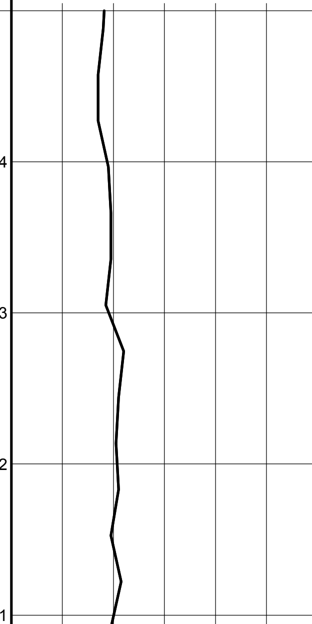
THURBER

RECORD OF BOREHOLE No PINE-04

METRIC

WP# 6098-10-01	LOCATION Pine River Bridge N 5 325 547.1 E 339 435.7	ORIGINATED BY MNW
HWY 61	BOREHOLE TYPE Dynamic Cone Penetration Test	COMPILED BY AN
DATUM Geodetic	DATE 2014.11.02 - 2014.11.02	CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P – NATURAL MOISTURE CONTENT w – LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page						194				
							193				
							192				
190.9 / 24.1	END OF DCPT AT 24.1m.						191				



ONTMT4S 5121.GPJ 2012TEMPLATE(MTO).GDT 12/23/14

+ 3 , × 3 : Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PINE-05

METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 544.8 E 339 442.7	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Hollow Stem Augers/Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.10.31 - 2014.10.31	CHECKED BY	MEF

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
215.0	GROUND SURFACE
0.0	**ASPHALT:** (225mm)
0.2	**SAND** and **GRAVEL**, trace silt Loose to Very Dense Brown Dry (FILL)
210.7 / 4.3	Silty **CLAY**, trace sand Firm to Stiff Reddish Brown Moist Trace rootlets and wood fragments to 5.3m depth Trace gravel between 7.6m to 10.0m Occasional stratifications

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	52				
2	SS	29	42	50	8 (SI+CL)	
3	SS	19				
4	SS	11				
5	SS	9				
6	SS	6				
7	SS	3				
8	SS	4	0	6	58	36
9	SS	6				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (214, 213, 212, 211, 210, 209, 208, 207, 206, 205); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m^3.

Field vane sensitivity values: 5.1 (at approx. elev. 208), 3.8 (at approx. elev. 206.5).

Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

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RECORD OF BOREHOLE No PINE-05

METRIC

WP# 6098-10-01 | LOCATION Pine River Bridge N 5 325 544.8 E 339 442.7 | ORIGINATED BY MNW
HWY 61 | BOREHOLE TYPE Hollow Stem Augers/Casing | COMPILED BY AN
DATUM Geodetic | DATE 2014.10.31 - 2014.10.31 | CHECKED BY MEF

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE)	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
						6.3						
	Wet	10	SS	3	204		o					
						7.3						
	Occasional silt lenses	11	SS	2	203		o					
					202	9.7						
	With silt layers	12	SS	21	201		o					
200.2 / 14.8	Silty **CLAY** Stiff Brown Moist				200							
		13	SS	13	199		o					
		14	SS	8	198		o		0	0	31	69
	Lenses of sand				197							
		15	SS	7	196		o					
					195	5.8						

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No PINE-05

METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 544.8 E 339 442.7	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Hollow Stem Augers/Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.10.31 - 2014.10.31	CHECKED BY	MEF

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE)	REMARKS GR	SA	SI	CL
		16	SS	8						
					194	4.2				
					193					
		17	SS	8	192					
					191					
					190					
	With saturated silt layers, dilatent	18	SS	1	189		0	0	24	76
188.2 26.8	**SILT**, some clay to clayey, trace sand, trace gravel, occasional seams of silty clay Compact Dark Greyish Brown Moist				188					
					187					
	Trace sand seams	19	SS	19	186					
					185					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

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Ontario

THURBER

RECORD OF BOREHOLE No PINE-05

METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 544.8 E 339 442.7	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Hollow Stem Augers/Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.10.31 - 2014.10.31	CHECKED BY	MEF

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Continued From Previous Page									
184.4 / 30.6	Sandy **GRAVEL**, trace silt, trace cobbles Very Dense Dark Grey Moist				184					
		20	SS	109	183	○ (w)	65	27	8 (SI+CL)	
182.7 / 32.3	END OF BOREHOLE AT 32.3m DUE TO ARTESIAN CONDITIONS. WATER LEVEL MEASURED IN CASING AT 1.7m ABOVE ROAD SURFACE. BOREHOLE BACKFILLED WITH HOLEPLUG TO 4.6m, CUTTINGS TO 0.3m, CONCRETE TO 0.1m, THEN ASPHALT COLD PATCH TO SURFACE. MOVED 1.5m NORTH AND AUGERED TO 12.2m FOR INSTALLATION OF PIEZOMETER. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 13.0m slotted screen.									

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Nov 01/14	2.0	213.0
Nov 02/14	2.3	212.7
Nov 03/14	2.3	212.7

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No PINE-06

METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 563.2 E 339 441.2	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.11.03 - 2014.11.03	CHECKED BY	MEF

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
215.1	GROUND SURFACE							
0.0	**ASPHALT:** (125mm)							
0.1	**SAND** and **GRAVEL**, trace silt Compact Dark Brown Moist (FILL)	1	SS	23				
		2	SS	29				
	Burnt wood (150mm) at 2.0m	3	SS	21	41	52	7 (SI+CL)	
213.0 / 2.1	Silty **CLAY**, trace sand Firm to Stiff Reddish Brown Moist to Wet	4	SS	7				
	With lenses of silt	5	SS	5				
		6	SS	3				
		7	SS	4				
		8	SS	3				
	Brown	9	SS	0	0	0	51	49

Elevation scale: 215, 214, 213, 212, 211, 210, 209, 208, 207, 206

Field vane sensitivity values: 5.3 (at approx. El. 211), 4.4 (approx. El. 209.5), 6.8 (approx. El. 207.9), 5.6 (approx. El. 206.4)

Continued Next Page

+ ³, × ³ : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No PINE-06

METRIC

WP#	6098-10-01	LOCATION	Pine River Bridge N 5 325 563.2 E 339 441.2	ORIGINATED BY	MNW
HWY	61	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2014.11.03 - 2014.11.03	CHECKED BY	MEF

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
204.9							205	5.6			
10.2	END OF BOREHOLE AT 10.2m. BOREHOLE OPEN TO 3.7m AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.3m, DRY CEMENT TO 0.1m, THEN ASPHALT COLD PATCH TO SURFACE.										

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

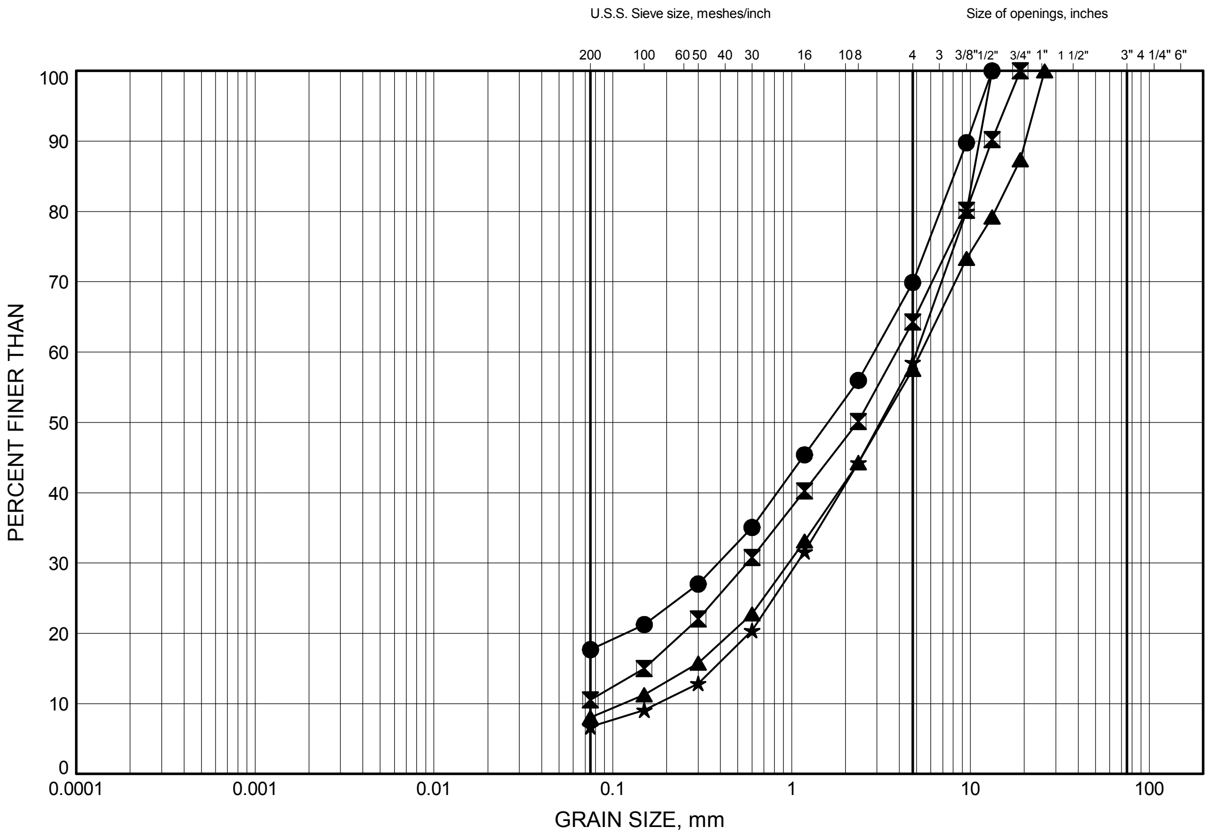
20 / 15 5 / 10 (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

Pine River Bridge GRAIN SIZE DISTRIBUTION	FIGURE B1

SAND & GRAVEL TO GRAVELLY SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-01	1.83	213.27
⧗	PINE-02	0.38	214.62
▲	PINE-05	1.07	213.93
★	PINE-06	1.75	213.34

Date December 2014

WP# 6098-10-01

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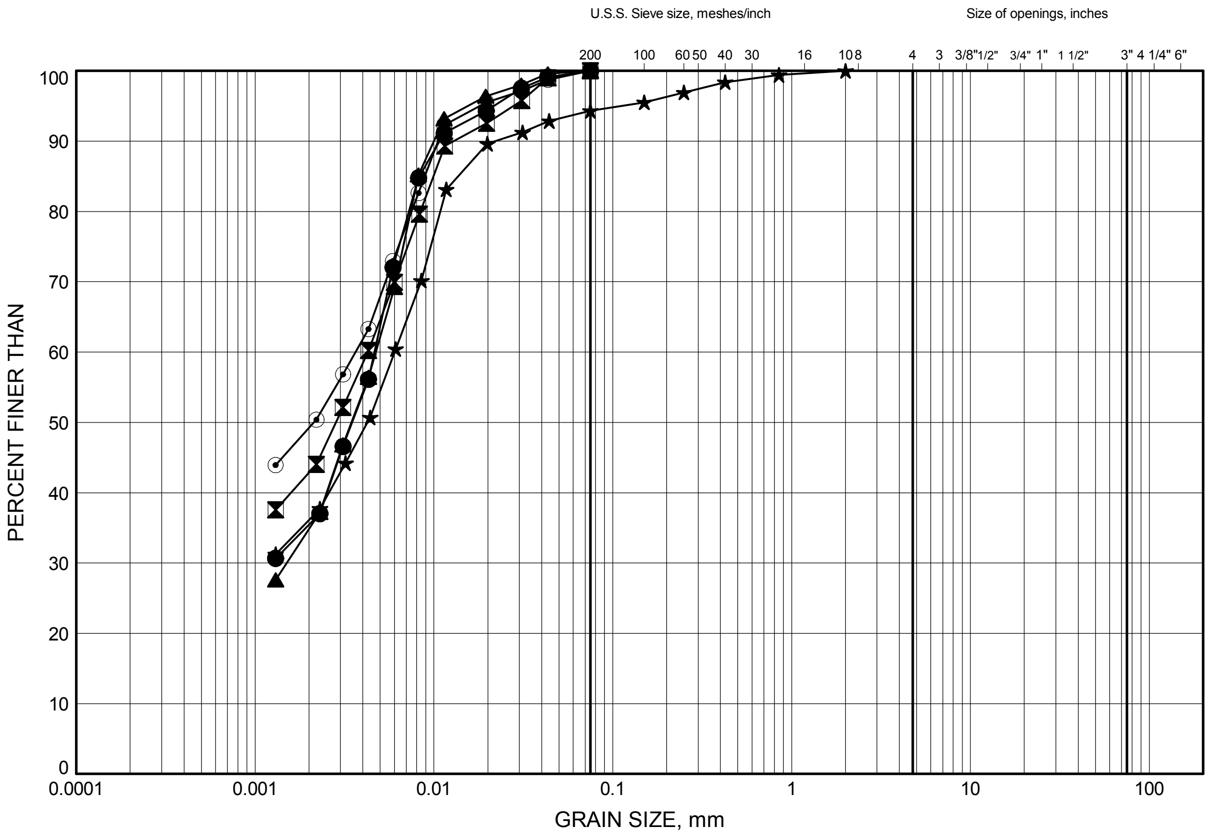
GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 12/5/14

Pine River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B2

Upper SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-01	4.88	210.22
⧗	PINE-01	7.92	207.17
▲	PINE-02	10.97	204.02
★	PINE-05	7.92	207.07
⊙	PINE-06	9.45	205.65

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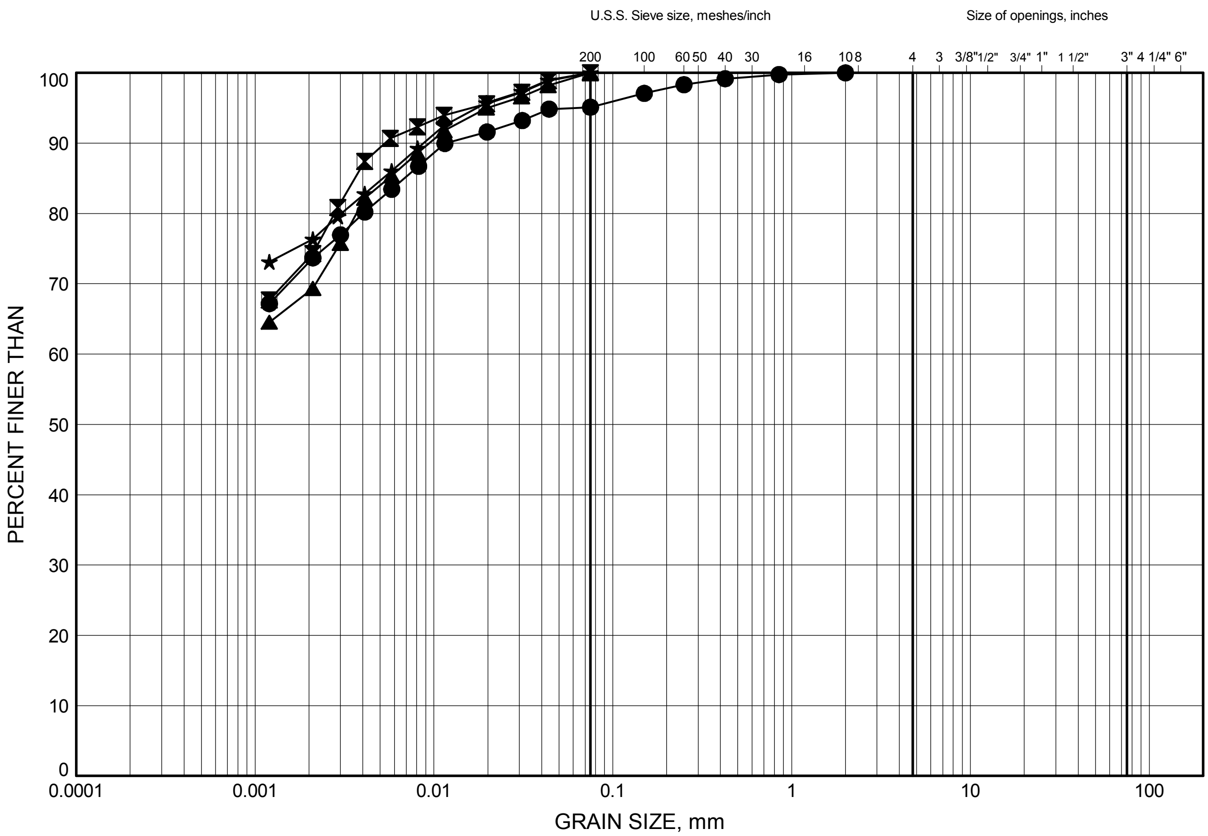
GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 12/5/14

Pine River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B3

Lower SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-02	17.07	197.93
⧗	PINE-02	23.16	191.83
▲	PINE-05	17.07	197.93
★	PINE-05	26.21	188.78

Date December 2014

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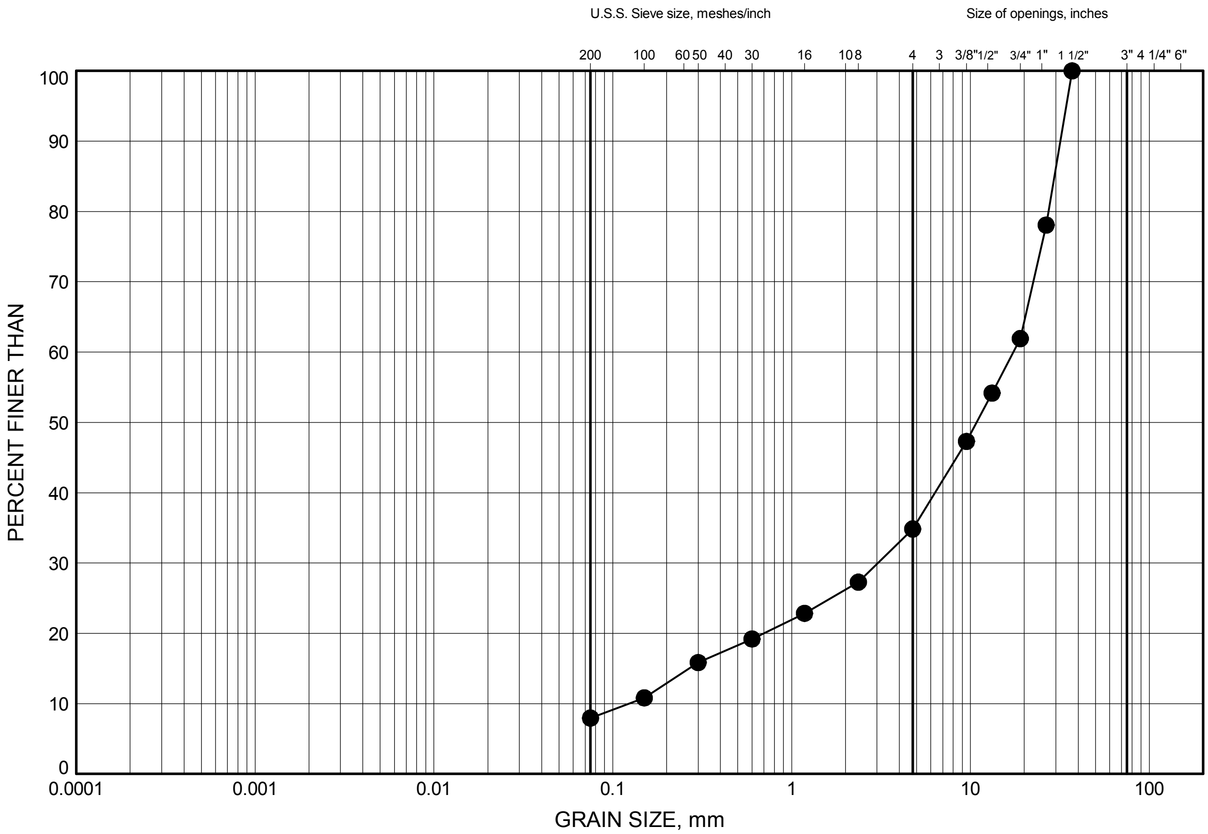
Pine River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B4

SANDY GRAVEL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-05	32.16	182.84

GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 12/5/14

Date December 2014

WP# 6098-10-01

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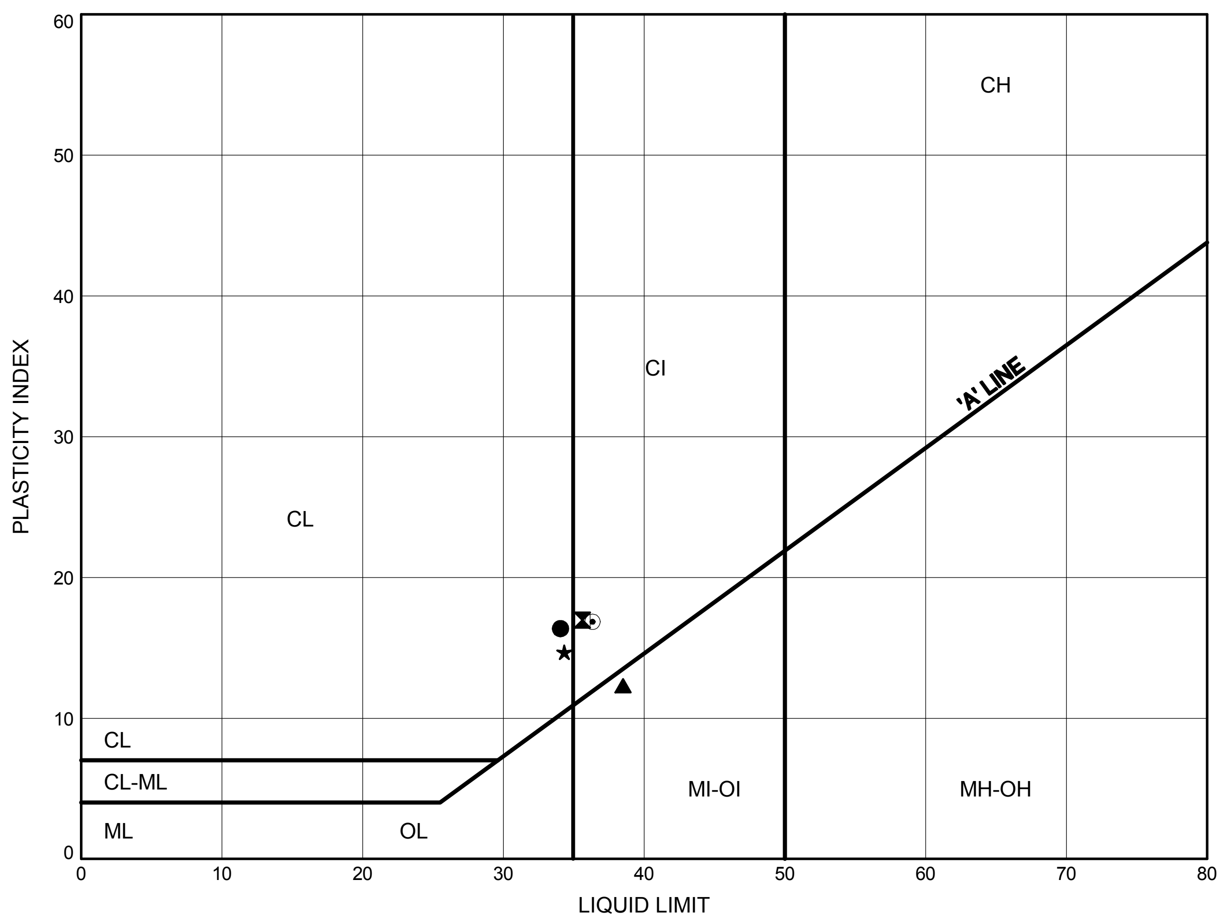
Pine River Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

Upper SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-01	4.88	210.22
⧗	PINE-01	7.92	207.17
▲	PINE-02	10.97	204.02
★	PINE-05	7.92	207.07
⊙	PINE-06	9.45	205.65

Date December 2014

WP# 6098-10-01

THURBER

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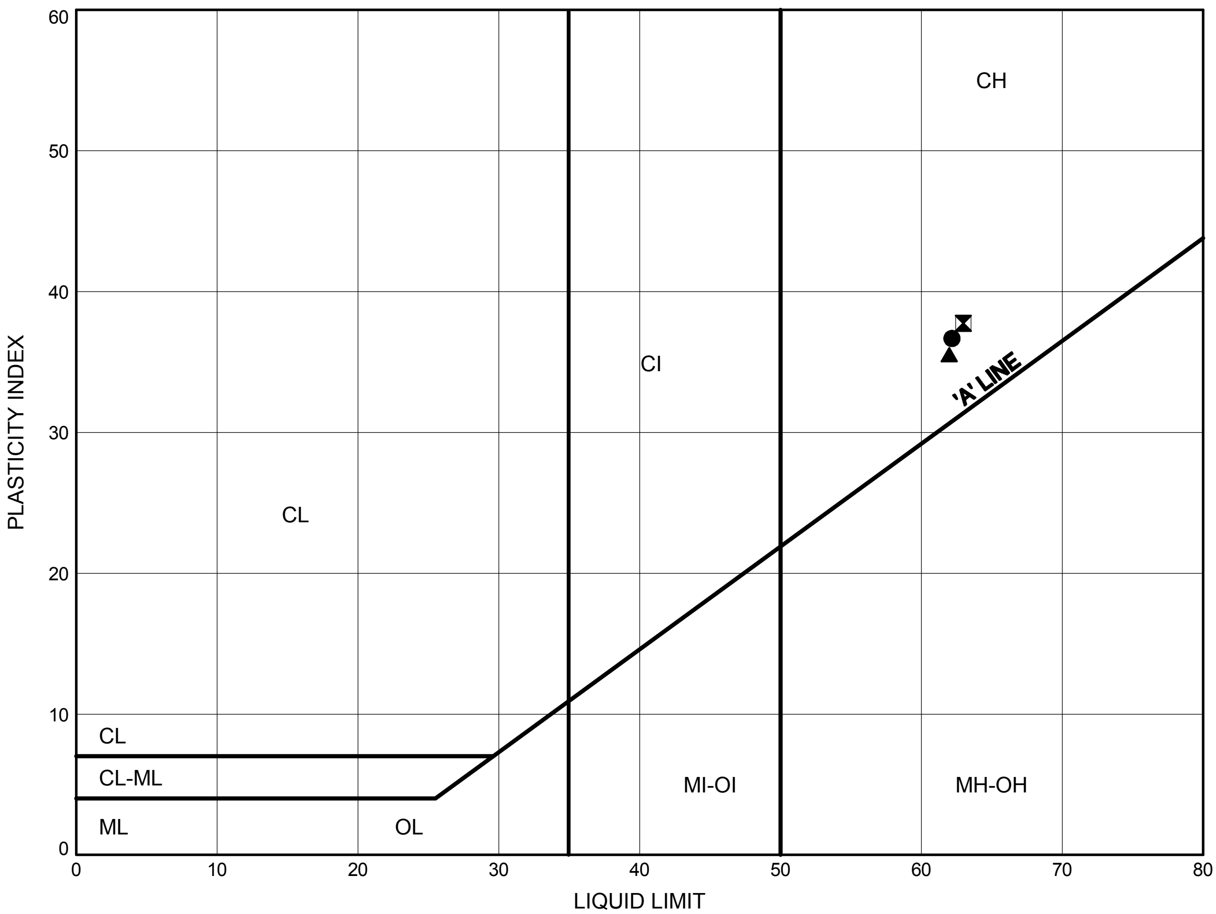
THURBALT 5121.GPJ 12/10/14

Pine River Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

Lower SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-02	17.07	197.93
⧗	PINE-02	23.16	191.83
▲	PINE-05	26.21	188.78

THURBALT 5121.GPJ 12/10/14

Date December 2014

WP# 6098-10-01

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THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No : 19-1605-121 **Client :** HMM

Date Drilled : 10/30/2014

Project Name : Pine River Bridge **Date Tested :** 11/17/2014

Core Size : NQ **BH No :** Pine-02 **Tester :** ISP

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (kPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	40.1	D	26400.0	47.1	200.0	74.8	Limestone	Strong
2	1	40.8	D	24960.0	47.2	200.0	66.6	Limestone	Strong
3	2	41.2	ial or Diame	31160.0	47.1	72.6	118.9	Limestone	Very Strong
4	2	41.8	D	31800.0	47.1	200.0	69.9	Limestone	Strong
5									
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 x D on either side of test point.

Appendix C

Site Photographs

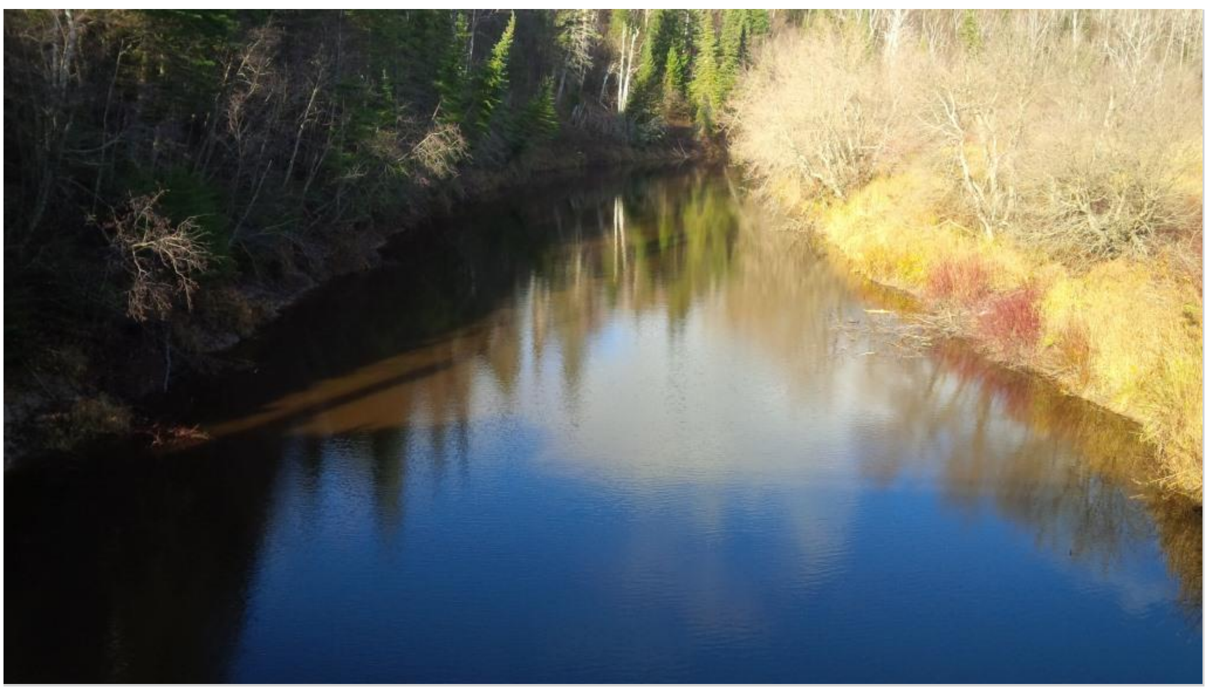
Photograph 1 - Pine River Bridge Looking North

Photograph 2 – Looking South

Photograph 3 - East Bridge Elevation - Looking at South Abutment

Photograph 4 - East Bridge Elevation - Looking at North Abutment

Photograph 5 - Looking at South Abutment

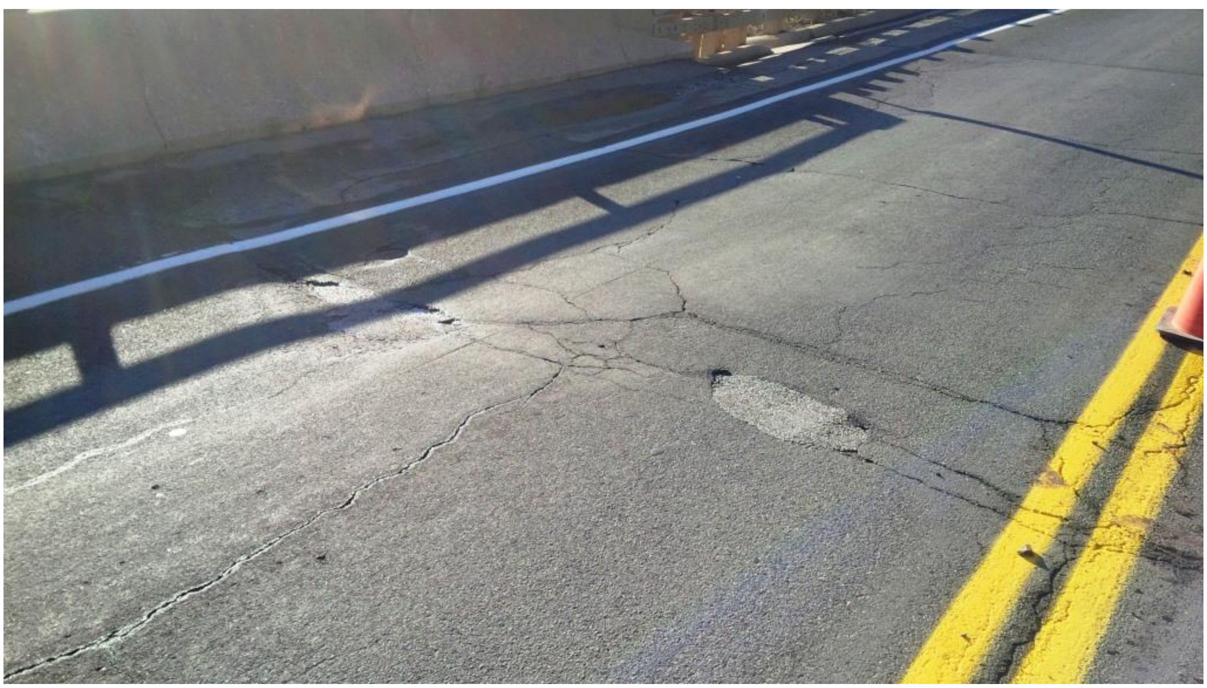


Photograph 6 - Pine River – Looking West

Photograph 7 - Looking South at South Approach

Photograph 8 – Settlements at Southeast Wingwall

Photograph 9 – Settlements at North Approach – Leaning Guide Rail



Photograph 10 – Settlements/Fractures at the location of South Abutment Wall

Appendix D

Comparison of Foundation Alternatives

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Engineered Fill	Driven Piles	Caissons
Advantages i. Ease of construction. ii. Lower cost than deep foundations.	***Advantages:*** i. Generally less costly construction than deep foundation elements. ii. Allows use of perched abutments. iii. Higher geotechnical resistance than for spread footings placed directly on native soil.	***Advantages:*** i. Piles will develop high geotechnical resistance on very dense soils or bedrock. ii. Installation of piles could continue in freezing weather. iii. Allows integral abutment design. iv. Requires less excavation than spread footings.	***Advantages:*** i. Higher resistances may be available for caissons than for spread footings founded in native soils. ii. Construction of caissons could continue in freezing weather.
Disadvantages: i. Low geotechnical resistance available in native soils at abutments. ii. Potential for significant consolidation settlement in silty clay. iii. Dewatering may be required, depending on depth of excavation. **NOT RECOMMENDED**	***Disadvantages:*** i. Cost of engineered fill placement. ii. Potential for consolidation settlement in silty clay. iii. Dewatering may be required, depending on depth of excavation. **NOT RECOMMENDED**	***Disadvantages:*** i. Higher unit costs than for spread footings. ii. Possibility that cobbles and/or boulders may be encountered in the fill and native deposits. iii. Piles may encounter refusal at varying depths. **RECOMMENDED**	***Disadvantages:*** i. Relatively low capacities in native soils compared to H-pile foundations. ii. High cost of construction, as caissons would need to be relatively deep. iii. Possibility of encountering cobbles and boulders during augering and liner installation. iv. Difficulty in cleaning and inspecting bases. **NOT RECOMMENDED**

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Appendix E

List of Standard Specifications and Special Provisions

1) The following Standard Specifications and Special Provisions are referenced in this report:

OPSS 501
OPSS 539
OPSS 804
OPSS 902
OPSS 903

OPSS.PROV 1010
OPSS.PROV 206

OPSD 3101.150
SS103-11 (Hiley Formula)

2) Recommended wording for "NSSP – Use of Heavy Construction Equipment"

The use of heavy construction equipment and in particular heavy lift cranes may be required during removal of the existing and erection of the new bridge. The impact of the heavy equipment loads on the underlying sensitive soils, river banks and existing bridge foundations must be considered during selection of the methodology and equipment employed for construction.

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and methodology, and determine requirements and/or restrictions necessary to safely support the loads. All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO's RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – High Complexity.

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

- Determining appropriate setbacks for heavy equipment from the river banks and existing foundations;
- Evaluating the need for preventing heavy equipment from travelling or operating on the areas adjacent to the river, possibly requiring restriction of heavy loads to the existing highway embankment platform;
- Determining the permissible ground pressure that may be applied to the foundation soils by the equipment; and
- Providing recommendations for crane pad design to distribute the crane loads without causing foundation failure.

The Contractor shall submit the findings of the geotechnical assessment and details of the proposed equipment and construction methodology to the Contract Administrator for information purposes a minimum of two weeks prior to the start of construction.

3) Recommended wording for "NSSP – Monitoring of Existing Structure"

The Contractor shall ensure the existing structure remains stable during removal.

It is recommended that the Contract Documents include a monitoring program for the existing structure. As a minimum, the monitoring program should require the Contractor to establish reference points over each abutment of the existing structure and to monitor movement of these points relative to known, fixed reference points on a regular basis. The suggested frequency is:

- Three readings on separate days prior to construction to establish a baseline;
- Twice daily while any foundation construction or other subsurface construction is in progress;
- Daily for one week after completion of foundation construction.

The vertical and horizontal accuracy of readings should be ± 2 mm. All readings must be reported to the Contract Administrator within 24 hours and immediately if any movement exceeds limits set by the structural designer.

The Contract Administrator must be advised of the importance of monitoring and be required to advise the Ministry immediately if the vertical and horizontal movements exceed the specified limits.

Appendix F

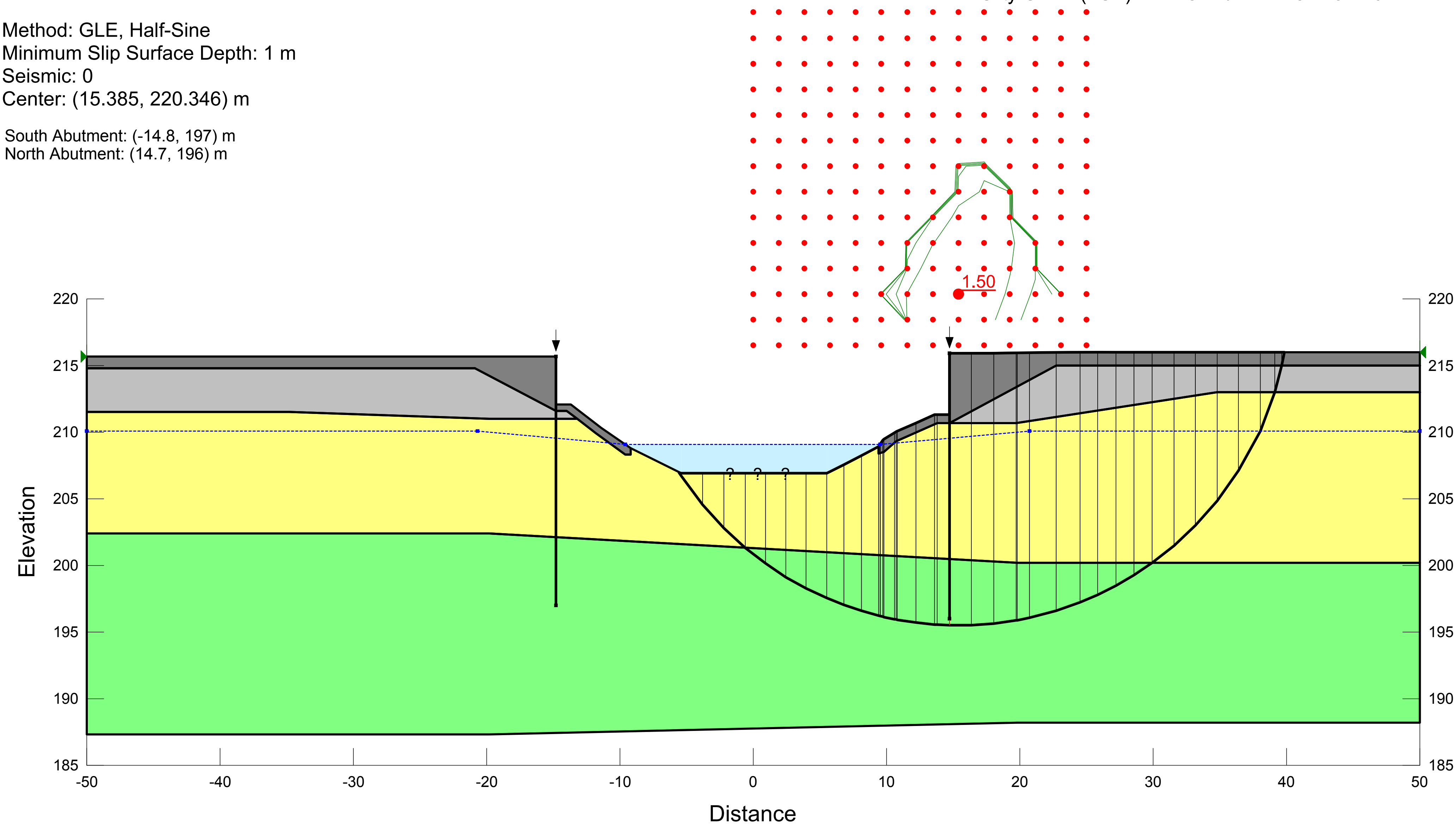
Slope Stability Analysis Results

Title: Highway 61, Pine River, Ontario
Comments: Stability Assessment
Name: North Abutment: TSA

New FILL	21 kN/m³	0 kPa	32 °	1
Existing FILL	20 kN/m³	0 kPa	30 °	1
Clayey SILT (TSA)	17 kN/m³	35 kPa	0 °	1
Silty CLAY (TSA)	18 kN/m³	45 kPa	0 °	1

Method: GLE, Half-Sine
Minimum Slip Surface Depth: 1 m
Seismic: 0
Center: (15.385, 220.346) m

South Abutment: (-14.8, 197) m
North Abutment: (14.7, 196) m



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Last Edited By: Stephen Peters
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Figure 1

Title: Highway 61, Pine River, Ontario
Comments: Stability Assessment
Name: North Abutment: ESA

New FILL	21 kN/m³	0 kPa	32 °	1
Existing FILL	20 kN/m³	0 kPa	30 °	1
Clayey SILT (ESA)	17 kN/m³	2 kPa	26 °	1
Silty CLAY (ESA)	18 kN/m³	2 kPa	24 °	1

Method: GLE, Half-Sine
Minimum Slip Surface Depth: 1 m
Seismic: 0
Center: (9.615, 218.423) m

South Abutment: (-14.8, 200) m
North Abutment: (14.7, 200) m

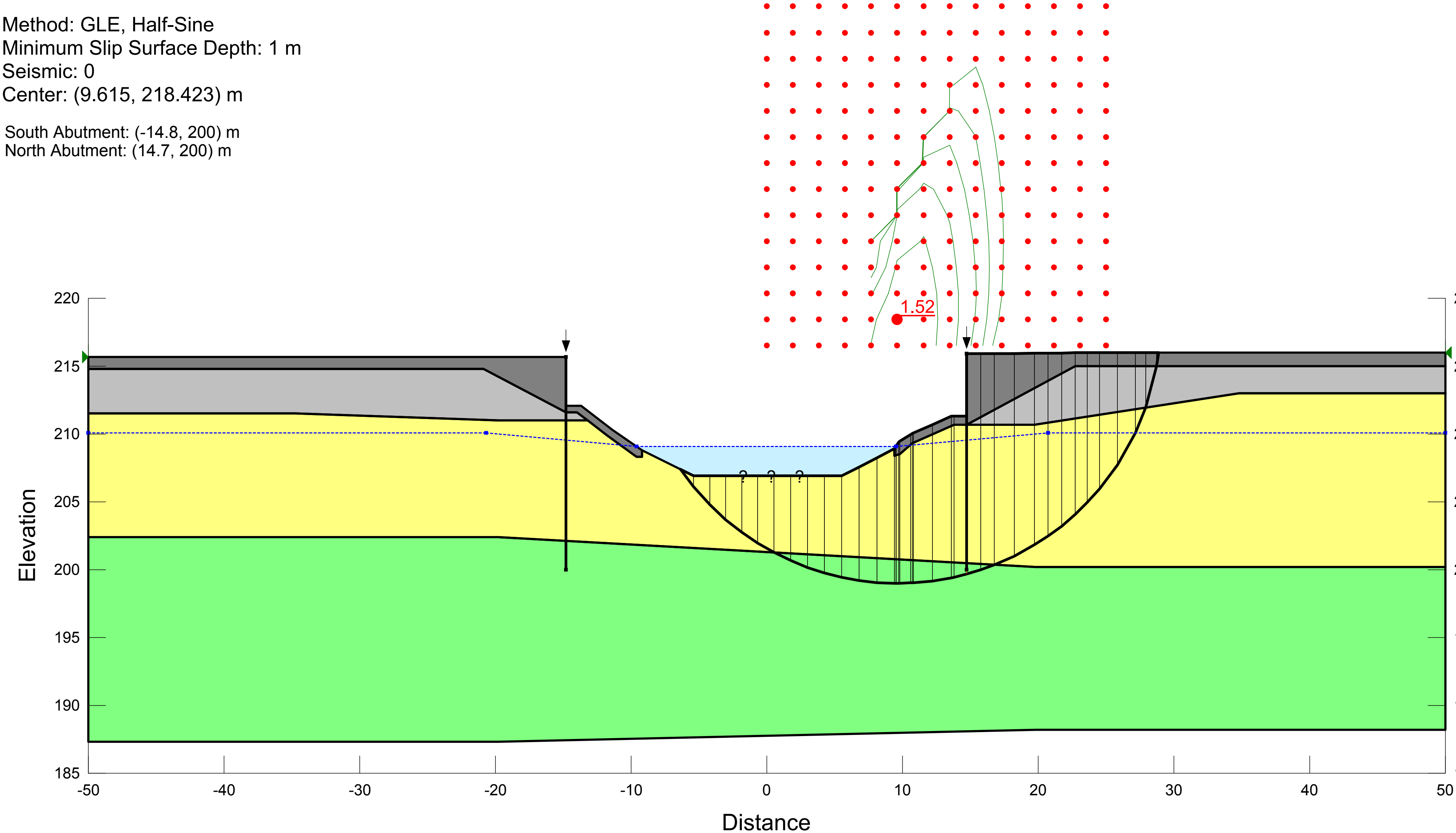

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Figure 2

Title: Highway 61, Pine River, Ontario
Comments: Stability Assessment
Name: South Abutment: TSA

New FILL	21 kN/m³	0 kPa	32 °	1
Existing FILL	20 kN/m³	0 kPa	30 °	1
Clayey SILT (TSA)	17 kN/m³	35 kPa	0 °	1
Silty CLAY (TSA)	18 kN/m³	45 kPa	0 °	1

Method: GLE, Half-Sine
Minimum Slip Surface Depth: 1 m
Seismic: 0
Center: (-13.462, 218.423) m

South Abutment: (-14.8, 197) m
North Abutment: (14.7, 196) m

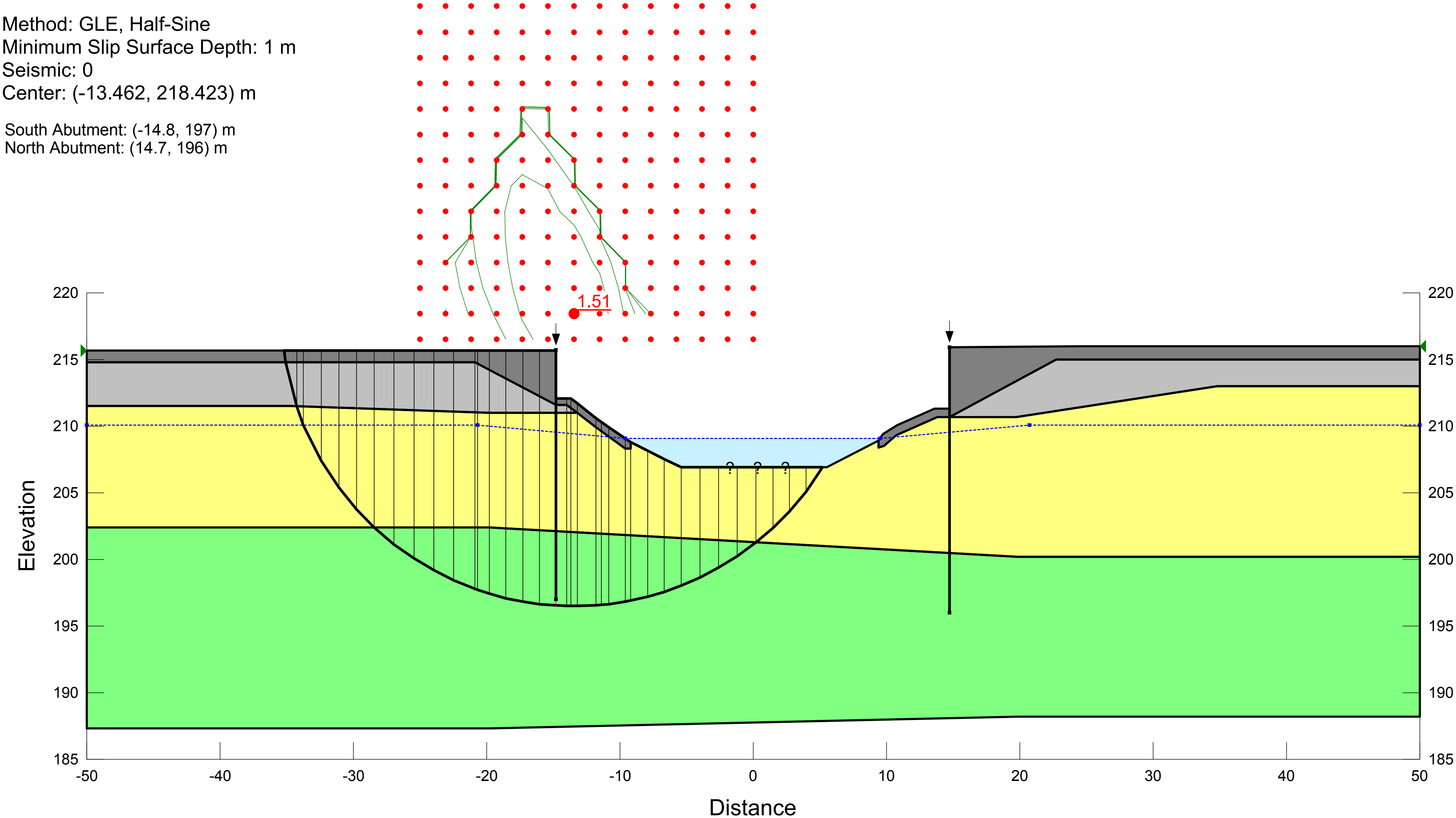

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Figure 3

Title: Highway 61, Pine River, Ontario
Comments: Stability Assessment
Name: South Abutment: ESA

New FILL	21 kN/m³	0 kPa	32 °	1
Existing FILL	20 kN/m³	0 kPa	30 °	1
Clayey SILT (ESA)	17 kN/m³	2 kPa	26 °	1
Silty CLAY (ESA)	18 kN/m³	2 kPa	24 °	1

Method: GLE, Half-Sine
Minimum Slip Surface Depth: 1 m
Seismic: 0
Center: (-9.615, 218.423) m

South Abutment: (-14.8, 200) m
North Abutment: (14.7, 200) m

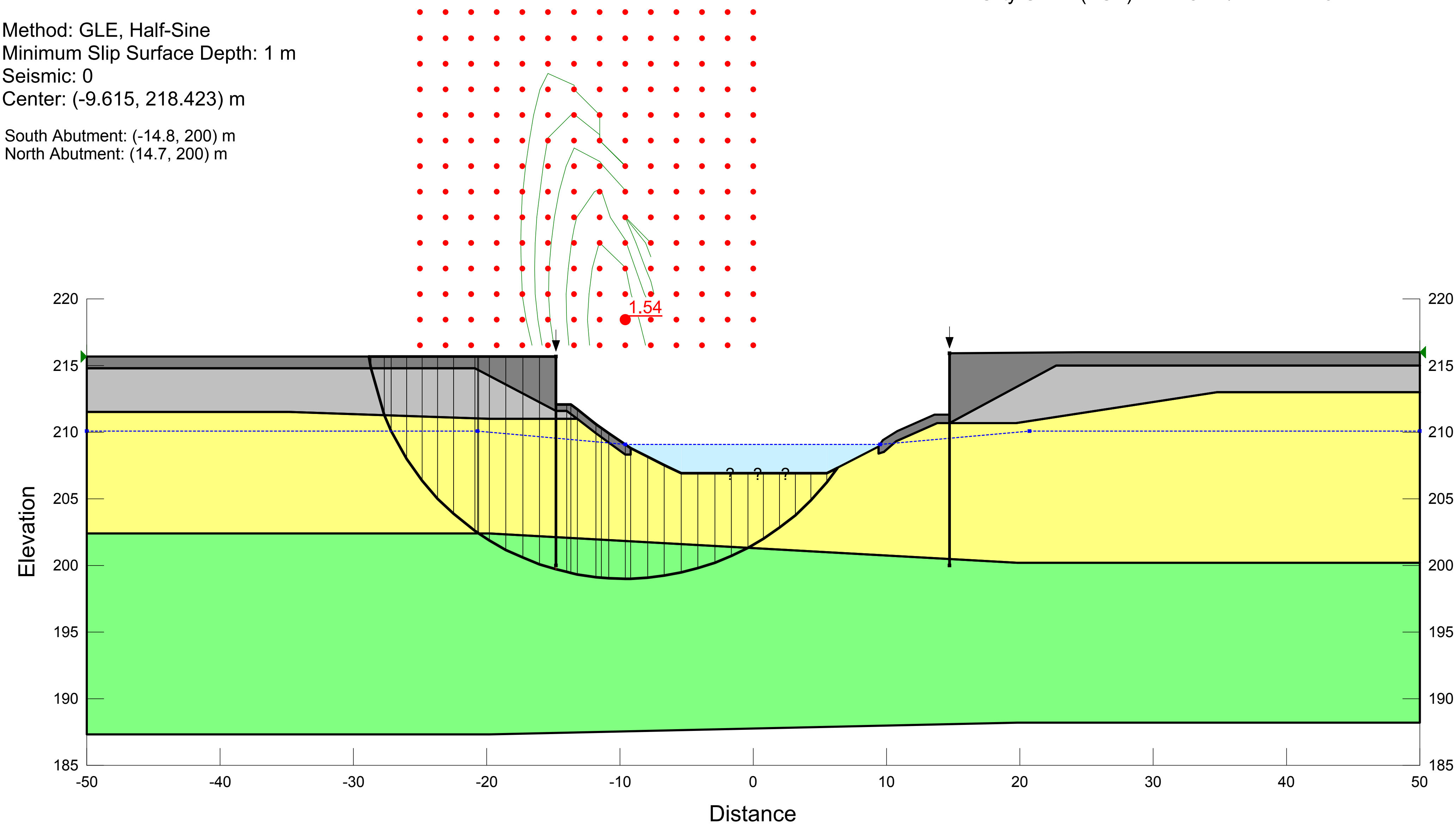

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Figure 4

Title: Highway 61, Pine River, Ontario
Comments: Stability Assessment
Name: North Abutment: TSA

Method: GLE, Half-Sine
Minimum Slip Surface Depth: 1 m
Seismic: 0
Center: (13, 221) m

New FILL	21 kN/m³	0 kPa	32 °	1
Existing FILL	20 kN/m³	0 kPa	30 °	1
Clayey SILT (TSA)	17 kN/m³	35 kPa	0 °	1
Silty CLAY (TSA)	18 kN/m³	45 kPa	0 °	1

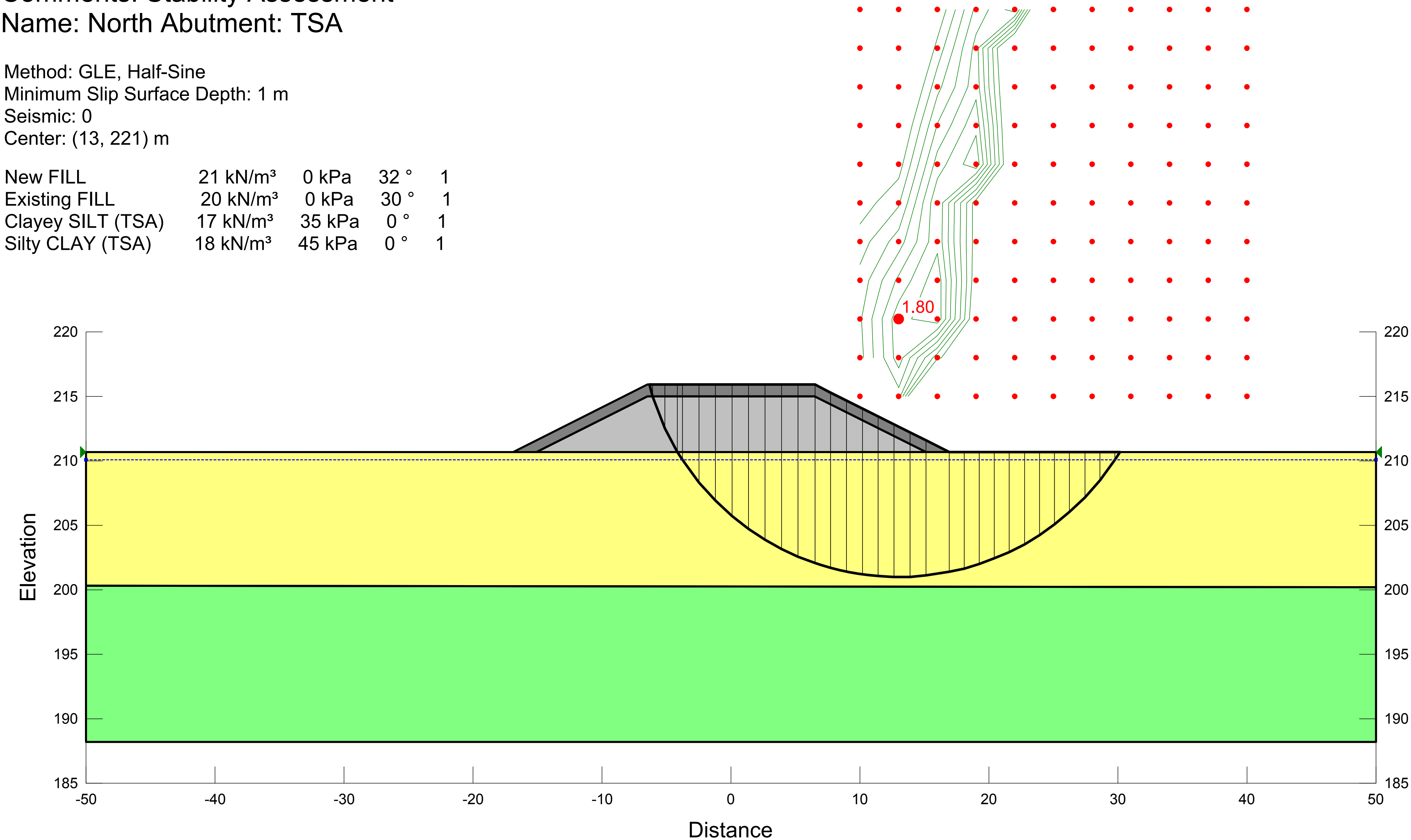

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Figure 5

Title: Highway 61, Pine River, Ontario
Comments: Stability Assessment
Name: North Abutment: ESA

Method: GLE, Half-Sine
Minimum Slip Surface Depth: 1 m
Seismic: 0
Center: (16, 221) m

New FILL	21 kN/m³	0 kPa	32 °	1
Existing FILL	20 kN/m³	0 kPa	30 °	1
Clayey SILT (ESA)	17 kN/m³	2 kPa	26 °	1
Silty CLAY (ESA)	18 kN/m³	2 kPa	24 °	1

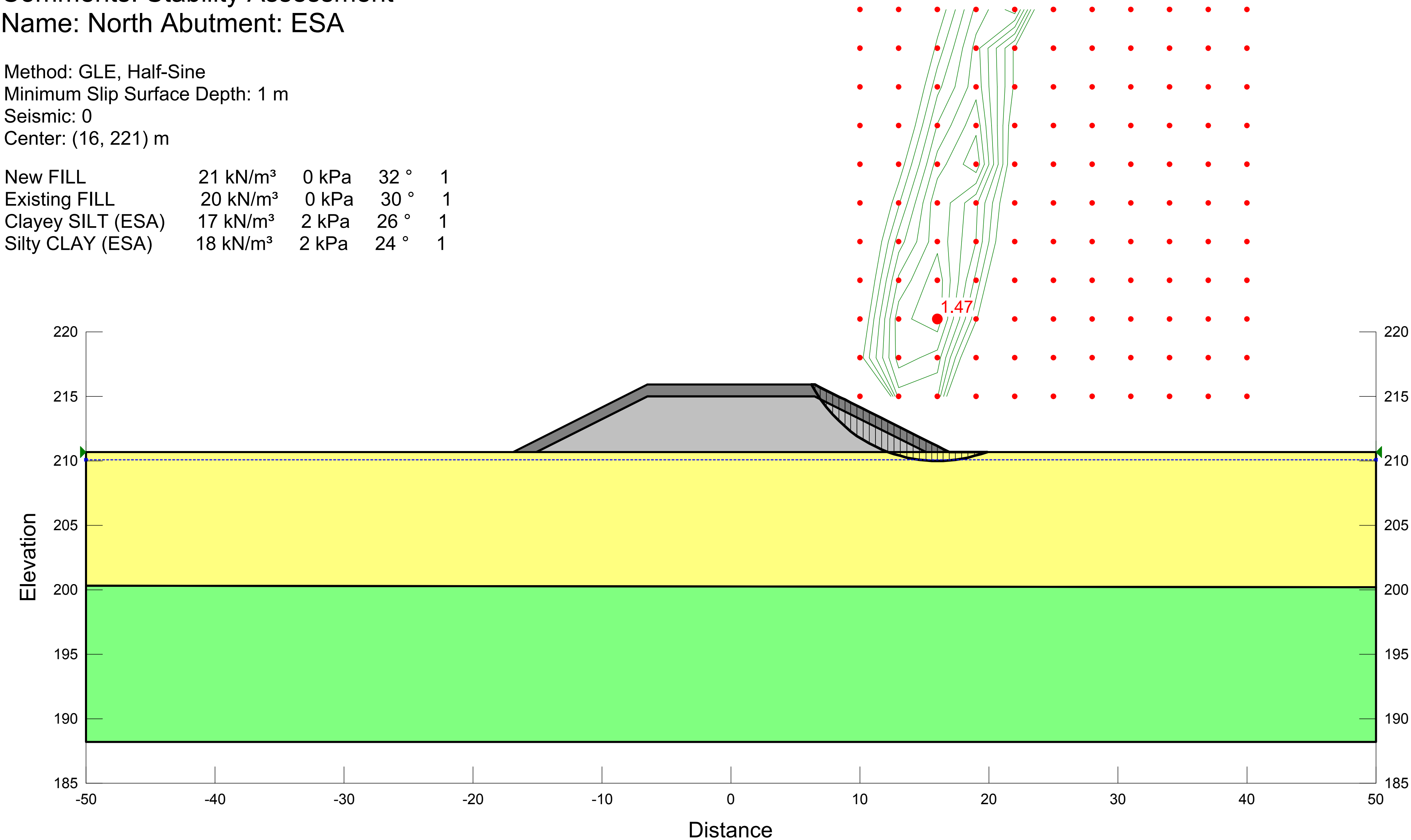

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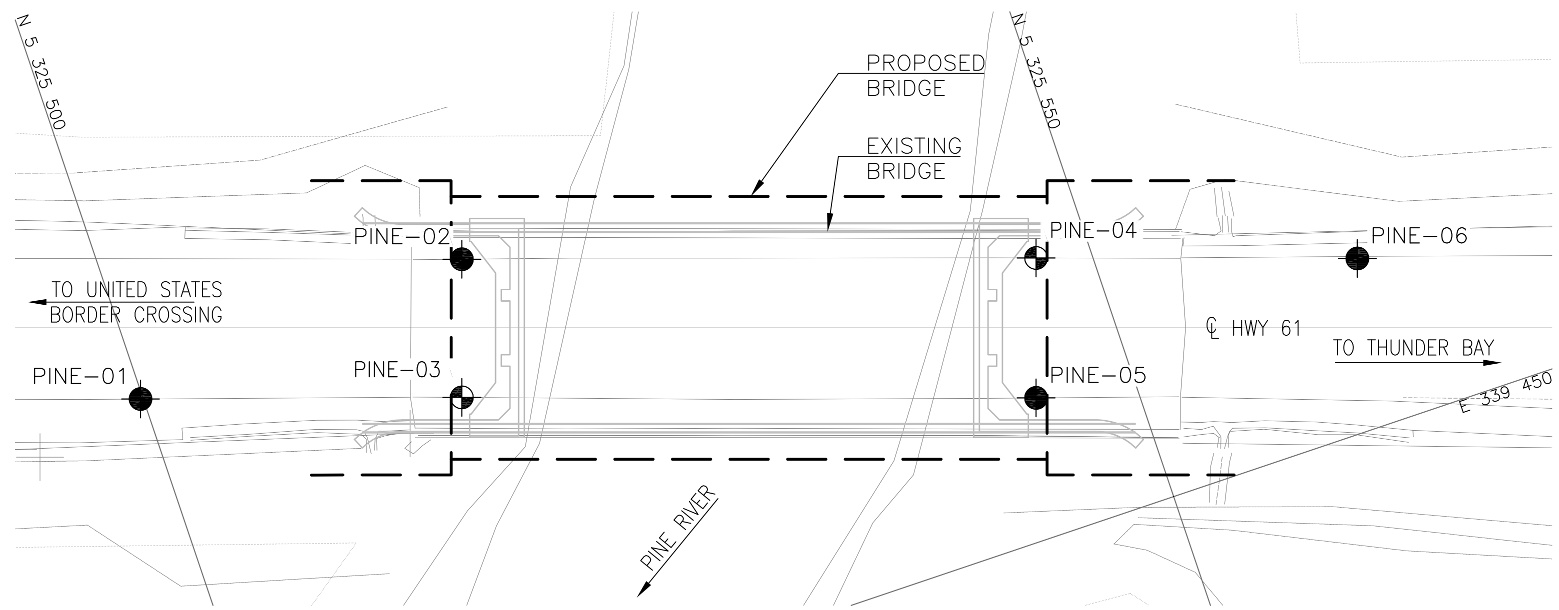
Figure 6

Appendix G

Borehole Locations and Soil Strata Drawing

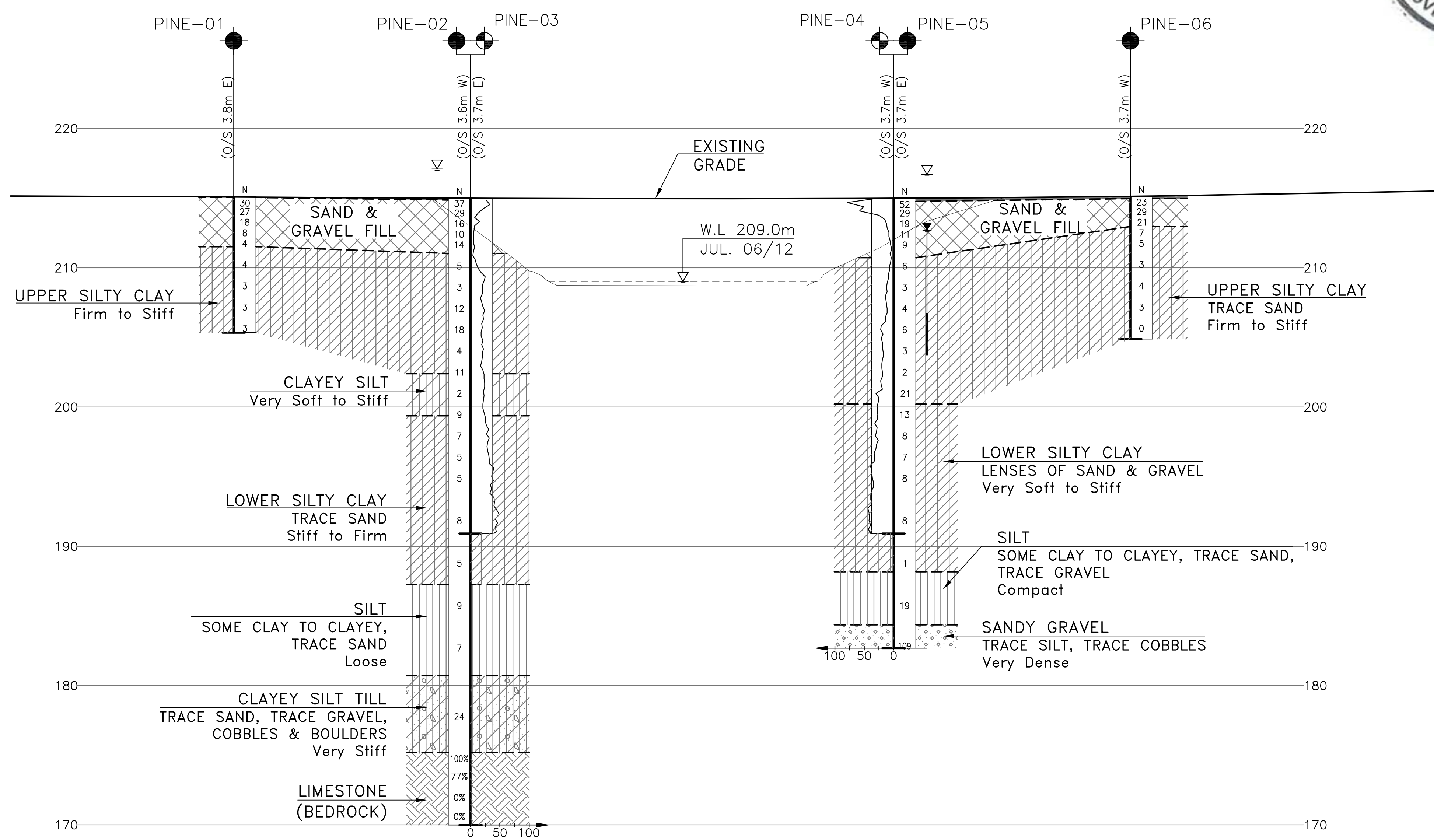
MINISTRY OF TRANSPORTATION, ONTARIO



PLAN

8 0 8 16m

SCALE 1:400



PROFILE ALONG ℄ HWY 61

8 0 8 16m

SCALE 1:400

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
Jan 15/15
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Jan 15/15
PROVINCE OF ONTARIO

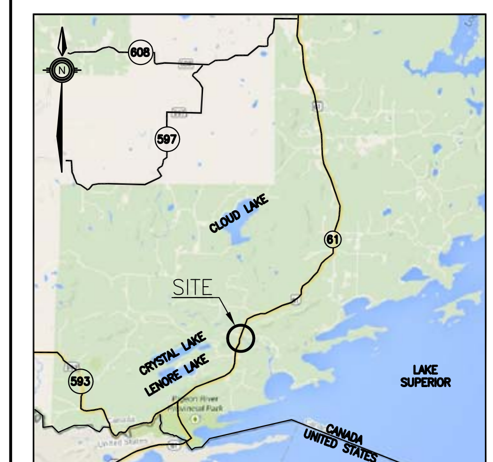
CONT No
WP No 6098-10-01

HIGHWAY 61
PINE RIVER BRIDGE
STRUCTURAL REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 11

Hatch Mott MacDonald

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

●	Borehole
⊕	Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
⊻	Water Level During Drilling
⊻	Water Level In Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
PINE-01	215.1	5 325 499.9	339 427.6
PINE-02	215.0	5 325 518.3	339 426.0
PINE-03	215.0	5 325 516.0	339 433.0
PINE-04	215.0	5 325 547.1	339 435.7
PINE-05	215.0	5 325 544.8	339 442.7
PINE-06	215.1	5 325 563.2	339 441.2

-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. 52A-196

REVISIONS	DATE	BY	DESCRIPTION

DESIGN AP | CHK MRA | CODE CAN/CSA S6-06 | LOAD CL-625-ONT | DATE JAN 2015
DRAWN AN | CHK AP | SITE 48W-105 | STRUCT | DWG 2

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