THURBER ENGINEERING LTD.

FOUNDATION INVESTIGATION AND DESIGN REPORT
PINEWOOD RIVER LRB BRIDGE #1 REPLACEMENT
TOWNSHIP OF NELLES, DISTRICT OF RAINY RIVER, ONTARIO
SITE NO. 45-31

ASSIGNMENT NO. 6015-E-0023
W.O.# 2017-11036

GEOCRES No.: 52D-31

Report

to

MINISTRY OF TRANSPORTATION ONTARIO

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual data obtained from a foundation investigation carried out by Thurber Engineering Ltd. (Thurber) at the existing Pinewood River Bridge #1 in the Township of Nelles, District of Rainy River, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and to provide a borehole location plan, stratigraphic profile, record of borehole sheets, laboratory test results, and a written description of the subsurface conditions encountered at the site.

Thurber was retained by Ministry of Transportation Ontario (MTO) to carry out this foundation investigation under the MTO Retainer Agreement Number 6015-E-0023.

2. SITE DESCRIPTION

The site is located on a farm road, approximately 350 m south of Brown Road and 1.6 km east of Highway 619, in the Township of Nelles, District of Rainy River, Ontario. The key plan showing the general location of the bridge site is presented on the Borehole Location and Soil Strata Drawing in Appendix D.

The farm road runs in a general north-south direction at the bridge site. The existing structure is a single span, 4.6 m long and 6.7 m wide timber bridge and has an unknow construction date. The bridge superstructure consists of timber decking resting on timber girders. The sub-structure consists of timber abutments resting on timber bents, which act as pile caps. Timber ballast walls extending from top to bottom behind the bents and beyond the deck width are acting as abutment and wingwall.

Pinewood River at the bridge site flows in a west to east direction. The land surrounding the site generally contains low lying agricultural lands with several bogs and marshes within the vicinity of the site. The land is generally of low relief, undulating plains. Photographs in Appendix C show the general nature of the site and the existing bridge.

Based on published geological information, the subsurface materials at the bridge site consist of silt and clay of glaciolacustrine plains and alluvial deposits over bedrock.

3. INVESTIGATION PROCEDURES

The site investigation and field testing for this bridge consisted of drilling and sampling four (4) boreholes (17-09, 17-09R, 17-10 and 17-10R) to depths ranging between 7.0 m and 29.6 m below the existing ground surface.

The original scope of work for the foundation investigation of this assignment included advancing two boreholes at the site. Boreholes 17-09 and 17-10 were originally advanced at the site on April 21 and 22, 2017. However, the soil samples retrieved from the boreholes were lost by the shipping company during their transit from Thunder Bay to Thurber's laboratory. Therefore, a second mobilization was made by Thurber on June 6 and 7, 2017 during which time Boreholes 17-09R and 17-10R were drilled to obtain sample of the overburden soils for laboratory testing. This was discussed and agreed upon with the MTO Foundation Office.

Boreholes 17-09 and 17-09R were drilled on the south side of the existing bridge and Boreholes 17-10 and 17-10R were drilled on the north side.

The approximate locations of the boreholes are shown on the Boreholes Locations and Soil Strata Drawing included in Appendix D.

Utility clearances were obtained prior to the start of drilling. The ground surface elevations for the boreholes were derived from cross sections and topographic drawings provided to Thurber by MTO. The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing included in Appendix D. The coordinate system MTM NAD 83, Zone 15 was used for these boreholes.

The boreholes were drilled using a rubber tire buggy mounted drill rig equipped with continuous flight, hollow and solid stem augers. Samples of the overburden soils were obtained from the boreholes at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). A Dynamic Cone Penetration Test (DCPT) was carried out near Boreholes 17-09 and 17-10 to a depth of approximately 30.5 m.

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

Groundwater conditions in the open boreholes were observed during and after the drilling operations. The boreholes were backfilled in general accordance with MOE Regulation 903. Completion details of the boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Borehole Number	Coordinates (MTM NAD 83, Zone 15)		Ground Surface Elevation (m)	Termination Depth (m)	Completion Details
	Northing (m)	Easting (m)			
17-09	5,405,211.7	211,215.4	332.1	29.6	Bentonite holeplug and cuttings to ground surface.
17-09R	5,405,212.3	211,220.6	332.3	7.0	Backfilled with cuttings to ground surface.
17-10	5,405,221.9	211,222.1	332.3	29.6	Standpipe piezometer was installed in the borehole. After removel of the piezometer, the borehole was backfilled with bentonite holeplug and cuttings to ground surface.
17-10R	5,405,222.2	211,217.1	332.4	10.1	Bentonite holeplug and cuttings to ground surface.

4. LABORATORY TESTING

All recovered soil samples from Boreholes 17-09R and 17-10R were subjected to visual identification and natural moisture content determination. Selected samples were subjected to grain size distribution analyses (sieve and hydrometer) and Atterberg limit testing. The results of the testing program are summarized on the Record of Borehole sheets included in Appendix A and on the figures presented in Appendix B.

In order to assess the potential for sulphate attack on concrete foundations, as well as the potential for corrosion associated with any metal portion of the structure, a sample of the existing native soil, and a sample of the surface water from the river upstream of the existing culvert were collected. The samples were submitted to SGS Canada Inc., a CALA accredited analytical laboratory in Lakefield, Ontario, for analytical testing of corrosivity parameters and sulphate content. The results of the analytical testing are summarized in Section 6 and are presented in

Appendix B.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. The Record of Boreholes 17-09 and 17-10 were prepared based on the visual identification at the site only. Details of the encountered soil stratigraphy are presented in these sheets and on the "Borehole Locations and Soil Strata" drawing included in Appendix D. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and should be used for interpretation of site conditions. It must be recognized and expected that soil conditions may vary between and beyond the borehole locations.

In general, the subsurface conditions consisted of silty sand fill overlying a deposit of soft to very stiff silty clay to clay. Descriptions of the individual strata are presented below.

5.1 Silty Sand Fill

Silty sand fill to gravelly silty sand fill containing trace clay was encountered in all boreholes and the fill extended to depths of 0.8 m to 1.8 m below the existing ground surface (Elevations 330.5 m to 331.5 m). The fill was generally brown in colour and contained rootlets and organics.

SPT 'N' values recorded in the silty sand fill ranged from 4 to 13 blows per 0.3 m of penetration, indicating a loose to compact relative density. Measured moisture contents in the fill ranged from 3% to 19%.

The results of grain size distribution analyses conducted on samples of the fill are presented on the Record of Borehole sheets included in Appendix A and are summarized in the following table. The results are also presented on Figure B1 in Appendix B.

Soil Particle	**Percentage (%)**
Gravel	11 and 23
Sand	74 and 59
Silt	10
Clay	5
Silt and Clay	18

5.2 Silty Clay

A deposit of silty clay was encountered below the fill in all boreholes. The boreholes were terminated within the deposit at depths ranging from 7.0 m to 29.6 m (Elevations 325.3 m and 302.5 m). The silty clay was brown to grey in colour.

SPT 'N' values recorded in the deposit varied between 0 (i.e., 0.3 m of penetration under static weight of hammer) and 8 blows per 0.3 m penetration. In-situ vane shear testing conducted in the silty clay measured undrained shear strengths ranging from 21 kPa to greater than 100 kPa indicating that the silty clay to clay is soft to very stiff, predominantly soft to firm. The clay gets stiffer with depth. Measured moisture contents in the silty clay to clay ranged from 26% to 70%.

The results of Atterberg Limits tests conducted on selected samples of the deposit 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:

Measured Limit	Percentage (%)
Liquid Limit	49 to 89
Plastic Limit	30 to 57
Plasticity Index	19 to 32

The results of the Atterberg Limits testing indicate that the deposit is a silty clay of intermediate plasticity (CI) to clay of high plasticity (CH).

5.3 Groundwater Conditions

Where possible, water levels were measured in the open boreholes during and upon completion of drilling. One standpipe piezometer was installed in Borehole 17-10. The piezometer was decommissioned upon taking a water level measurement of 3.8 m below surface (Elevation

328.5 m), one day after completion of drilling on April 22, 2017. The results of the groundwater readings are presented in Table 5.1.

Table 5.1 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
17-09	April 22, 2017	4.6	327.5	In open borehole
17-09R	June 7, 2017	Dry	-	Borehole was open to a depth of 4.0 m
17-10	April 21, 2019	1.2	331.1	In open borehole
	April 22, 2017	3.8	328.5	In standpipe piezometer*
17-10R	June 6, 2017	Dry	-	Borehole was open to a depth of 9.0 m

*: The screen of the standpipe piezometer in Boreholes 17-10 was between depths of 25.9 m and 28.9 m within the clay deposit.

The normal and the high river water levels were reported to be about 330.9 m and 331.8 m.

The groundwater levels are very short-term readings and seasonal fluctuations of the groundwater levels are to be expected. In particular, the groundwater levels may be at a higher elevation after periods of significant or prolonged precipitation.

6. CORROSIVITY AND SULPHATE TEST RESULTS

One sample of the native soil from Borehole 17-09R and a sample of the river water were submitted for chemical testing of corrosivity parameters and sulphate. The results of the analytical tests are shown in Table 6.1. The laboratory certificates of analysis are presented in Appendix B.

Table 6.1 – Analytical Test Results

Parameter	Units (Soil)	Units (Water)	Test Results	
			17-09R SS#6, 4.5 m – 5.1 m (Silty Clay)	River Water
Sulphide	%	mg/L	0.03	0.018

Parameter	Units (Soil)	Units (Water)	Test Results	
			17-09R SS#6, 4.5 m – 5.1 m (Silty Clay)	River Water
Chloride	µg/g	mg/L	1.9	0.68
Sulphate	µg/g	mg/L	51	0.43
pH	No unit	No unit	8.48	6.89
Electrical Conductivity	µS/cm	µS/cm	157	78
Resistivity	Ohms.cm	Ohms.cm	6,370	12,900
Redox Potential	mV	mV	12	207

7. MISCELLANEOUS

Thurber obtained subsurface utility clearances prior to drilling. Thurber obtained the northing and easting coordinates and ground surface elevations from measurements taken in the field relative to the topographic plans provided by Hatch.

RPM Drilling Inc. of Thunder Bay, Ontario supplied and operated the drilling, sampling and in-situ testing equipment for the field investigation. The field investigation was supervised on a full time basis by Mr. Amir Fereidouni of Thurber. Overall supervision of the field program was provided by Mr. Cory Zanatta, B.A.Sc. of Thurber.

Geotechnical laboratory testing was carried out at Thurber's geotechnical laboratory. Analytical laboratory testing was carried out by SGS Canada Inc. Interpretation of the field data and preparation of this report was carried out by Mr. Cory Zanatta, B.A.Sc., EIT and Mr. Mehdi Mostakhdemi, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

8. GENERAL

This report provides an interpretation of the geotechnical data in the factual report, and presents foundation design recommendations for the proposed replacement of Pinewood River Bridge #1 in the Township of Nelles, District of Rainy River, Ontario.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The design-build contractor must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The existing bridge is located on a farm road, approximately 350 m south of Brown Road and 1.6 km east of Highway 619. The farm road runs in a general north-south direction at the bridge site. The existing structure is a single span, 4.6 m long and 6.7 m wide timber bridge and has an unknow construction date. The bridge superstructure consists of timber decking resting on timber girders. The sub-structure consists of timber abutments resting on timber bents. Timber ballast wall extending from top to bottom behind the bents and beyond the deck width are acting as abutment and wingwall.

The November 2014 structural inspection report indicates that the bridge is in a poor condition due to the abutment-ballast walls rotating, and cracking and rotting of all timber elements and the deck.

9. STRUCTURE REPLACEMENT ALTERNATIVES

This section presents discussions on proposed replacement options and foundation alternatives, and provides foundation design recommendations for the replacement of the Pinewood River Bridge#1. It is understood that the replacement bridge will be longer than the existing bridge.

The Structural Design Report (SDR) has discussed three (3) replacement options for this site:

- A Single Cell Precast Concrete box (closed) culvert;
- A Precast Slab supported on Sheet Pile Abutments; and
- A 9.3 m long Lessard Modular Bridge with sheet pile wall abutments and wingwalls.

In general, the foundation soil stratigraphy consists of silty sand fill materials over a mainly soft to firm silty clay. The short-term water level in the boreholes was at about Elevation 328.5 m. The high-water level is reported to be at 331.8 m and the normal reported water level is 330.9 m. The groundwater level will likely reflect the river water level.

All three replacement alternatives are feasible from foundation design perspective. However, the SDR indicates that the concrete box culvert and the precast slab options are not feasible without significant grade raise at the site. The SDR has indicated that the modular bridge option is the preferred alternative for the proposed replacement. No grade raise is mentioned for this option. Therefore, only this option was further discussed in this report.

9.1 Structure Foundation Design for Modular Bridge

Based on the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments of a modular bridge at the site. A summary of the advantages and disadvantages associated with each option is provided below:

- **Footings on Native Soils:** Footings placed directly on the native soft to firm silt clay below the fill are not recommended due to the relatively low geotechnical resistances available in the silty clay and its relatively high compressibility.

- **Footings on a compacted granular pad in the approach embankments:** This option would be advantageous in providing higher geotechnical resistances compared to the footing option supported on native soil.
- **Driven steel piles:** Driven steel H-piles are feasible for support of new abutments although this is likely to be a more expensive foundation option compared with footings on a granular pad.

A comparison of the foundation options based on their respective advantages and disadvantages is included in Appendix E. Recommendations for design of the feasible foundation options are also presented below and a preferred foundation option is recommended.

9.2 Spread Footings on Engineered Fill

The preliminary GA drawing for the modular bridge option indicates a footing founding elevation of about 331 m. At that elevation, both abutments will be founded in the existing sand fill or native soft to firm silty clay. Any loose/very soft materials containing organics should be sub-excavated from the underside of the footings and backfilled with engineered fill. The following construction sequence is recommended for the footing construction:

1. Excavate to remove all timber abutments, boulders, rock protection and other deleterious material from the footprint of the new foundations.
2. The minimum depth of excavation must accommodate the concrete foundation slab and at least 1.0 m of engineered fill below the slab, as described below.
3. The subgrade below the 1 m engineered fill pad should be inspected to detect and sub-excavate soft spots and confirm that the subgrade is uniformly compacted.
4. The dimensions of the base of the excavation must be determined by assuming a granular pad 1.0 m wider than the footing at the level of the footing base and projecting outward at 2H:1V.

The excavation for the footings will be conducted after installation of the sheet pile abutment walls. The base of the excavation may be below the river water level and/or the groundwater table at the site. Temporary dewatering within the area of the proposed foundation footprint may be required depending on the groundwater level at the time of construction.

The engineered fill pad should consist of OPSS Granular “A” or Granular B Type II placed in 150 mm lifts and compacted to 100% of its SPMDD at ±2% of optimum moisture content. The top of the engineered fill pad should be at least 1 m wider than the footprint of the spread footing at the underside of the footing and must be at least 1.0 m thick.

Excavations for the engineered pad construction and footing placement will likely require the existing superstructure to be removed or temporarily supported during construction.

The following axial geotechnical resistances may be used for design of 1.5 m to 2 m wide spread footings of founded at or below Elevation 331 m on at least 1 m thick engineered fill:

- Factored Geotechnical Resistance at Ultimate Limit State (ULS) of 125 kPa
- Factored Geotechnical Resistance at Serviceability Limit State (SLS) of 65 kPa for a settlement of 25 mm.

The consequence factor of 1 was utilized in this design adopting a “typical” consequence level. The geotechnical resistance factor of 0.5 for bearing, and 0.8 for settlement (both adopted for “typical” degree of understanding) were used to obtain the above values, in accordance with Section 6.9 of the Canadian Highway Bridge Design Code (CHBDC) 2014.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should therefore be reviewed if the foundation width or founding elevation differs significantly from that given above.

The lateral resistance developed along the base of the footings founded on the engineered fill should be computed using an ultimate friction coefficient of 0.6 for cast-in-place concrete and 0.5 for pre-cast concrete. The friction coefficients provided above are “ultimate” values and require a degree of sliding movement to occur to fully mobilize the resistance.

9.3 Driven Piles

Bedrock and/or competent soils (i.e., hard/dense to very dense) were not encountered within a depth of about 30 m below the existing ground surface at the site and end bearing resistance of the piles are not expected to be sufficiently high. Therefore, a system of driven steel H-piles developing resistance primarily through shaft friction (friction piles) could be considered to support the bridge loads. Given the soft and compressible nature of the soils directly below the shallow existing embankment fill, friction piles will have to be driven to significant depths into that deposit to develop adequate resistance. HP 310x110 piles driven to about Elevation 315 m in the firm to soft silty clay to clay may be designed using factored axial geotechnical resistances at ULS and SLS of 225 kN and 200 kN, respectively.

The pile length was estimated to be about 16 m, assuming a pile cut-off at Elevation 331 m. The recommended pile tip elevation is considered approximate and the actual tip elevation required to develop the design resistance will need to be confirmed by monitoring during installation.

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

The piles should be installed behind the existing timber abutments or the timber abutments must be removed prior to pile driving.

Piles should be installed in accordance with OPSS 903. Pile driving at both abutments 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” should have a minimum value of twice the design load at ULS.

Pile tip protection should not be used for driven H-piles developing resistance through shaft friction at this site.

The alignment of the H-piles should be carefully selected to be outside of the footprint of the existing crib abutments and away from the river banks.

9.4 Preferred Foundation Option

From a geotechnical perspective and based on the subsurface conditions, spread footings on engineered fill are considered as the preferred option for the modular bridge alternative. Deep foundations are likely to be more expensive and hence has not been discussed any further.

9.5 Frost Protection

The depth of frost penetration at this site is approximately 2.2 m as per Ontario Provincial Standard Drawing (OPSD) 3090.100 (Foundation Frost Depths for Northern Ontario).

If piles are used, the base of pile caps must be provided with a minimum of 2.2 m of earth cover as protection against frost action. If it is not practical to provide 2.2 m of earth cover, consideration should be given to providing the frost protection using expanded polystyrene insulation (EPS). Typically, 25 mm of EPS can be considered equivalent to 600 mm of earth cover. If EPS is used, it must be provided with long term protection against erosion, environmental degradation and spills.

Concrete slab foundations for modular bridges may be founded on an engineered fill pad with a minimum embedment of 0.5 m.

9.6 Stability and Settlement

Since no grade raise is proposed for this site, no stability or settlement issues are anticipated.

10. EXCAVATION AND GROUNDWATER CONTROL

Where excavations extend below the water level, the Contractor must implement effective dewatering procedures.

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 and the native silty clay to clay may be classified as Type 3 soil.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902.

The dewatering system on site should conform to OPSS 518 (Construction Specifications for Control of Water from Dewatering Operation). The design of an effective dewatering system that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility and the need to engage a dewatering specialist. Suggesting wording for an NSSP in this regard is included in Appendix F. Additional assessment should be made to determine if a Permit to Take Water (PTTW) is required.

Stockpile of excavated materials and heavy construction equipment should be kept at least the same horizontal distance from the edge of excavation as the depth of the excavation to prevent local instabilities.

11. TEMPORARY SUPPORT SYSTEM

If required, the temporary excavation support system must be designed and constructed in accordance with OPSS 539. The protection system should be designed for Performance Level 2 (maximum 25 mm horizontal deflection). The Contractor should select the wall type and design taking into account the soil conditions encountered in the boreholes.

The following parameters may be used for design of the temporary shoring system:

γ = 21 kN/m^3 (bulk unit weight for fill)

= 19 kN/m^3 (bulk unit weight for native silts and sands)

γ' = 11 kN/m^3 (submerged unit weight for fill)

= 9 kN/m^3 (submerged unit weight for native silts and sands)

K_a = 0.33 (active pressure coefficient for fill)

= 0.39 (active earth pressure coefficient for native soil)

K_p = 3.0 (passive pressure coefficient for fill)

= 2.6 (passive earth pressure coefficient for native soil)

Full hydrostatic pressure should be considered assuming a water level at least equal to the design stream water level.

The design of temporary protection system is the responsibility of the Contractor. The actual pressure distribution acting on the protection/shoring system is a function of the construction sequence and the relative flexibility of the retaining system, and these factors have to be considered when designing the shoring system. All protection systems should be designed by a Professional Engineer experienced in such designs, who will determine an appropriate support system.

12. LATERAL EARTH PRESSURES

Backfill to the abutments for the modular bridge should consist of free-draining, non-frost susceptible granular materials such as Granular A or B Type II conforming to the requirements of OPSS.PROV 1010. Reference should be made to the backfill arrangements stipulated in OPSD 803.010, as appropriate.

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 Tables 12.1 and 12.2)

γ = unit weight of retained soil (see Tables 12.1 and 12.2)

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)

Loading Condition	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 Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active K_A (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At-rest K_0 (Restrained Wall)	0.43	-	0.47	-
Passive K_P	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 active and passive earth pressure coefficients 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 general the lateral earth pressure applied to a retaining structure depends on the lateral movement of the structure to activate active, passive or at rest earth pressure. If the wall support does not allow lateral movement (restrained wall), at rest earth pressures should be assumed for design. If the wall support allows lateral movements (unrestrained stem), active earth pressure should be used in the design of the wall. The minimum lateral movement to allow active pressures to develop within the backfill is outlined in Section C6.12 of the Commentary on CHBDC 2014.

In accordance with Clause 6.12.3 of the CHBDC 2014, a lateral pressure representing the compaction surcharge should be added in design of retaining walls. The magnitude of the lateral pressure should be 12 kPa at the top of fill which linearly decreases to zero at a depth of 1.7 m (for Granular B Type I) or at a depth of 2.0 m (for Granular A or B Type II). If the wall is retaining sloping backfill, appropriate earth pressure parameters from Table 11.1 for sloping backfill should be used.

12.1 Sheet Pile Abutment Walls for Modular Bridge Option

The SDR considers the use of sheet pile wall abutments (and wingwalls) for the modular bridge configuration. The sheet piles will provide containment and resistance to lateral earth pressures applied from the approach fill. The sheet piles should be installed behind the existing timber abutments or the timber abutments should be removed prior to sheet pile installation.

The stability of the sheet pile wall system (including but not limited to global stability, basal stability, anchor design, bending) should be evaluated by the wall designer and the depths of penetration (or sheet pile tip elevations) be determined for a minimum factor of safety of 1.5 using the geotechnical design parameters presented in Table 12.2. The lateral impact of the foundations loads on the sheet pile wall system should be taken into account in the design if shallow spread footings on engineered fill is considered to support the modular bridge foundations.

The interaction between the sheet pile wall and the adjacent soil may be analysed using a soil-spring model and a coefficient of horizontal subgrade reaction, k_s. The value of k_s for cohesive soils is shown in the table below and may be assumed to be constant with depth. In cohesionless soils, the horizontal subgrade reaction per linear meter varies with depth and can be calculated as follows:

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

where z = depth of embedment in metres

n_h = coefficient related to soil density, see table below ($\mathrm{kN/m^3}$)

For soil-spring analysis, the spring constant, K_s, may be obtained by the expression $K_s = k_s L$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction ($\mathrm{kN/m^3}$) and L is the length (m) of the pile segment or element used in the analysis. The ultimate passive pressure mobilized per unit length of the pile should not exceed the value provided below:

$$P_{ult} = k_p.\gamma'.\, z$$

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 12.2 – Soil Parameters for Sheet Pile Analysis

Foundation Element (Reference Borehole)	Soil Unit	Elevation (m)		γ' (kN/m^3)	K_a	K_0	K_p	k_s (kN/m^3)	n_h (kN/m^3)
		Top	Bottom						
South Abutment (17-09 and 17-09R)	Existing Fill	332.1*	330.5	21	0.33	0.5	3.0	-	2,500
	Firm to soft silty clay to clay	330.5	302.5	9	0.39	0.56	2.6	1,650	-
North Abutment (17-10 and 17-10R)	Existing Fill	332.3*	331.3	21	0.33	0.5	3.0	-	2,500
	Firm to soft silty clay to clay	331.3	302.5	9	0.39	0.56	2.6	1,650	-

Note: * Elevation of top of sheet pile varies.

In general, backfill to the sheet pile walls should be in accordance with OPSS 902 and should consist of Granular A, Granular B Type II or 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.PROV 501.

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.

12.2 Cellular Bin Type Abutment Walls

This type of walls, if used, may be supported on an engineered granular pad resting on the native silty clay subgrade. Any topsoil/organic must be removed from the wall subgrade and replaced with granular fill compacted as per OPSS 501. The engineered pad is required to provide subgrade uniformity along the wall alignment. This pad should consist of compact Granular A materials and have a minimum thickness of 0.5 m. Local sub-excavation may be required to accommodate the design grades or to remove unsuitable subgrade materials. The walls should be founded at or below Elevation 331.0 m and the base of the granular pad should be founded at or below 330.5 m. For a 1 m to 2 m high wall founded on a 0.5 m thick granular pad a factored geotechnical resistance at ULS of 100 kPa and a geotechnical resistance at SLS of 65 kPa (for 25 mm of settlement) may be used for design.

Resistance to lateral forces / sliding resistance between wall base and the underlying engineered gravel pad should be evaluated in accordance with the CHBDC 2014 assuming an ultimate coefficient of friction of 0.4.

Lateral earth pressures acting on the walls should be computed as described in Section 12. If the wall is retaining sloping backfill, appropriate earth pressure parameters for sloping backfill should be used.

13. SEISMIC CONSIDERATIONS

The new structure is considered as a Seismic performance category 1 based on Table 4.10 of the CHBDC 2014; therefore, it does not need to be analyzed for seismic loads regardless of its importance and geometry in accordance with Section 4.4.5.1 of the CHBDC 2014.

14. EMBANKMENT RESTORATION

The existing road embankment slopes appear to be performing satisfactorily. Provided that the embankment is reconstructed at the same slope inclination as the existing embankment, but not steeper than 2H:1V, the restored embankment slope should remain stable.

It is anticipated that there will be no significant grade raise or embankment widening at this site for the bridge replacement, and therefore settlement of the embankment is not a concern. Any settlement due to changes in the bridge configuration is expected to be less than 25 mm.

Embankment restoration after completion of the replacement should be carried out in accordance with OPSS.PROV 206. The embankment material may consist of imported Granular A, Granular B Type II, or Granular B Type III material. Alternatively, the existing embankment fill may be used, provided it is unfrozen, free of organics, and at a moisture content that is suitable for compaction.

In general, surface vegetation, topsoil, organic deposits, disturbed material or otherwise loose/soft soils should be stripped from the foundation footprints, and within the embankment footprints. Inspection and approval of the foundation surfaces by qualified geotechnical personnel should be conducted.

15. SCOUR AND EROSION PROTECTION

Erosion protection should be provided at the bridge abutment. Design of the erosion protection measures should consider hydrologic and hydraulic factors and should be carried out by specialists experienced in this field.

Typically, rock protection should be provided over all surfaces with which river water is likely to be in contact. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

16. CORROSION & SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on the native soil and the river water indicate the following:

- The potential for sulphate attack on concrete foundations from the surrounding soil or surface water is considered to be negligible due to the low concentration of sulphate in the samples tested.
- The potential for corrosion on metal structural elements is considered to be mild.
- The effect of road de-icing salt should be considered in the choice of concrete and metal structure elements.

17. CONSTRUCTION CONCERNS

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

- Seasonal fluctuations of the groundwater and river level are to be expected. In particular, the water level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall, which may impact the construction.
- The Contractor’s selection of construction equipment and methodology should include assessment of the capability of the existing embankment 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 during construction. The design and safety of any temporary works is the responsibility of the Contractor.
- Native soil below the fill at this site consists of soft to firm clay. Significant grade raises will cause long term settlements and therefore, should be avoided.

18. CLOSURE

Engineering analysis and preparation of this report was carried out by Cory Zanatta, EIT., and Mr. Mehdi Mostakhdemi, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Oct 13/17
PROVINCE OF ONTARIO

P.K. Chatterji, P.Eng.
Review Principal, Designated MTO Contact

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.

RECORD OF BOREHOLE No 17-09

1 OF 4

METRIC

W.P. ____
LOCATION Pinewood River Bridge N 5 405 211.7 E 211 215.4
ORIGINATED BY AHF
HWY ____
BOREHOLE TYPE Hollow Stem Augers/DCPT
COMPILED BY AN
DATUM Geodetic
DATE 2017.04.22 - 2017.04.22
CHECKED BY CZ

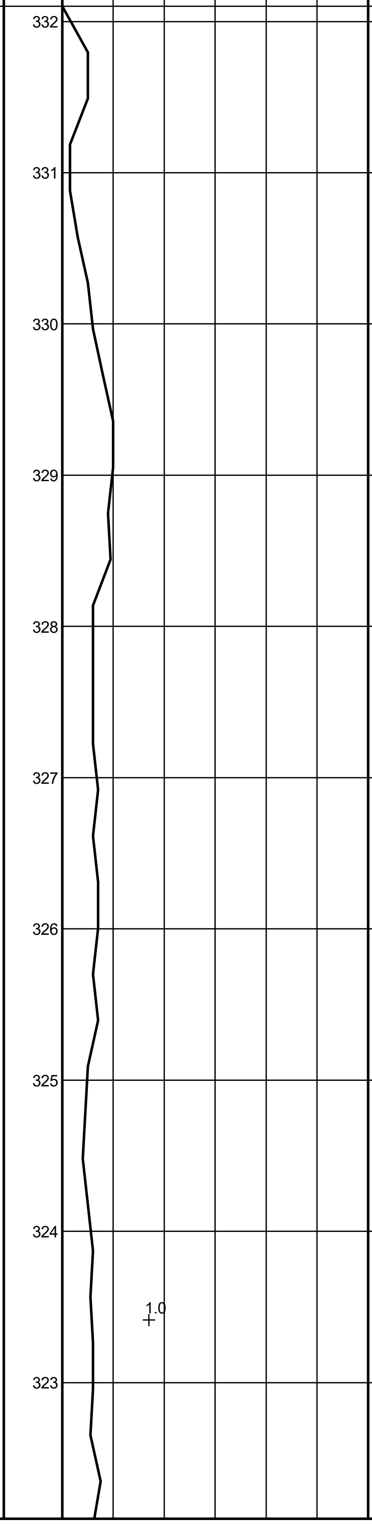
ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
332.1	GROUND SURFACE						
0.0	Silty **SAND**, some clay and gravel, trace roots and rootlets and organics Brown Moist (FILL)	1	GS				
331.3 0.8	Start DCPT from surface Silty **CLAY**, some sand and gravel Soft to Firm Brown Moist	1	SS	3			
		2	SS	6			
	Grey below 2.1m Trace sand and gravel, trace roots and rootlets	3	SS	3			
		4	SS	1			
		5	SS	1	▽		
		6	SS	1			
		7	SS	1			
		8	SS	WH			

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



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

ONTMT4S MTO-17792.GPJ 2017TEMPLATE(MTO).GDT 7/7/17

RECORD OF BOREHOLE No 17-09

2 OF 4

METRIC

W.P.		ORIGINATED BY	AHF
LOCATION	Pinewood River Bridge N 5 405 211.7 E 211 215.4	COMPILED BY	AN
HWY		CHECKED BY	CZ
BOREHOLE TYPE	Hollow Stem Augers/DCPT		
DATUM	Geodetic		
DATE	2017.04.22 - 2017.04.22		

SOIL PROFILE — ELEV DEPTH | DESCRIPTION | STRAT PLOT

Continued From Previous Page

Silty **CLAY**
Firm to Stiff
Grey
Wet

SAMPLES

NUMBER	TYPE	"N" VALUES
9	SS	1
10	SS	4
11	SS	4
12	SS	3
13	SS	1
14	SS	1

GROUND WATER CONDITIONS

ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

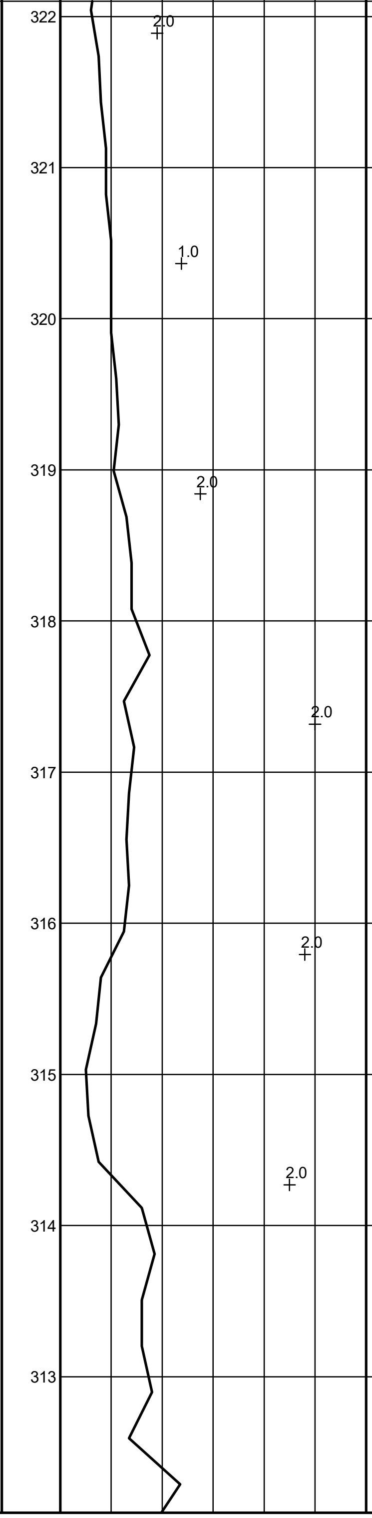
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

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



Continued Next Page

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

20 15 5 10 (%) STRAIN AT FAILURE

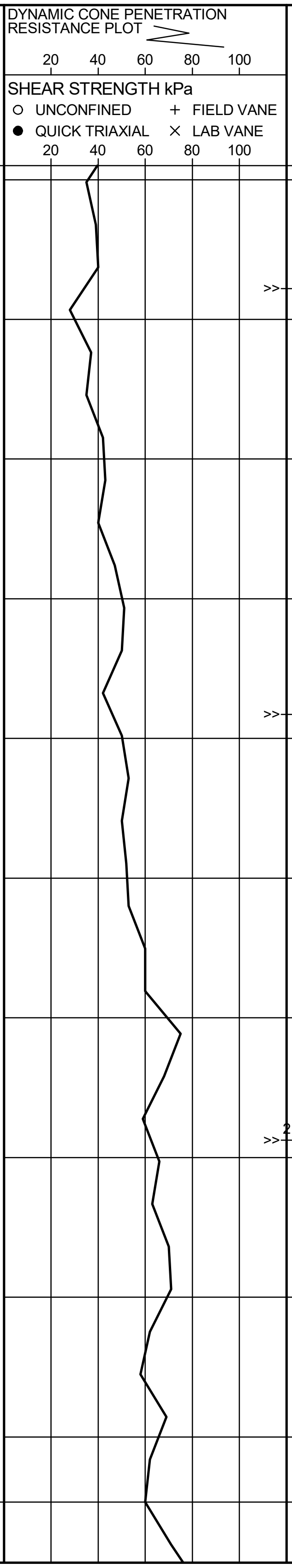
RECORD OF BOREHOLE No 17-09

3 OF 4 **METRIC**

W.P.		ORIGINATED BY	AHF
LOCATION	Pinewood River Bridge N 5 405 211.7 E 211 215.4		
HWY		COMPILED BY	AN
BOREHOLE TYPE	Hollow Stem Augers/DCPT		
DATUM	Geodetic	CHECKED BY	CZ
DATE	2017.04.22 - 2017.04.22		

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page							
	Silty **CLAY**, some sand Stiff to Very Stiff Grey Wet		15	SS	3		312	
							311	
							310	
			16	SS	1		309	
							308	
							307	
	Trace sand and gravel		17	SS	1		306	
							305	
							304	
302.5			18	SS	3		303	
29.6	END OF BOREHOLE AT 29.6m. WATER LEVEL AT 4.6m. BOREHOLE BACKFILLED WITH							

Continued Next Page



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

UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

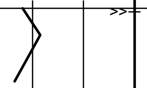
THURBER

RECORD OF BOREHOLE No 17-09

4 OF 4

METRIC

W.P. ____________ LOCATION Pinewood River Bridge N 5 405 211.7 E 211 215.4 ORIGINATED BY AHF
HWY ____________ BOREHOLE TYPE Hollow Stem Augers/DCPT COMPILED BY AN
DATUM Geodetic DATE 2017.04.22 - 2017.04.22 CHECKED BY CZ

SOIL PROFILE			SAMPLES					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 Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	BENTONITEH HOLEPLUG AND CUTTINGS TO SURFACE. END OF DCPT AT 30.5m.										

+ ³, × ³: Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

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Ontario

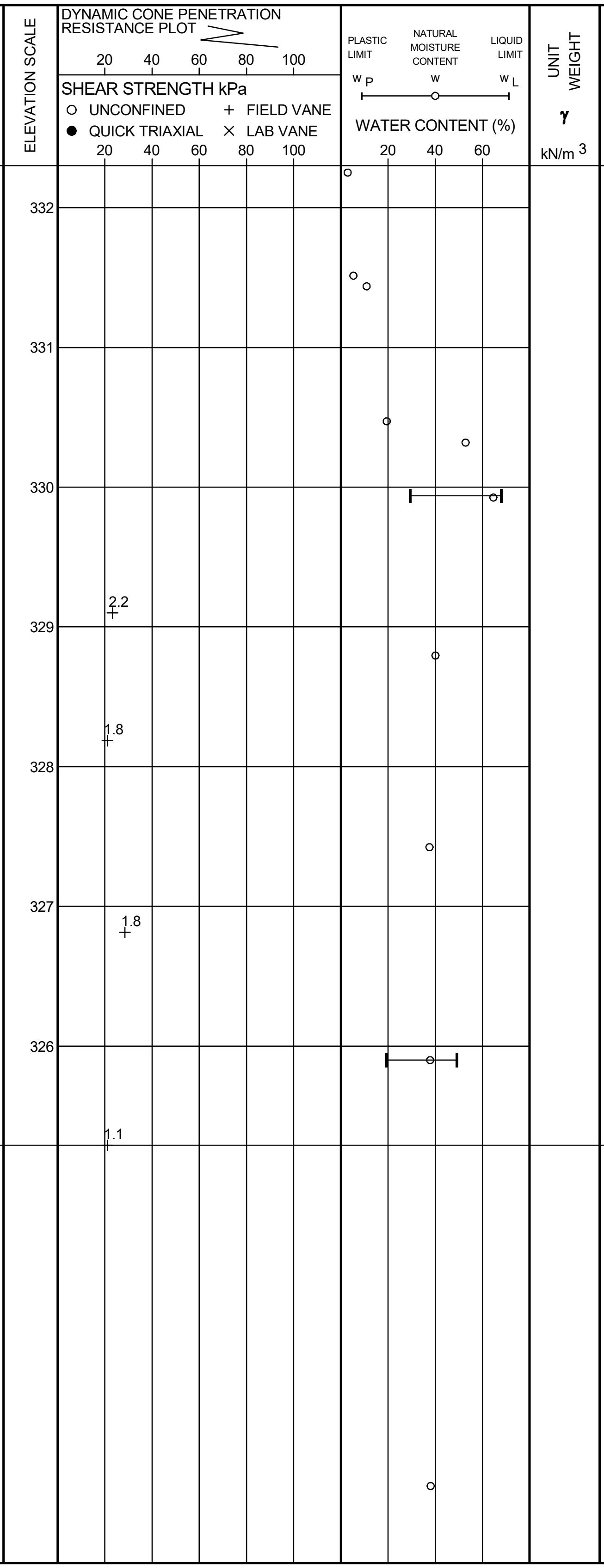
THURBER

RECORD OF BOREHOLE No 17-09R

1 OF 1 **METRIC**

W.P. ______
HWY ______
DATUM Geodetic
LOCATION Pinewood River Bridge N 5 405 212.3 E 211 220.6
BOREHOLE TYPE Hollow Stem Augers
DATE 2017.06.07 - 2017.06.07
ORIGINATED BY SMP
COMPILED BY AN
CHECKED BY CZ

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
332.3	GROUND SURFACE						
0.0	Silty **SAND**, some gravel Loose to Very Loose Brown Moist (FILL)	⊠	1	SS	6		
			2	SS	7		11 74 10 5
330.5 1.8	Silty **CLAY** Soft to Firm Grey Moist	▨	3	SS	3		
			4	SS	3		
			5	SS	WH		
			6	SS	WH		
			7	SS	WH		
325.3 7.0	END OF BOREHOLE AT 7.0m. BOREHOLE OPEN TO 4.0m AND DRY. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.						



+3, ×3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 17-10

1 OF 4

METRIC

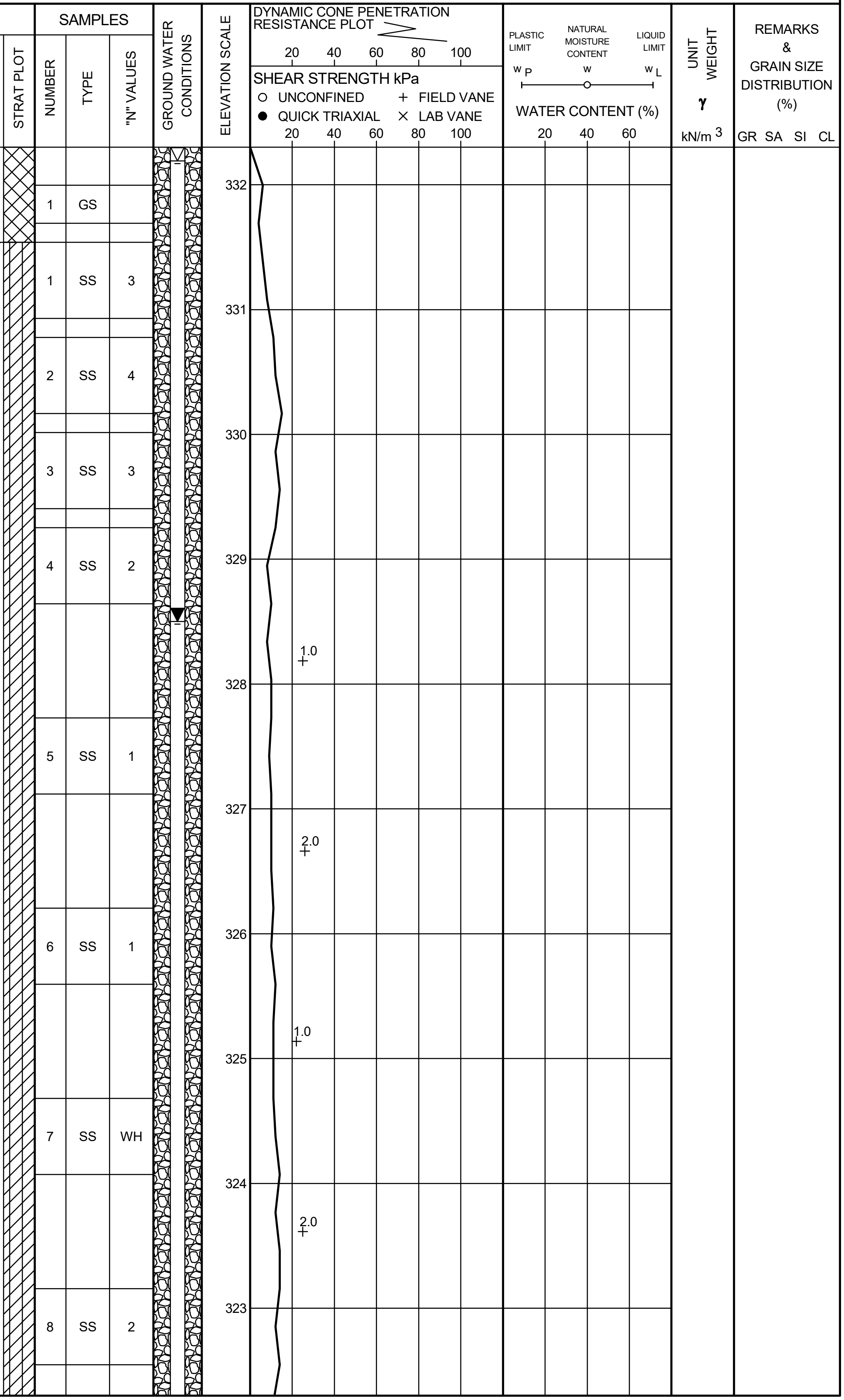
W.P.		LOCATION	Pinewood River Bridge N 5 405 221.9 E 211 222.1	ORIGINATED BY	AHF
HWY		BOREHOLE TYPE	Hollow Stem Augers/DCPT	COMPILED BY	AN
DATUM	Geodetic	DATE	2017.04.21 - 2017.04.21	CHECKED BY	CZ

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
332.3	GROUND SURFACE
0.0	Silty **SAND**, sod, trace roots and rootlets Brown Moist (FILL)
331.5 0.8	Start DCPT from surface
	Silty **CLAY**, trace sand Soft to Firm Grey Moist

SAMPLES

NUMBER	TYPE	"N" VALUES
1	GS	
1	SS	3
2	SS	4
3	SS	3
4	SS	2
5	SS	1
6	SS	1
7	SS	WH
8	SS	2



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+ 3 , × 3 : Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 17-10

2 OF 4

METRIC

W.P. ______

LOCATION Pinewood River Bridge N 5 405 221.9 E 211 222.1

ORIGINATED BY AHF

HWY ______

BOREHOLE TYPE Hollow Stem Augers/DCPT

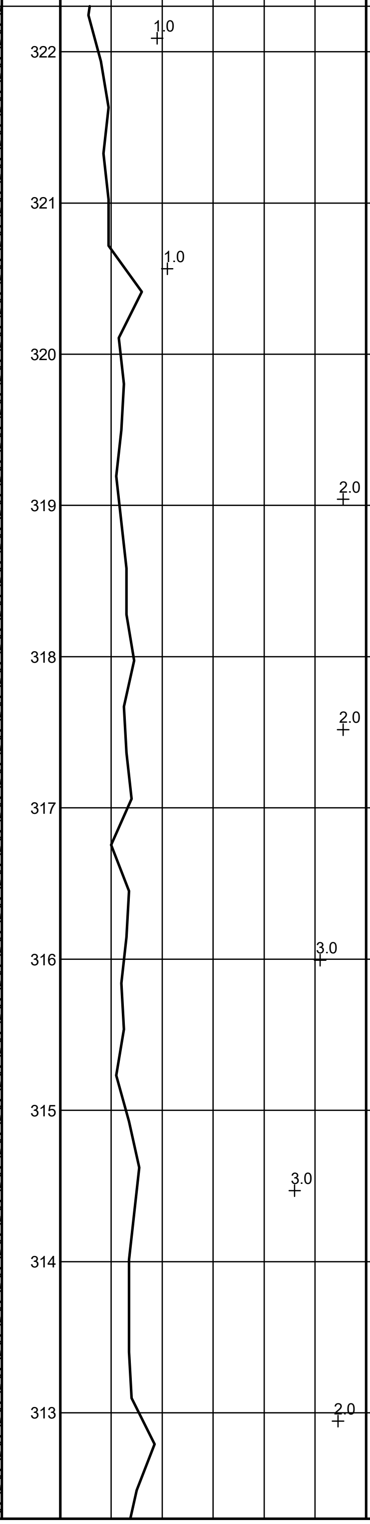
COMPILED BY AN

DATUM Geodetic

DATE 2017.04.21 - 2017.04.21

CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT wP / NATURAL MOISTURE CONTENT w / LIQUID LIMIT wL / WATER CONTENT (%)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Silty **CLAY** Firm to Stiff Grey Wet						322				
			9	SS	1		321				
			10	SS	1		320				
							319				
			11	SS	1		318				
			12	SS	1		317				
							316				
			13	SS	5		315				
							314				
			14	SS	4		313				



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+ 3, × 3: Numbers refer to Sensitivity

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

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Ontario

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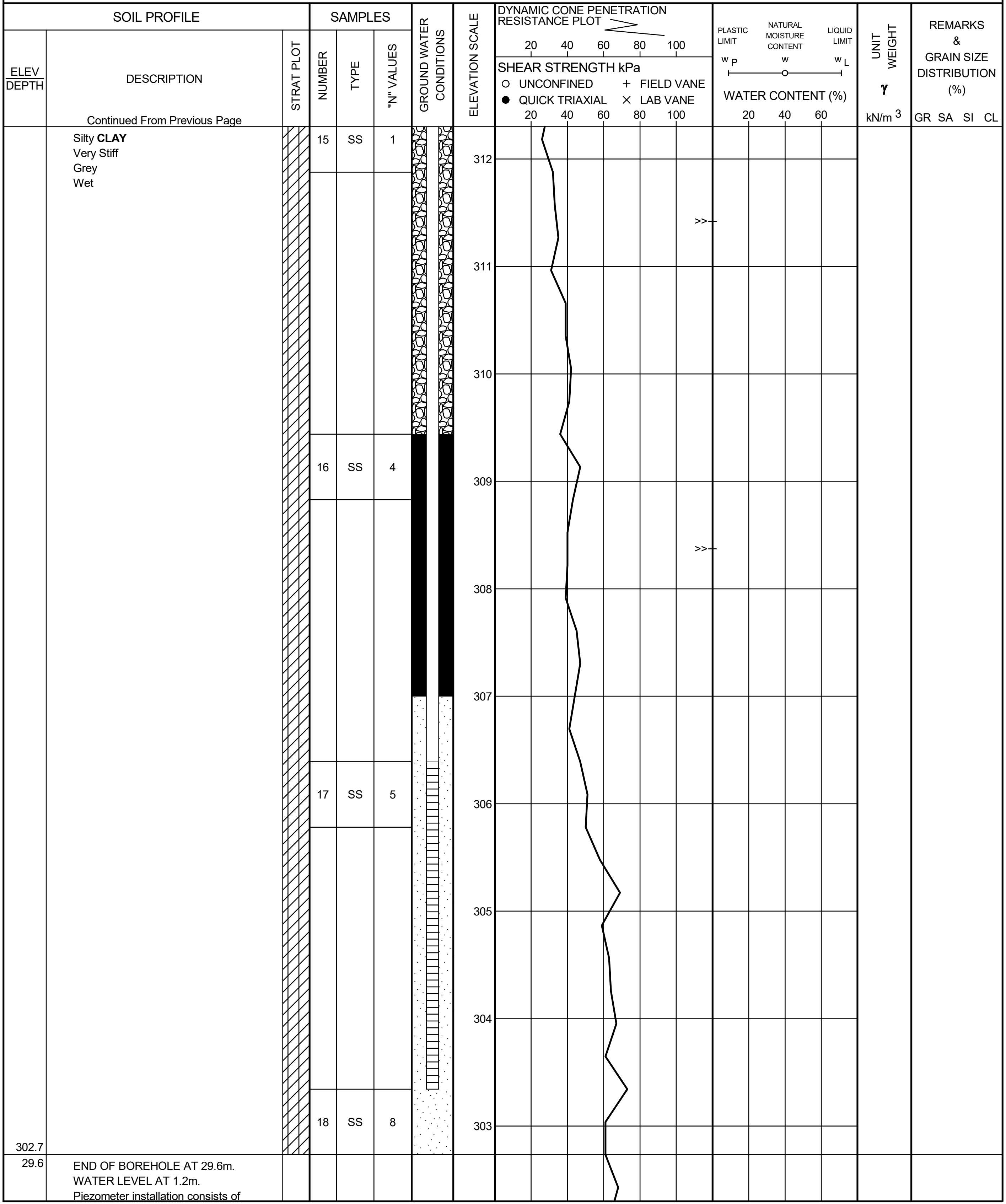
RECORD OF BOREHOLE No 17-10

3 OF 4

METRIC

W.P. ____
LOCATION Pinewood River Bridge N 5 405 221.9 E 211 222.1
ORIGINATED BY AHF
HWY ____
BOREHOLE TYPE Hollow Stem Augers/DCPT
COMPILED BY AN
DATUM Geodetic
DATE 2017.04.21 - 2017.04.21
CHECKED BY CZ



Continued From Previous Page

Silty **CLAY**
Very Stiff
Grey
Wet

NUMBER	TYPE	"N" VALUES
15	SS	1
16	SS	4
17	SS	5
18	SS	8

302.7
29.6

END OF BOREHOLE AT 29.6m.
WATER LEVEL AT 1.2m.
Piezometer installation consists of

Continued Next Page

+ 3 , × 3 : Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 17-10

4 OF 4

METRIC

W.P. ______ LOCATION Pinewood River Bridge N 5 405 221.9 E 211 222.1 ORIGINATED BY AHF
HWY ______ BOREHOLE TYPE Hollow Stem Augers/DCPT COMPILED BY AN
DATUM Geodetic DATE 2017.04.21 - 2017.04.21 CHECKED BY CZ

SOIL PROFILE			SAMPLES					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 Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. END OF DCPT AT 30.5m. WATER LEVEL READINGS DATE DEPTH(m) ELEV.(m) 2017.04.22 3.8 328.5										

+ 3 , × 3 : Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 17-10R

1 OF 2

METRIC

W.P.

HWY

DATUM Geodetic

LOCATION Pinewood River Bridge N 5 405 222.2 E 211 217.1

BOREHOLE TYPE Hollow Stem Augers

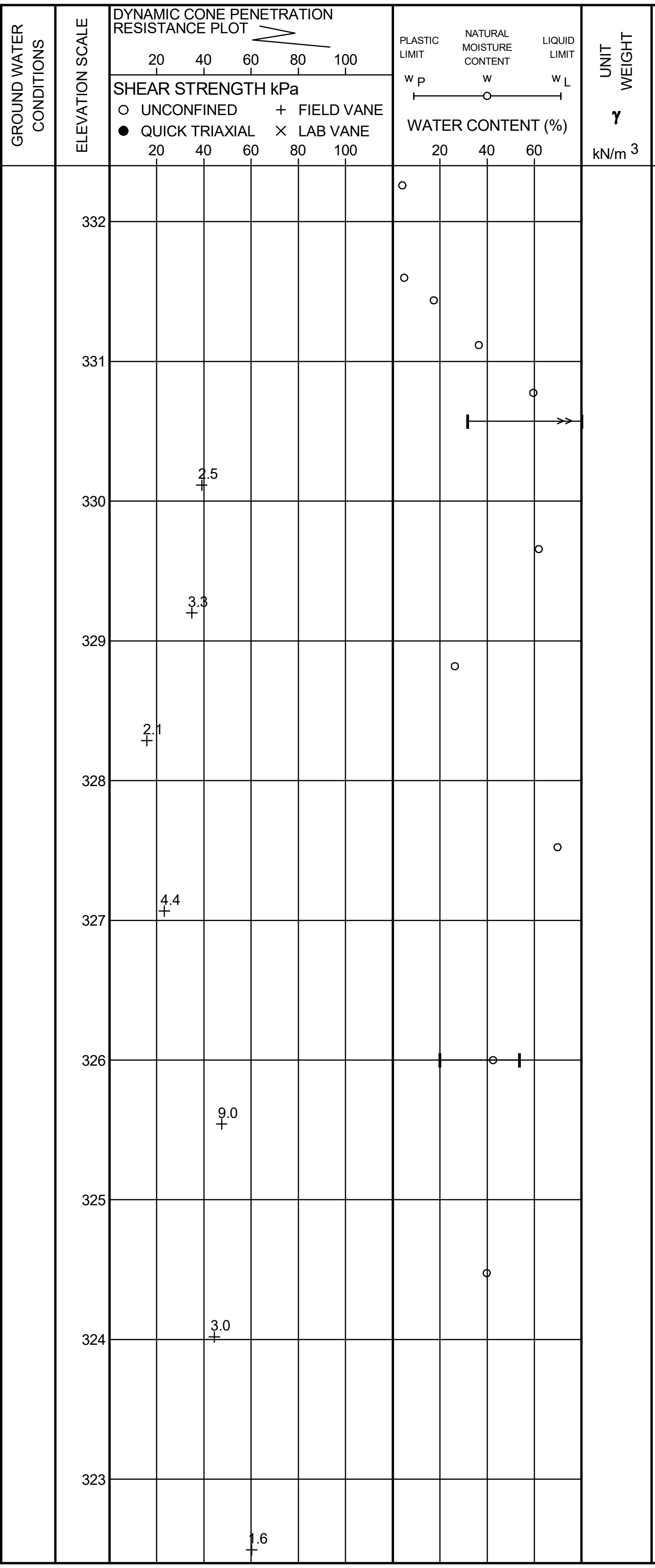
DATE 2017.06.06 - 2017.06.06

ORIGINATED BY SMP

COMPILED BY AN

CHECKED BY CZ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
332.4	GROUND SURFACE							
0.0	Silty **SAND**, some gravel to gravelly, trace organics Compact Brown Moist (FILL)	1	SS	13	23	59	18 (SI+CL)	
		2	SS	4				
331.3 1.1	Silty **CLAY** Soft to Stiff Grey Moist	3	SS	4				
		4	SS	2				
		5	SS	WH				
		6	SS	WH				
		7	SS	WH				
		8	SS	WH				
		9	SS	WH				



Continued Next Page

+ 3 , × 3 : Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-10R

2 OF 2

METRIC

W.P.
LOCATION Pinewood River Bridge N 5 405 222.2 E 211 217.1
ORIGINATED BY SMP
HWY
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY AN
DATUM Geodetic
DATE 2017.06.06 - 2017.06.06
CHECKED BY CZ

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										
322.3 10.1	END OF BOREHOLE AT 10.1m. BOREHOLE OPEN TO 9.0m AND DRY. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.										

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

20
15-⊕-5 (%) STRAIN AT FAILURE
10

Appendix B

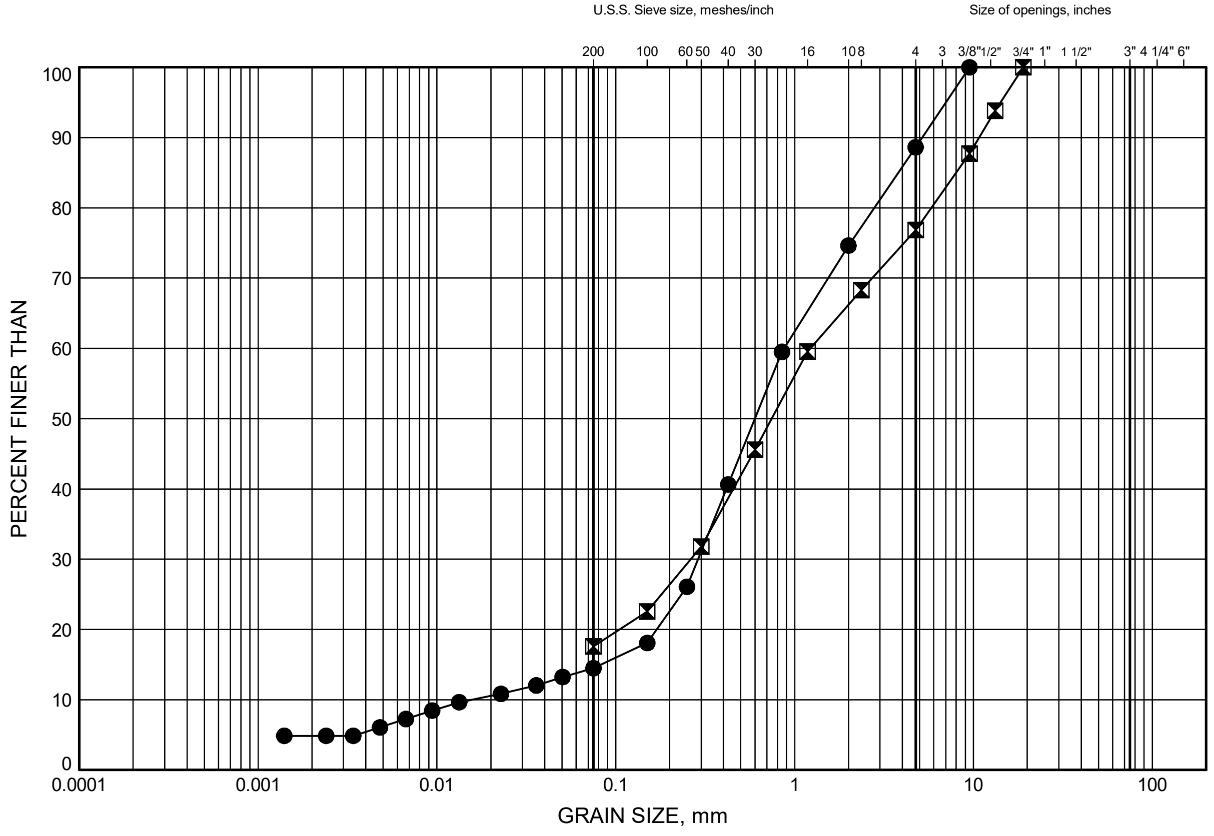
Geotechnical and Analytical Laboratory Test Results

Pinewood River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B1

Silty Sand Fill



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-09R	0.8	331.5
☒	17-10R	0.1	332.3

GRAIN SIZE DISTRIBUTION - THURBER MTO-17792.GPJ 7/6/17

Date July 2017

W.P.

THURBER

Prep'd MFA

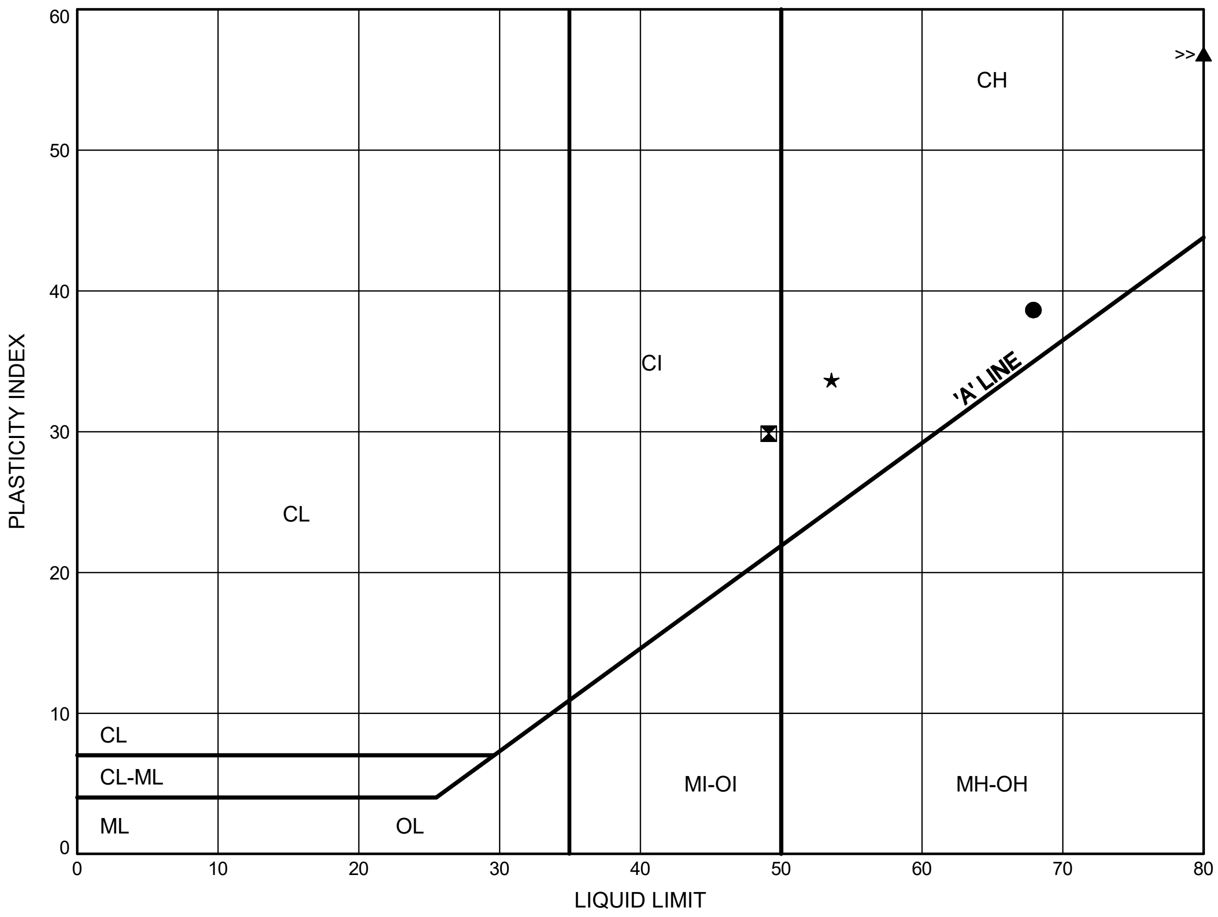
Chkd. MM

Pinewood River Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE B2

Silty Clay to Clay



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-09R	2.4	329.9
☒	17-09R	6.4	325.9
▲	17-10R	1.8	330.6
★	17-10R	6.4	326.0

Date July 2017

W.P.

THURBER

Prep'd MFA

Chkd. MM

Certificate of Analysis

SGS Canada Inc.
185 Concession St. Box 4300
Lakefield, Ont.,Canada,K0L 2H0

Attention: Cory Zanatta
Project#: 17792

Client	**Thurber Engineering**
SGS LIMS Number	**CA14503-JUN17**
Analysis Package:	**Corrosivity**

Sample ID	**Unit**	**Analysis Start Date**	**Analysis Approval Date**	**17-09R SS6**	**17-07R SS3**	**17-06R SS4**
Sample Date/Time				07-Jun-17	05-Jun-17	06-Jun-17
Temperature Upon Receipt	°C			4.0	4.0	4.0
Corrosivity Index	none	01-Jun-17	01-Jun-17	8.5	4.0	4.5
Soil Redox Potential	mV	29-May-17	30-May-17	12	233	285
Sulphide	%	01-Jun-17	01-Jun-17	0.03	<0.02	0.02
% Moisture (wet wt)	%	30-May-17	01-Jun-17	26.0	16.8	13.30
pH	no unit	30-May-17	31-May-17	8.48	7.08	7.89
Chloride	µg/g	31-May-17	01-Jun-17	1.9	8.0	40
Sulphate	µg/g	31-May-17	01-Jun-17	51	11	26
Conductivity	uS/cm	30-May-17	31-May-17	157	43	89
Resistivity (calculated)	ohms.cm	30-May-17	01-Jun-17	6370	23300	11200

Corrosivity Index is based on the AWWA Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

Deanna Edwards

Deanna Edwards B.Sc., C.Chem
Project Specialist
Environment, Health and Safety

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

Project : 17840/17792

16-May-2017

Thurber Engineering Ltd
Attn : Cory Zanatta

2010 Winston Park Dr
Oakville, ON
L6H 5R7,

Phone: 905-829-8666 x 240
Fax:

Date Rec. : 10 May 2017
LR Report: **CA14294-MAY17**
Reference: 17840/17792 Cory Zanatta

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Temperature Upon Receipt °C	pH no unit	Conductivity µS/cm	Resistivity (calculated) ohms.cm	Redox Potential mV	Chloride mg/L	Sulphate mg/L	Sulphide mg/L
1: Analysis Start Date		---	11-May-17	11-May-17	---	11-May-17	15-May-17	15-May-17	11-May-17
2: Analysis Start Time		---	10:30	10:41	---	13:57	18:20	18:20	12:10
3: Analysis Approval Date		--	15-May-17	15-May-17	---	15-May-17	16-May-17	16-May-17	12-May-17
4: Analysis Approval Time		--	10:54	10:51	---	10:32	13:24	13:24	16:01
5: MDL		---	0.05	2	---	---	0.04	0.04	0.006
6: Rossmere Creek	25-Apr-17	9.0	6.35	115	8700	197	24	1.1	0.014
7: Two Island Lake	25-Apr-17	9.0	6.42	35	28700	218	2.0	2.0	< 0.006
8: Wawing Creek	25-Apr-17	9.0	6.30	47	21200	221	5.8	1.8	0.009
9: Hawkeye Lake	25-Apr-17	9.0	6.71	40	25000	213	1.4	1.9	< 0.006
10: Pinewood River	25-Apr-17	9.0	6.89	78	12900	207	0.68	0.43	0.018

```
Temperature of Sample upon Receipt: 9 degrees C
Cooling Agent Present: yes
Custody Seal  Present: no
```

Brian Graham B.Sc.
Project Specialist
Environmental Services, Analytical

OnLine LIMS

0001000062

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
Anions by IC	ME-CA-[ENV]IC-LAK-AN-001	EPA300/MA300-Ions1.3
Conductivity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2510
pH	ME-CA-[ENV]EWL-LAK-AN-006	SM 4500
Redox Potential		SM 2580
Sulphide by SFA	ME-CA-[ENV]SFA-LAK-AN-008	SM 4500

Quality Control Report

Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank				LCS / Spike Blank			Matrix Spike / Reference Material		
					RPD	Acceptance Criteria	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
						%		Low	High		Low	High
Anions by IC - QCBatchID: DIO0256-MAY17												
Chloride	0.04	mg/L	<0.04		2	20	97	80	120	100	75	125
Sulphate	0.04	mg/L	<0.04		0	20	96	80	120	89	75	125
Anions by IC - QCBatchID: DIO0269-MAY17												
Chloride	0.04	mg/L	<0.04		0	20	100	80	120	119	75	125
Sulphate	0.04	mg/L	<0.04		0	20	97	80	120	102	75	125
Conductivity - QCBatchID: EWL0183-MAY17												
Conductivity	2	µS/cm	< 2		0	10	99	90	110	NA		
pH - QCBatchID: EWL0182-MAY17												
pH	0.05	no unit	NA		1		100			NA		
Redox Potential - QCBatchID: EWL0192-MAY17												
Redox Potential	no	mV	NA		0	20	103	80	120	NA		
Sulphide by SFA - QCBatchID: SKA0095-MAY17												
Sulphide	0.006	mg/L	<0.006		ND	20	80	80	120	NV	75	125
Sulphide by SFA - QCBatchID: SKA0105-MAY17												
Sulphide	0.006	mg/L	0.009		ND	20	96	80	120	125	75	125

Appendix C

Selected Site Photographs

Photograph 1 – Pinewood River Bridge – North Abutment - Looking South

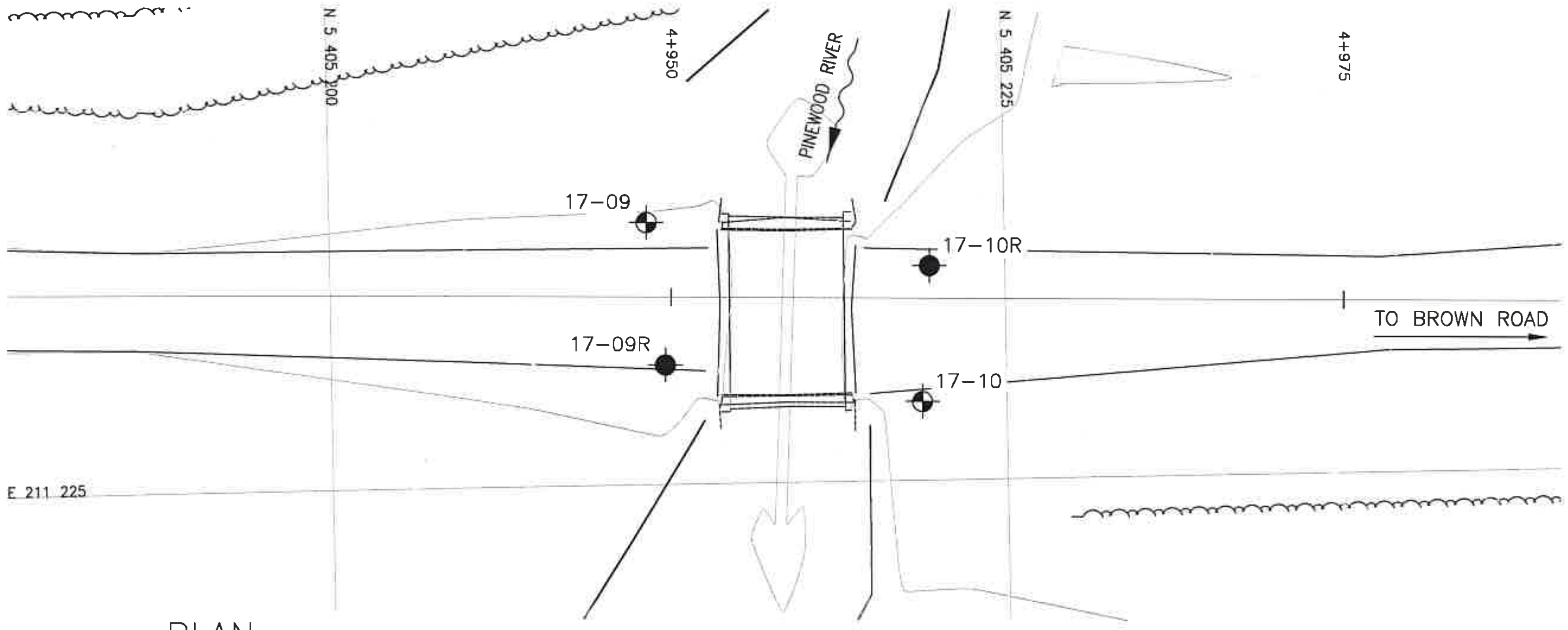
Photograph 2 – Pinwood River Bridge - South Abutment – Looking North

Photograph 3 – Pinwood River Bridge – West End - Inlet

Appendix D

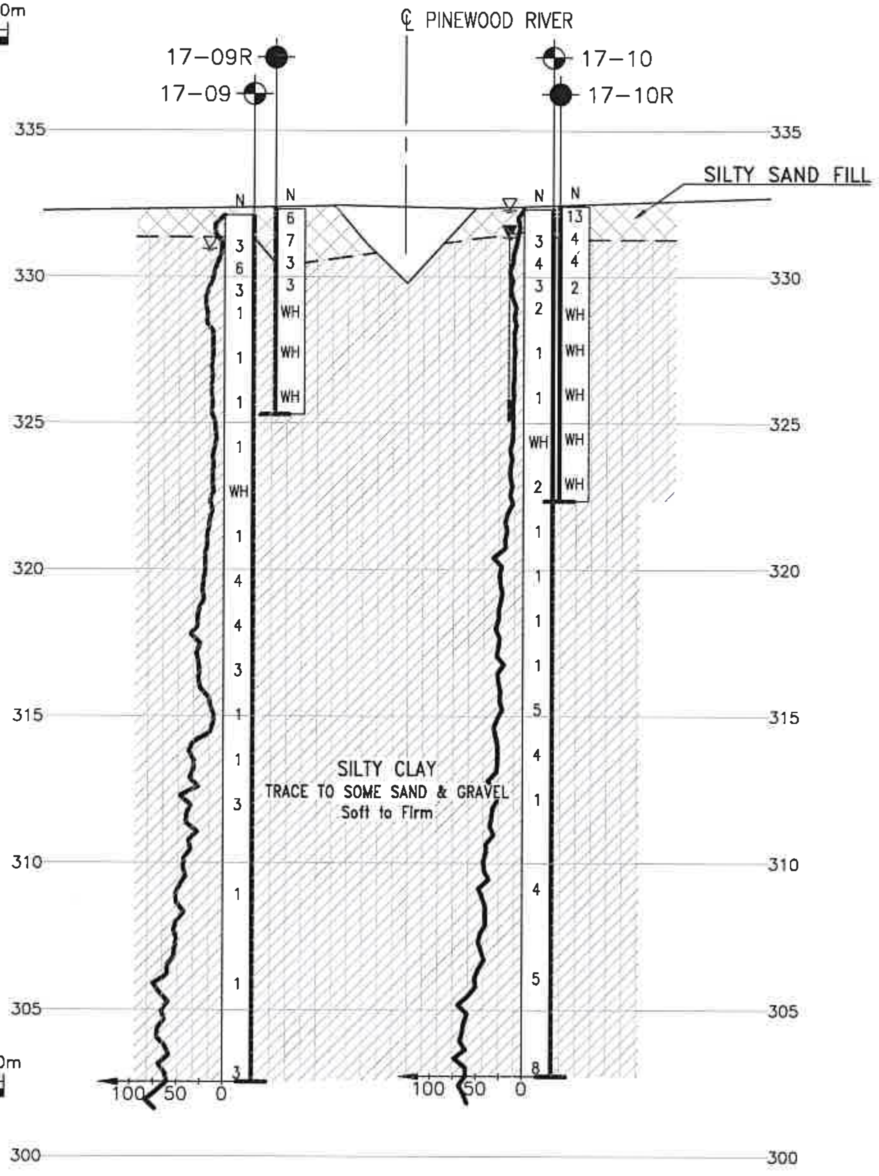
Borehole Locations and Soil Strata Drawing

MINISTRY OF TRANSPORTATION, ONTARIO



PLAN

5 0 5 10m
SCALE 1:250



PROFILE

5 0 5 10m
SCALE 1:250

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

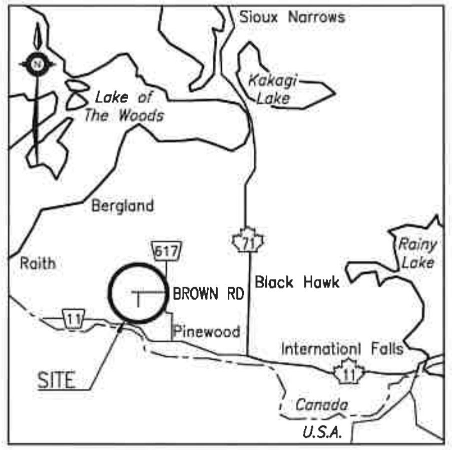
CONT No
WP No

SHEET

PINEWOOD RIVER
BRIDGE #1
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

Ontario

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

●	Borehole
⊕	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
▼	Head Artesian Water
│	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
17-09	332.1	5 405 211.7	211 215.4
17-09R	332.3	5 405 212.3	211 220.6
17-10	332.3	5 405 221.9	211 222.1
17-10R	332.4	5 405 222.2	211 217.1

Oct.13/17
M. MOSTAKHDEMI
100185447

P.K. CHATTERJI
Oct 13/17

-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. 52D-31

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN CZ	CHK PKC	CODE	LOAD	DATE OCT 2017	
DRAWN MFA	CHK CZ	SITE	STRUCT	DWG 1	

FILENAME: H:\Drafting\17000\17792\TED-17792-BHPP-PR.dwg
PLOTDATE: 10/13/2017 9:02 AM

Appendix E

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Compacted Granular Pad	**Steel Driven Friction Piles**
Advantages: i. Ease of construction. ii. Cost Effective	Advantages: i. Requires less excavation and groundwater control
Disadvantages: i. Requires excavation and groundwater control	Disadvantages: i. Likely more expensive than footings ii. Doesn't provide high axial geotechnical resistances due to presence of think weak soils and deep bedrock at the site iii. Chance of pile miss-alignment during pile driving
FEASIBLE - PREFERRED	**FEASIBLE**

Appendix F

List of Specifications and Suggested Wording for NSSP

1. List of OPSS and OPSD Documents Relevant to this Project

- OPSS PROV 206
- OPSS PROV 209
- OPSS.PROV 421
- OPSS PROV 422
- OPSS PROV 501
- OPSS.PROV 517
- OPSS PROV 539
- OPSS PROV 804
- OPSS PROV 902
- OPSS PROV 1004
- OPSS PROV 1010
- OPSS PROV 1205
- OPSS 511
- OPSS 1860
- OPSD 802.010
- OPSD 802.014
- OPSD 803.010
- OPSD 803.031
- OPSD 810.010
- OPSD 3090.100

2. Suggested Wording for NSSP on Dewatering

"The Contractor is notified that the site has high groundwater levels and that these levels may be higher than the water levels shown in the Foundation Investigation Report prepared for this site. While reference should be made to that report for a description of the encountered conditions, the Contractor must satisfy himself regarding the groundwater levels likely to prevail at the time of construction and be prepared to implement dewatering procedures.

The Contractor is further notified that failure to implement dewatering in advance of excavating below the groundwater table may result in sloughing and boiling of the soil in the excavation and a loss in stability and bearing resistance.

Design and provision of an effective dewatering system is the responsibility of the Contractor. Subgrade preparation, culvert construction and backfilling must be carried out in the dry."