



**THURBER** ENGINEERING LTD.



**FOUNDATION INVESTIGATION AND DESIGN REPORT  
WHITEWOOD CREEK CULVERTS REPLACEMENT  
TOWNSHIP OF O'CONNOR, DISTRICT OF THUNDER BAY, ONTARIO  
SITE No. 48W-311/C  
HIGHWAY 595**

**ASSIGNMENT NO. 6015-E-0023**

**GEOCRES Number: 52A-228  
W.O.# 2017-11029**

**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) for the proposed replacement of the Whitewood Creek twin Culvert on Highway 595, located in the Township of O'Connor, District of Thunder Bay, Ontario.

The purpose of this investigation was to explore the subsurface conditions in the twin culvert area and, based on the data obtained, 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 the Ministry of Transportation Ontario (MTO) to carry out this foundation investigation under the MTO Retainer Agreement Number 6015-E-0023.

**2. SITE DESCRIPTION**

The Whitewood Creek Culverts site is located on Highway 595, in the Township of O'Connor approximately 4.3 km south of Highway 590, and approximately 8 km north of Highway 588, in the District of Thunder Bay, Ontario. The key plan showing the general location of the site is presented on the Borehole Locations and Soil Strata drawing in Appendix D.

Highway 595 runs in the general north-south direction with the culverts perpendicular to the centreline of the highway. The Whitewood Creek is a tributary of the Kaministiquia River and the stream flowing through the creek is from west to east at the site.

The terrain in the culvert area is gently undulating and forested outside of the right-of-way. The

existing culvert is a twin Corrugated Steel Pipe (CSP) culvert approximately 35 m in length. The Structural Inspection Report (SIR) prepared by McCormick Rankin, a member of MMM Group and dated January 2014 indicated that the culverts are in fair to good conditions apart from accumulation of debris build-up at the north and south outlets and minor damages at the edges of both outlets due to past debris removal activities.

The invert elevation of the existing culverts was indicated at about Elevation 278.0 m at the inlet and about Elevation 277.8 m at the outlet. The stream water level was reported to be at about Elevation 278.5 m at the upstream and about Elevation 277.6 m at the downstream on August 2015. At the culvert location, the highway embankment grade is at approximately Elevation 282.0 m. The depth of cover over the existing culvert is approximately 1.8 m to 2.0 m.

Photographs in Appendix C show the general nature of the site and the existing culvert.

Based on published geological information, the culvert lies close to the border of glaciolacustrine plain (including deposits of silts and clays with minor sands) and silty clay to silt deposit of glacial tills. The bedrock at the site consists of rocks of Gunflint Formation.

### **3. INVESTIGATION PROCEDURES**

The field investigation and testing program for this project was specified in the Terms of Reference. The field work was carried out on April 6, 7, 12 and 30, 2017 during which time four (4) boreholes designated as Boreholes 17-01 to 17-04 were advanced at the site. Boreholes 17-01 and 17-04 were advanced adjacent to the inlet and outlet of the culverts and Boreholes 17-02 and 17-03 were advanced through the highway embankment just south and north of the culverts, respectively.

Utility clearances were obtained prior to the start of drilling. A rubber tire buggy mounted drill rig and a track-mounted CME 75 drill rig were used to advance the boreholes at the site using hollow stem augers. An NQ core barrel was used to obtain 3 m of rock core in Boreholes 17-02 and 17-03.

Soil samples were obtained at selected intervals with a 50 mm outside diameter split spoon sampler driven in conjunction with the Standard Penetration Test (SPT) procedures as per ASTM D1586. Dynamic Cone Penetration Test (DCPT) was also conducted adjacent to Boreholes 17-01 and 17-02 from the ground surface to refusal. Where bedrock was cored, rock quality (i.e., TCR, SCR, RQD, weathering and strength indices), discontinuity characteristics and classification

data were recorded in the field based on visual inspection of the recovered rock cores upon extraction from the core barrel. The bedrock was sequentially photographed and selected samples were properly packed and transported to our laboratory for strength testing (point load index).

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

Groundwater conditions were observed in the open boreholes throughout the drilling operations. One standpipe piezometer using 19 mm diameter PVC pipes was installed within the overburden in Borehole 17-01 to permit monitoring of the groundwater levels at the site. The piezometer was decommissioned and the borehole was back filled on April 30, 2017. All other boreholes were backfilled on completion of drilling in general accordance with Ontario Regulation 903, as amended.

The coordinates and ground surface elevations for the boreholes were derived from topographic plans provided by the MTO. The coordinate system MTM NAD 83, Zone 14 was used for the boreholes. The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing included in Appendix D. The borehole coordinates, ground surface elevations, drilled depths and the completion details are summarized in Table 3.1.

**Table 3.1 – Borehole Completion Details**

Borehole Number	Coordinates (MTM NAD 83, Zone 14)		Ground Surface Elevation (m)	Borehole Depth (m)	Completion Details
	Northing (m)	Easting (m)			
17-01	5,357,870.4	326,991.7	279.5	9.8	Standpipe piezometer was installed in the borehole. After removal of the piezometer, the borehole was backfilled with bentonite holeplug and cuttings to ground surface.
17-02	5,357,859.8	327,005.1	282.5	16.2	Bentonite holeplug to 1.8 m, cuttings to 0.3 m then asphalt cold patch to ground surface.
17-03	5,357,869.7	327,009.5	281.8	15.7	Bentonite holeplug to 1.6 m, cuttings to 0.1 m then asphalt cold patch to ground surface.

Borehole Number	Coordinates (MTM NAD 83, Zone 14)		Ground Surface Elevation (m)	Borehole Depth (m)	Completion Details
	Northing (m)	Easting (m)			
17-04	5,357,857.8	327,020.0	278.5	11.0	Bentonite holeplug and cuttings to ground surface.

#### 4. LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. Selected soil samples were also subjected to grain size distribution analyses (sieve and/or hydrometer). Selected bedrock core specimen were also subjected to point load strength index test. The results of the laboratory testing program are shown on the Record of Borehole sheets included in Appendix A and on the figures included in Appendix B.

In order to assess the potential for sulphate attack on concrete foundations, as well as the potential for corrosion associated with the structure, two samples of the native soils near the invert level, and a sample of the surface water from the creek upstream of the existing culverts 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 of this report and are presented in Appendix B.

#### 5. DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil stratigraphy are presented on the Record of Borehole 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 must be used for interpretation of the site conditions. It must be recognized and expected that subsurface conditions may vary between and beyond the borehole locations.

In general, the subsurface conditions encountered in the boreholes consisted of embankment fill comprising of sand and gravel, sand, and/or pockets of silty clay overlying a native silt deposit which is in turn underlain by a deposit of silty clay. The silty clay is underlain by a relatively thin layer of cobbles (or fractured bedrock) over the bedrock. Descriptions of the individual strata are

presented below.

### **5.1 Asphalt**

Boreholes 17-02 and 17-03 were drilled through the existing asphalt pavement on Highway 595. Asphalt thicknesses of 40 mm and 25 mm were measured at the two borehole locations, respectively. The thickness of asphalt may vary along the highway.

### **5.2 Topsoil**

An approximately 50 mm thick layer of topsoil was encountered at the ground surface in Borehole 17-01. The topsoil thickness may vary in other areas of the site.

### **5.3 Fill**

Fill was encountered below the topsoil in Borehole 17-01, below the asphalt in Boreholes 17-02 and 17-03 and from the ground surface in Borehole 17-04. The fill generally consisted of sand and gravel to gravelly sand at the borehole locations. Approximately 1 m thick layer of the clayey silt fill and 1.5 m thick layer of sand fill were encountered below the sand and gravel fill in Borehole 17-02. The fill was 2.2 m and 1.5 m thick in Boreholes 17-01 and 17-04 which were drilled at the inlet and outlet of the culverts. The fill was about 5.6 m and 4.7 m in Boreholes 17-02 and 17-03 which were drilled on Highway 595.

The relative density of the non-cohesive fill below the highway embankment was compact to very dense with the SPT-N values recorded from 19 blows per 0.3 m of penetration to 50 blows per 0.15 m of penetration. A single SPT-N value of 7 blows per 0.3 m of penetration was measured in the clayey silt fill indicating a firm consistency.

The relative density of the sand and gravel fill adjacent to the inlet and outlet of the culverts encountered in Boreholes 17-01 and 17-04 is loose to compact with SPT-N values recorded between 4 blows and 16 blows per 0.3 m of penetration.

The results of grain size analyses conducted on selected samples of the fill are presented on the Record of Borehole sheets included in Appendix A, and on Figure B1 in Appendix B.

The results are summarized in the following table:



Soil Particle	Percentage (%)	
	Sand and Gravel	Gravelly Sand
Gravel	37 and 39	30
Sand	45 and 49	62
Silt and Clay	12 and 18	8

The measured moisture content of the fill materials ranged between 3% and 24%.

#### 5.4 Silt/Clayey Silt

A deposit of grey silt with some clay to clayey silt was encountered underlying the fill in all boreholes advanced at the site. The silt extended to depths of 2.2 m to 8.7 m below existing ground surface (base elevation ranging from 273.8 m to 276.3 m). The deposit was compact to very dense, or stiff to hard, as indicated by SPT 'N' values between 12 and 51 blows per 0.3 m of penetration. The measured moisture content of the silt ranged between 18% and 22%.

The results of grain size analysis and one Atterberg Limits test conducted on a sample of silt/clayey silt deposit are presented on the Record of Borehole sheets included in Appendix A and on Figures B2 and B3 in Appendix B.

The results are summarized in the following table:

Soil Particle	Percentage (%)
Gravel	0
Sand	0
Silt	78 to 85
Clay Size Fines	15 to 22
Measured Limit	Percentage (%)
Liquid Limit	25.6
Plastic Limit	18.7

#### 5.5 Silty Clay

A deposit of grey silty clay was encountered below the silt layer in all four boreholes extending to depths of 12.6 m and 12.3 m (Elevations 269.9 m and 269.5 m) in Boreholes 17-02 and 17-03, respectively. Boreholes 17-01 and 17-04 were terminated in the silty clay.

SPT 'N' values within the silty clay ranged between 26 blows per 0.3 m of penetration and 100 blows per 0.125 m of penetration indicating a very stiff to hard consistency. The measured moisture content of the silty clay ranged between 12% and 31%.

The results of grain size distribution analyses and Atterberg Limits tests conducted on selected samples of the silty clay are presented on the Record of Borehole sheet included in Appendix A and on Figures B4 and B5 in Appendix B. The results are summarized in the following table:

Soil Particle	Percentage (%)
Gravel	0
Sand	0
Silt	50 to 64
Clay Size Fines	36 to 50
<b>Measured Limit</b>	<b>Percentage (%)</b>
Liquid Limit	33 and 42
Plastic Limit	20

The results of the Atterberg Limits testing indicate that the silty clay has a low to intermediate plasticity with group symbol CL to CI.

## 5.6 Cobbles/Fractured Bedrock

Cobbles/fractured bedrock with a thickness of 0.3 m to 0.6 m was encountered below the silty clay at depths of 12.6 m and 12.3 m in Boreholes 17-02 and 17-03, respectively. This layer was encountered just above the bedrock.

## 5.7 Bedrock

Borehole 17-04 was terminated at a depth of 11.0 m (Elevation 267.5 m) due to refusal to further auger penetration on probable bedrock.

Bedrock was encountered at depths of approximately 13.2 m and 12.6 m (Elev. 269.3 m and Elev. 269.2 m) in Boreholes 17-02 and 17-03, respectively. The bedrock was proven in Boreholes 17-02 and 17-03 by coring approximately 3 m in both boreholes. The bedrock is generally described as grey diabase, greenish grey in colour. Total Core Recovery (TCR) in the bedrock ranged from 60% to 100% with Solid Core Recovery (SCR) ranging from 49% to 100%. The Rock Quality

Designation (RQD) determined from the recovered cores generally ranged from 40% to 100%, indicating a poor to excellent rock quality.

Point load strength index tests (PLT) were carried out on selected core samples. The axial and diametral point load strength index values (Is50) are presented in Appendix B. The interpreted average UCS values for each core run of the bedrock ranged between 94 MPa and 162 MPa based on correlations with the PLT, indicating the bedrock at the site is strong to very strong.

## 5.8 Groundwater Conditions

A standpipe piezometer was installed in Borehole 17-01 on April 12, 2017 and a groundwater level reading was taken in the piezometer on April 30, 2017. The groundwater levels measured in the open borehole and in the piezometer are summarized in Table 5.1 below.

**Table 5.1 – Groundwater Measurements**

Borehole	Date	Piezometer Installation		Water Level (m)		Remark
		Screen Depth/Elevation (m)	Screened Deposit	Depth	Elevation	
17-01	April 12, 2017	6.7 to 9.8 / 272.8 to 269.8	Silty Clay	1.9	277.6	Open borehole
	April 30, 2017			2.2	227.3	Piezometer

The groundwater level should be assumed to reflect the local creek water level. Water level measurements in the creek were reported on the MTO Site Plan Drawing, E-1078-595-4, which reported measurements of Elevation 278.5 m at the inlet and 277.6 m at the outlet in August 2015. The above groundwater levels are 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 during spring and after periods of significant or prolonged precipitation.

## 6. CORROSIVITY AND SULPHATE TEST RESULTS

Two samples of the native soil from Boreholes 17-02 and 17-03, and a sample of the surface water from the creek were submitted for analytical testing of corrosivity parameters and sulphate content. 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		
			17-02, SS#6, 6.1 m – 6.7 m	17-03, SS#5, 4.6 m – 5.2 m	Whitewood Creek
			(Silt)	(Silt)	(Creek Water)
Sulphide	%	mg/L	0.05	<0.02	<0.006
Chloride	µg/g	mg/L	55	25	7.3
Sulphate	µg/g	mg/L	110	61	3.2
pH	No unit	No unit	8.68	8.55	7.46
Electrical Conductivity	µS/cm	µS/cm	157	109	129
Resistivity	Ohms.cm	Ohms.cm	6,370	9,170	7,750
Redox Potential	mV	mV	200	237	295

## 7. MISCELLANEOUS

Thurber obtained the coordinates and ground surface elevations of the boreholes from measurements taken in the field and relative to the topographic plans provided by the MTO.

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 and Ms. Eckie Siu 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, 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 Whitewood Creek Culverts replacement on Highway 595, located in the Township of O'Connor, District of Thunder Bay, 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.

Information on the existing culverts site was obtained from the MTO Terms of Reference, and the Structural Inspection Report (SIR) by McCormick Rankin, a member of MMM Group, and dated January 2014. The SIR indicated that the structures are in fair to good conditions apart from the accumulation of debris build-up at the north and south outlets and minor damages at the edges of both outlets due to past debris removal activities. The SIR also indicated that signs of foundation settlement of the culverts were not noticed at the time of the inspection (July 20, 2013).

The existing structures is a twin Corrugated Steel Pipe (CSP) culvert with a total length of about 35 m and a diameter of about 2.2 m with approximately 2 m of fill above the obvert of the pipes. The Bridge Site Plan prepared by the Geomatics Section of the MTO's Engineering Office indicates that the culvert invert Elevation at the inlets is about 278.0 m, and the invert Elevation

at the outlets is about 277.8 m. The finished highway grade is indicated at about Elevation 282.0 m, which results in approximately 1.8 m to 2.0 m of fill above the culverts.

It is anticipated that the replacement culvert(s) will be constructed at the same location of the existing culverts. It is also understood that the highway grades will remain un-changed adjacent to the creek site, except for possible embankment fill placement adjacent to the extended culvert barrels. A Structural Design Report (SDR) and General Arrangement (GA) drawings which typically indicate the preferred replacement option and its dimensions as well as the location of the diversion pipe (if needed) were not available at the time of preparation of this report.

The discussions and recommendations presented in this report are based on information provided by MTO and on the factual data obtained during the investigation.

In general, the subsurface conditions encountered in the boreholes from the investigations consisted of embankment fill overlying a native deposit of compact to very dense silt (or stiff to hard clayey silt) which is in turn underlain by a very stiff to hard silty clay. A layer of cobbles/boulders and bedrock underlie the clay deposit at the site. The water level in the stream was measured at about Elevations 278.5 m and 277.6 m at the upstream and downstream of the culverts, respectively, in August 2015.

## **8.1 Culvert Design Alternatives**

This section presents discussions on available types of replacement culverts and foundation alternatives, and provides foundation design recommendations.

Three common culvert types that may be considered for the culvert replacement at this site are listed below:

- Concrete box (closed) culverts composed of pre-cast segments
- Corrugated steel pipe (CSP)
- CSP Arch or Concrete, open footing culverts

A comparison of the culvert types and foundation alternatives based on their respective advantages and disadvantages is included in Appendix E. From a foundations and constructability perspective, use of the CSP or pre-cast box culverts are both feasible options, based on the following considerations:

- Pre-cast box culvert or pipe culverts would result in reduced depth of excavation compared to the open footing culvert;

- Pre-cast concrete box or pipe segments can often be installed more expeditiously than cast in place open footing culverts, resulting in shorter durations for dewatering and construction;
- A segmental box (or pipe) structure are more tolerant of differential settlement along the culvert axis if the highway embankment is widened in the future; and
- Due to the relatively high frost penetration depth at the project site (i.e., 2.2 m), open footing culverts would require deeper excavations and consequently more robust temporary shoring and/or groundwater control compare to the other two alternatives.

Recommendations for the design and installation of CSP and concrete box culverts are presented below.

## **8.2 Foundation Design for Culverts**

Foundation design aspects for the replacement culvert includes subgrade conditions and preparation, geotechnical resistances, settlement of founding soils, lateral earth pressures, roadway protection system design, groundwater control, staged construction, and restoration of the roadway embankment.

### **8.2.1 Corrugated Steel Pipe Culvert**

Replacement of the culvert with CSPs on the same alignment is feasible from foundation design and constructability perspectives. In order to accommodate the hydraulic requirements, multiple pipes maybe required. The culvert invert should be at or below Elevation 278 m or the same invert elevation as the existing culverts, which corresponds to the compact to very dense silt/clayey silt or compact granular fill subgrade.

If this option is selected, the CSPs should be placed on a minimum 300 mm thick layer of bedding material conforming to OPSS.PROV 1010 Granular A or Granular B Type II as per OPSD 802.010. The bedding material should be placed on the prepared subgrade as soon as practical, following its inspection and approval. The subgrade preparation and placement and compaction of the bedding material must be carried out in the dry. Construction equipment must not be allowed to travel on the bedding or the prepared subgrade, which must be protected from disturbance during construction.

A modulus of subgrade reaction of 25 MN/m<sup>3</sup> may be used for design of a pipe culvert at this site.



### **8.2.2 Concrete Box Culvert**

Replacement of the culverts with a concrete box culvert on the same alignment is identified as a feasible option for this site. It is anticipated that the subgrade soils within the culvert footprint will not be subjected to any significant additional loading from the replacement culvert.

In order to provide a uniform foundation subgrade, minimum 300 mm thick layer of bedding material conforming to OPSS.PROV 1010 Granular A or Granular B Type II requirements should be provided under the base of the box culvert, similar to as shown on OPSD 803.010. The bedding material must be placed on the prepared subgrade as soon as practicable following its inspection and approval. The subgrade preparation and placement and compaction of the bedding material must be carried out in the dry. The prepared surface for support of the box units should have a 75 mm minimum thickness top levelling course consisting of un-compacted Granular A as per OPSS 422. The bedding and the prepared subgrade shall be protected from disturbance during construction, therefore, construction equipment should not travel on the bedding or the prepared subgrade.

The following axial geotechnical resistances may be used for design of a box culvert of 5 m to 6 m wide with the culvert invert at or below Elevation 278 m on the compact to very dense silt or compact granular fill subgrade:

- Factored Geotechnical Resistance at Ultimate Limit State (ULS) of 300 kPa
- Factored Geotechnical Resistance at Serviceability Limit State (SLS) of 175 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/culvert size, configuration and applied loads; the geotechnical resistances should therefore be reviewed if the culvert width or founding/invert elevation differs significantly from that given above.

The geotechnical resistances provided in this section are applicable for vertical, concentric loads only. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with Clause 6.10.3 and Clause 6.10.4 of the CHBDC 2014.

Resistance to lateral forces (sliding resistance) between the concrete and the underlying Granular A or B Type II should be calculated assuming an ultimate (un-factored) coefficient of friction of 0.45. A resistance factor of 0.8 should be applied for the calculation of the factored sliding resistance in accordance with Table 6.2 of CHBDC 2014 based on a “typical” degree of understanding.

The culvert should be designed to resist external loadings including frost forces, lateral earth pressures, hydrostatic pressure, weight of embankment fill, traffic loadings and surcharge due to construction equipment.

### **8.2.3 Culvert Headwalls**

If headwalls are designed and constructed at the inlet and outlet of the replacement culvert, consideration may be given to using Retained Soil Systems (RSS) walls or cantilevered concrete walls. RSS walls are more tolerant to small amounts of settlement/heave.

The borehole information indicates that the founding conditions at the wall locations generally consist of compact granular fill or compact to very dense silt.

#### **8.2.3.1 RSS Walls**

RSS walls are considered to be a suitable option provided differential settlements are within tolerable limits and adequate factor of safety against global instability is achieved. The performance of an RSS wall when settlement occurs depends primarily on the characteristics of its front facing system. A typical precast panel facing can typically tolerate up to 1 per cent differential settlement and up to 30-40 mm of total settlement.

To provide an acceptable foundation performance, the RSS walls are often placed on a 0.5 m to 1.0 m thick engineered (granular) pad to deal with circumstances such as variable subsurface conditions and provide a consistent founding materials under the facing. The pad should extend to 300 mm beyond the outside edge of the facing and then downward at 1 horizontal to 1 vertical (1H:1V) side slope to the native soil. The engineered fill must consist of OPSS.PROV Granular A or Granular B Type II compacted to 100% of its standard proctor modified dry density (SPMDD) at a moisture content within 2% of its optimum. The

engineered pad must be at least 300 mm beyond the limits of the RSS mass and levelling strip.

The RSS walls should meet the geometry, performance and appearance criteria as outlined in the MTO's RSS Design Guidelines, 2008. RSS walls should be designed and constructed similar to MTO requirements which are provided in MTO Special Provision SP 599S22 (Retained Soil Systems) and SP 599S23 (Retained Soil System – Facing Elements).

The performance of a RSS wall is dependent on, among other factors, the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure (global instability) of the system. The entire block of reinforced earth must be designed against various modes of failure including bearing, sliding, overturning as well as internal stability.

An RSS wall founded on the compact silt/clayey silt at or below about Elevation 278.0 m may be designed using a factored geotechnical resistance at ULS of 250 kPa and a factored geotechnical resistance at SLS of 150 kPa (for 25 mm of settlement).

The geotechnical resistances are applicable for vertical, concentric loads only. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with Clause 6.10.3 and Clause 6.10.4 of the CHBDC 2014.

Sliding resistance along the base of the wall may be estimated using an ultimate friction coefficient of 0.4 for an engineered granular fill subgrade.

Topsoil, organics, loose fill, and any soft/wet material must be stripped from the footprint of the RSS. The subgrade under the RSS foundation should be inspected and any soft spots sub-excavated and replaced with compacted granular materials prior to placing fill. The subgrade preparation for the RSS wall and placement and compaction of the granular fill must be carried out in the dry.

A geotextile filter fabric must be incorporated in the RSS design to prevent loss of fines from the granular material behind the wall subject to fluctuating water levels. If the wall is subjected to flooding, the strip lengths may have to be larger than the typical 0.7 times the height of the RSS wall. The RSS supplier/designer should be alerted of this.

Lateral earth pressures acting on the RSS walls should be computed as described in Section 11. If the wall is retaining sloping backfill, appropriate earth pressure parameters for sloping backfill should be used.

#### **8.2.3.2 Foundation for Concrete Walls**

Concrete headwalls may be supported on spread footings founded on the compact to very dense silt/clayey silt subgrade. Any topsoil/organic or soft soil must be removed from the wall subgrade and replaced with granular fill compacted as per OPSS 501. The walls should be provided with sufficient frost cover (minimum 2.2 m) and founded at Elevation 277.0 m or below. A factored geotechnical resistance at ULS of 300 kPa and a geotechnical resistance at SLS of 175 kPa (for 25 mm of settlement) may be used for design. A 300 mm thick granular levelling pad should be provided below the footing. Load inclination and eccentricity should also be taken into account according to the CHBDC 2014 Clauses 6.10.3 and 6.10.4.

Resistance to lateral forces / sliding resistance between precast concrete and the underlying silt/sand should be evaluated in accordance with the CHBDC 2014 assuming an ultimate coefficient of friction of 0.3 for the compact to very dense silt/clayey silt.

Lateral earth pressures acting on the concrete wingwalls should be computed as described in Section 11. If the wall is retaining sloping backfill, appropriate earth pressure parameters for sloping backfill should be used.

#### **8.2.4 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). The base of open footing concrete culvert or headwall footings, if employed, should be provided with a minimum of 2.2 m of earth cover as protection against frost action. The frost cover requirement does not apply to the base of a CSP or box culvert. Where top of the CSP or box culvert is above the depth of frost penetration, frost treatment or a frost taper should be constructed similar to the configurations presented in OPSD 803.031 for CSP culverts or OPSD 803.010 for a box culvert.

### **8.2.5 Subgrade Preparation and Protection**

Performance of the replacement culvert and any headwalls will depend on the preparation of the subgrade. After the excavation reaches the design subgrade elevation, the exposed surface should be inspected to confirm that the subgrade is suitable and uniformly competent. Any remaining fill, topsoil, disturbed soils and any deleterious materials within the replacement culvert and headwall footprint at the subgrade level must be removed and replaced with well compacted granular materials.

In the event that subexcavation is required, the width of the subexcavation should be defined by a line extending from 0.3 m beyond the outside edge of the proposed culvert, outward and downward at 1H:1V. The subexcavated area should then be backfilled with granular material meeting OPSS.PROV 1010 Granular A or Granular B Type II requirements and compacted as per OPSS.PROV 501.

The excavation and backfilling should be carried out in accordance with OPSS 902. The subgrade preparation, placement and compaction of granular material must be carried out in the dry.

Where fine grained soils (silt and clay) are exposed at the foundation subgrade level, they will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that construction equipment be not allowed to travel on the bedding or the prepared subgrade which has to be protected from disturbance during construction.

A separation layer consisting of a non-woven geotextile should be placed between the subgrade and the underside of the bedding material. The geotextile should meet the specifications for OPSS 1860 Class II, and have a Fabric Opening Size (FOS) not greater than 150 micrometres.

### **8.2.6 Settlement**

It is understood that the proposed replacement will not result in highway grade raise or re-location of the culverts. Therefore, minimal post construction settlement is expected at this site. It must be noted that any additional load imposed on the culvert replacement, including fill placed adjacent to the extended culvert barrels, will induce immediate settlement and some long term settlement at this site.

### **8.3 Construction Considerations**

Where construction staging is required to maintain one lane of traffic, the following items should be considered in the planning and execution of the staged construction sequencing:

- Diversion of the creek will be required for construction. In addition, a suitable dewatering program will be required to facilitate the construction of the culvert in the dry.
- Temporary roadway protection may be required during all stages of construction, including excavation and removal of the existing culvert, installation of the new culvert and backfilling.
- All culvert and headwall subgrade preparation and foundation preparation must be carried out in the dry.

## **9. OPEN CUT EXCAVATION AND GROUNDWATER CONTROL**

All excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the embankment fill and native silty clay at this site are classified as Type 3 soils. The silt below the embankment fill that are anticipated in the inlet and outlet areas should be classified as Type 4 soils.

Excavation and backfilling for culvert construction should be carried out in accordance with OPSS 902.

Excavations for culvert replacement will be carried out through the existing embankment fill and may extend into the native silt. Obstructions such as cobbles or debris might be encountered within the embankment fill. Suggested wording for an NSSP on potential obstructions in the fill is included in Appendix F.

Installation of the culvert should be carried out in the dry. It is anticipated that excavation for culvert replacement will be carried out at or below the creek water level, and diversion of the creek flow will be required. Seepage should be anticipated from the embankment fill. Depending on the time of construction, a combination of cofferdam enclosures and creek diversion along with pumping from filtered sumps within an enclosure will be required to maintain dry excavations during the course of staged construction.

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. Dewatering must remain operational and effective until the culvert is installed and backfilled. 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.

## **10. STREAM DIVERSION PIPE**

The existing structure at the site is a twin CSP culvert. It is understood that one of the existing CSPs would be used as a stream diversion pipe during the construction and therefore, a new diversion pipe is not required at the site.

## **11. CULVERT BACKFILL AND LATERAL EARTH PRESSURES**

Backfill to the culvert 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 802.010 or 803.010, as appropriate. Backfilling for the culvert should be in accordance with OPSS.PROV 401 for a CSP or OPSS 902 for a box culvert. All fills should be placed in regular lifts and be compacted in accordance with OPSS.PROV 501. The backfill should be placed and compacted in simultaneous lifts on both sides of the culvert, and the top of backfill elevation should not differ more than 500 mm on both sides of the culvert at all times. Heavy compaction equipment should not be used adjacent to the walls and on the roof of the culvert. Compaction equipment to be used adjacent to the culvert should be restricted in accordance with OPSS.PROV 501.

Lateral earth pressures acting on the culvert walls may be assumed a triangular distribution. For a fully drained backfill, the pressures should be computed in accordance with the CHBDC 2014, but are generally given by the expression below:

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

where

$p_h$	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see table below)
$\gamma$	=	bulk unit weight of retained soil (see table below)

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

$q$  = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the culvert walls are dependent on the material used as backfill. Recommended unfactored values are shown in Table 11.1 below.

**Table 11.1 – Lateral Earth Pressure Coefficients (K)**

Loading Condition	OPSS Granular A or Granular B Type II $\phi = 35^\circ$ ; $g = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I (modified) $\phi = 32^\circ$ ; $g = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48
At-rest (Restrained Wall)	0.43	0.62	0.47	0.70
Passive	3.7	-	3.3	-

Note: Submerged unit weight should be used below the groundwater level/high creek level.

In general the lateral earth pressure applied to a retaining structure (e.g., headwalls and/or vertical side walls of the culverts) 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 stem) such as in a box culvert configuration, at rest earth pressures should be assumed for geotechnical design. If the wall support allows lateral movements (unrestrained stem) such as in concrete headwalls, 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.

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

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 and vertical side walls of the culverts. 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).



## 12. SEISMIC CONSIDERATIONS

The stratigraphy of the site is typically a compact to dense silt over very stiff to hard silty clay which is underlain by bedrock at a depth of about 13 m. This corresponds to a Seismic Site Class C in accordance with Table 4.1, Clause 4.4.3.2 of the CHBDC 2014. The reference peak ground acceleration and velocity, PGA and PGV for a 2%, 5% and 10% probabilities of exceedance (equivalent of return periods of 475, 975 and 2475, respectively) in 50 years for Site Class C at the project site, based on the National Building Code of Canada (NBCC) 2015, are estimated and summarized in Table 12.1 below.

**Table 12.1 – Seismic Hazard Values for Reference Ground Conditions Site Class C**

Return Period (Years)	Probability of Exceedance	Coefficient of $PGA_{ref}$	Coefficient of $PGV_{ref}$
475	10% in 50 Years	0.010	0.007
975	5% in 50 Years	0.018	0.013
2475	2% in 50 Years	0.037	0.025

The new structure is assigned Seismic performance category 1 based on Table 4.10 of the CHBDC 2014, unless it is recognized as a “Lifeline bridge”. Structures in Seismic performance category 1 need not be analyzed for seismic loads regardless of their importance and geometry in accordance with Section 4.4.5.1 of the CHBDC 2014.

For “Lifeline bridge” structures, retaining structures should be designed using active ( $K_{AE}$ ) earth pressure coefficient that incorporate the effects of earthquake loading, in accordance with Clause 4.6.5 of the CHBDC 2014. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total active earth pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma' d + (K_{AE} - K_a) \gamma' (H-d)$$

where

- $\sigma_h(d)$  = the lateral earth pressure at depth d, (kPa)
- $K_a$  = either the static active earth pressure coefficient ( $K_a$ )
- $K_{AE}$  = the seismic active earth pressure coefficient;
- $\gamma'$  = the effective unit weight of the backfill soil (kN/m<sup>3</sup>),  
taken as soil unit weight given above;
- d = the depth below the top of the wall (m); and

$H$  = the total height of the wall above its toe (m).

The coefficients of horizontal earth pressure for seismic loading presented in Table 12.2 may be used:

**Table 12.2 – Active Earth Pressure Coefficients for Earthquake Loading**

Condition	Earth Pressure Coefficient (K)	
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I (modified) $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$
Active ( $K_{AE}$ )*	0.28	0.32

\* After Mononobe and Okabe.

The site is underlain by compact to dense silt overlying very stiff to hard silty clay. Therefore, liquefaction is not considered to be a concern at this site.

### 13. TEMPORARY PROTECTION SYSTEM

The temporary roadway protection system should be implemented in accordance with OPSS.PROV 539 and designed for Performance Level 2, provided that any nearby utility/structure can tolerate this magnitude of deformation.

Options for roadway protection are a soldier pile-lagging system or sheet piles, although the sheet piles may be difficult to drive in the fill, which could include obstructions such as cobbles and boulders.

The soil parameters in Table 13.1 may apply for design of the temporary roadway protection system with horizontal backfill.

**Table 13.1 –Soil Parameters for Temporary Protection System Design**

Soil Parameter	Existing Fill	Native Silt/Sand
$\gamma$	21 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
$\gamma_w$	10 kN/m <sup>3</sup>	10 kN/m <sup>3</sup>
$K_a$	0.33	0.33
$K_p$	3.0	3.0
$K_0$	0.5	0.5

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

#### **14. EMBANKMENT RESTORATION**

The existing Highway 595 embankment is approximately 2 m above the culvert at the site location and the 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 grade raise or embankment widening at this site for the culvert replacement, and therefore settlement of the embankment is not a concern. Any settlement due to changes in the culvert configuration is expected to be less than 25 mm. Additional settlement would be induced if the final configuration includes additional fill adjacent to the culvert barrels.

Embankment restoration after completion of the culvert 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 above the culvert granular cover and below the roadbase granular fill, 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 areas around the culvert inlets and outlets, 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 culvert inlet and outlet. Design of the erosion protection measures should consider hydrologic and hydraulic factors and should be carried out

by specialists experienced in this field and in accordance with OPSD 810.010, OPSS 511 and OPSS.PROV 1004.

Typically, rock protection should be provided over all surfaces with which creek 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.

A concrete cut-off wall or clay seal should be used at the inlet to minimize the potential for erosion or piping around the culvert. The clay seal should extend to approximately 0.3 m above the high water level and laterally for the width of the granular material, and have a minimum thickness of 0.5 m. The material requirements should be in accordance with OPSS.PROV 1205. A geosynthetic clay liner may be used in lieu of a compacted clay seal.

## **16. CORROSION AND SULPHATE ATTACK POTENTIAL**

The results of the corrosivity and sulphate analytical tests conducted on the native soil and creek water indicates the following conditions at the locations tested:

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

## **17. OTHER CONSTRUCTION CONCERNS**

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

- A suitable dewatering / unwatering system must be employed to enable culvert construction in the dry and prevent base boiling, sloughing and instability of the excavation walls.
- The water level in the creek may fluctuate and be at higher elevation at the time of construction than indicated in the report.

- Buried obstructions may be encountered during excavation in the existing embankment fill and may interfere with installation of the temporary roadway protection system.
- 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.

## 18. CLOSURE

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

Thurber Engineering Ltd.



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Senior Geotechnical Engineer



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 <sup>(1)</sup> 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

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


### 4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

### 5. LEGEND FOR RECORDS OF BOREHOLES

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

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

 Water Level  
 Shear Strength Determination by Pocket Penetrometer

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



## EXPLANATION OF ROCK LOGGING TERMS


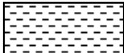



### ROCK WEATHERING CLASSIFICATION

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

### DISCONTINUITY SPACING

<b>Bedding</b>	<b>Bedding Plane Spacing</b>
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

<b>Rock Strength</b>	<b>Approximate Uniaxial Compressive Strength</b>		<b>Field Estimation of Hardness*</b>
	<b>(MPa)</b>	<b>(psi)</b>	
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.

# 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 <sub>L</sub> < 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 <sub>L</sub> < 30%).
		CI	Inorganic clays of medium plasticity, silty clays. (30% < W <sub>L</sub> < 50%).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS W <sub>L</sub> > 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			

# RECORD OF BOREHOLE No 17-01

1 OF 2

METRIC

W.P. \_\_\_\_\_ LOCATION Whitewood Creek N 5 357 870.4 E 326 991.7 ORIGINATED BY ES  
 HWY 595 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2017.04.12 - 2017.04.12 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
279.5	GROUND SURFACE							20 40 60 80 100						
0.0	<b>TOPSOIL:</b> (50 mm)		1	GS			279							
	<b>SAND</b> and <b>GRAVEL</b> , some silt, trace roots and rootlets Loose to Compact Brown Moist (FILL)  Trace clay		1	SS	8		278							37 45 18 (SI+CL)
			2	SS	16		277							
277.3							276							
2.2	<b>SILT</b> , some clay Dense Grey Moist		3	SS	32		275							0 0 78 22
			4	SS	46		274							
							273							
274.9			5	SS	44		272							
4.6	Silty <b>CLAY</b> Hard Brown Moist (CI)						271							
			6	SS	38		270							0 0 58 42
							269.8							
			7	SS	40									
	Occasional silt lenses													
			8	SS	39									
269.8														
9.8	END OF BOREHOLE AT 9.8 m.													

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 17-01

2 OF 2

METRIC

W.P. \_\_\_\_\_ LOCATION Whitewood Creek N 5 357 870.4 E 326 991.7 ORIGINATED BY ES  
 HWY 595 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2017.04.12 - 2017.04.12 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
							20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>			
	Continued From Previous Page																
	GROUND WATER LEVEL AT 1.9 m UPON COMPLETION. Standpipe piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 3.0 m slotted screen.  PIEZOMETER REMOVED ON APRIL 30, 2017. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.  DCPT STARTED ABOUT 1.4 m NORTH OF THE BOREHOLE AT A DEPTH OF 0.9 m DUE TO NEAR SURFACE BOULDERS AND ENDED AT A DEPTH OF 5.0 m DUE TO REFUSAL.  WATER LEVEL READINGS DATE      DEPTH(m)    ELEV.(m) 2017.04.30      2.2      277.3																

ONTMT4S MTO-17840.GPJ 2015TEMPLATE(MTO).GDT 5/31/17

# RECORD OF BOREHOLE No 17-02

1 OF 2

METRIC

W.P. \_\_\_\_\_ LOCATION Whitewood Creek N 5 357 859.8 E 327 005.1 ORIGINATED BY ES  
 HWY 595 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2017.04.06 - 2017.04.06 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
282.5	GROUND SURFACE							20 40 60 80 100		20 40 60				
0.0	ASPHALT: (40 mm)		1	GS			282							
	SAND and GRAVEL, trace to some fines Dense Brown Moist (FILL)		1	SS	33									39 49 12 (SI+CL)
	Occasional cobbles		2	SS	30		281							
			3	SS	34		280							
279.4														
3.1	Clayey SILT, some sand, trace gravel Firm Brown Moist (FILL)		4	SS	7		279							
278.4														
4.1	SAND, trace silt, trace clay Compact Brown Wet (FILL)		5	SS	19		278							
276.9							277							
5.6	Clayey SILT Hard Grey (CL-ML)		6	SS	51		276							0 0 82 18
	Trace sand		7	SS	41		275							
273.8							274							
8.7	Silty CLAY Hard Grey Moist (CI)		8	SS	31		273							

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

ONTMT4S MTO-17840 GPJ 2015TEMPLATE(MTO).GDT 5/31/17

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No 17-03

1 OF 2

METRIC

W.P. \_\_\_\_\_ LOCATION Whitewood Creek N 5 357 869.7 E 327 009.5 ORIGINATED BY ES  
 HWY 595 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2017.04.07 - 2017.04.07 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W <sub>P</sub> W      W <sub>L</sub>				WATER CONTENT (%)				
								○ UNCONFINED      + FIELD VANE	● QUICK TRIAXIAL      × LAB VANE											
281.8	GROUND SURFACE							20	40	60	80	100		20	40	60				
0.0	ASPHALT: (25 mm)																			
	SAND, some gravel to gravelly, trace fines Dense to Very Dense Brown Moist (FILL)		1	GS									○							
			1	SS	55								○							
	Occasional cobbles		2	SS	51								○							
			3	SS	33								○							
			4	SS	50/ 0.150								○							
													</							

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

## METRIC

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity



# RECORD OF BOREHOLE No 17-04

1 OF 2

METRIC

W.P. \_\_\_\_\_ LOCATION Whitewood Creek N 5 357 857.8 E 327 020.0 ORIGINATED BY AHF  
 HWY 595 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2017.04.30 - 2017.04.30 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100					
278.5	GROUND SURFACE												
0.0	<b>SAND</b> and <b>GRAVEL</b> , trace silt Loose Brown Wet (FILL)		1	GS									
			1	SS	4								
277.0													
1.5	<b>SILT</b> , some clay, trace fine sand Compact Grey Moist		2	SS	19								0 0 85 15
276.3													
2.2	Silty <b>CLAY</b> Hard to Very Stiff Grey Moist		3	SS	73								
			4	SS	66								
			5	SS	49								
			6	SS	40								0 0 61 39
			7	SS	26								
			8	SS	26								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-04

2 OF 2

METRIC

W.P. \_\_\_\_\_ LOCATION Whitewood Creek N 5 357 857.8 E 327 020.0 ORIGINATED BY AHF  
 HWY 595 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2017.04.30 - 2017.04.30 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)						
						20	40	60	80	100	20	40	60				
	Continued From Previous Page																
267.5			9	SS	40/ 0.075												
11.0	END OF BOREHOLE AT 11.0 m DUE TO AUGER REFUSAL. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.																

ONTMT4S MTO-17840.GPJ 2015TEMPLATE(MTO).GDT 5/31/17

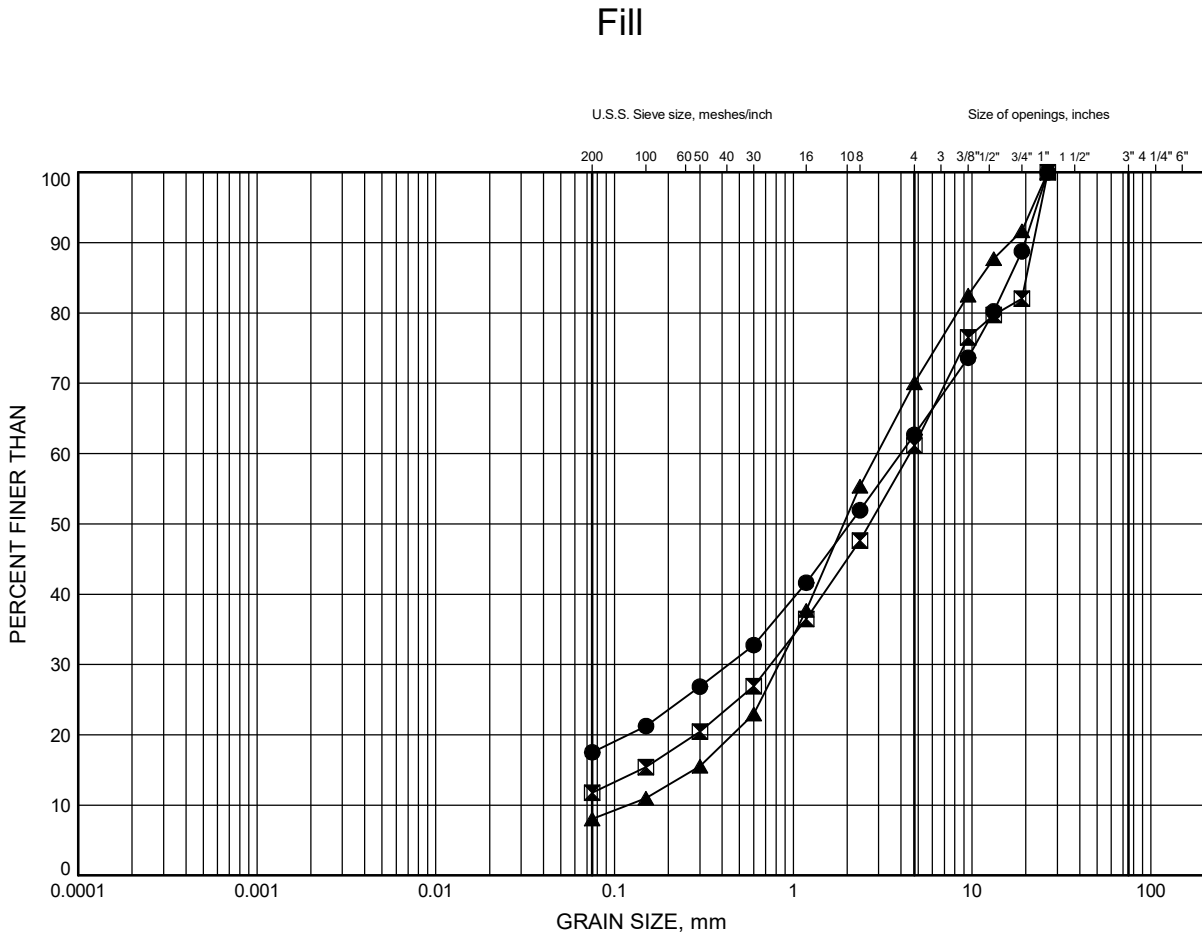


## **Appendix B**

### **Geotechnical and Analytical Laboratory Test Results**

# Whitewood Creek GRAIN SIZE DISTRIBUTION

FIGURE B1



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

## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-01	1.1	278.5
⊠	17-02	1.1	281.4
▲	17-03	1.8	280.0

Date May 2017  
W.P. \_\_\_\_\_

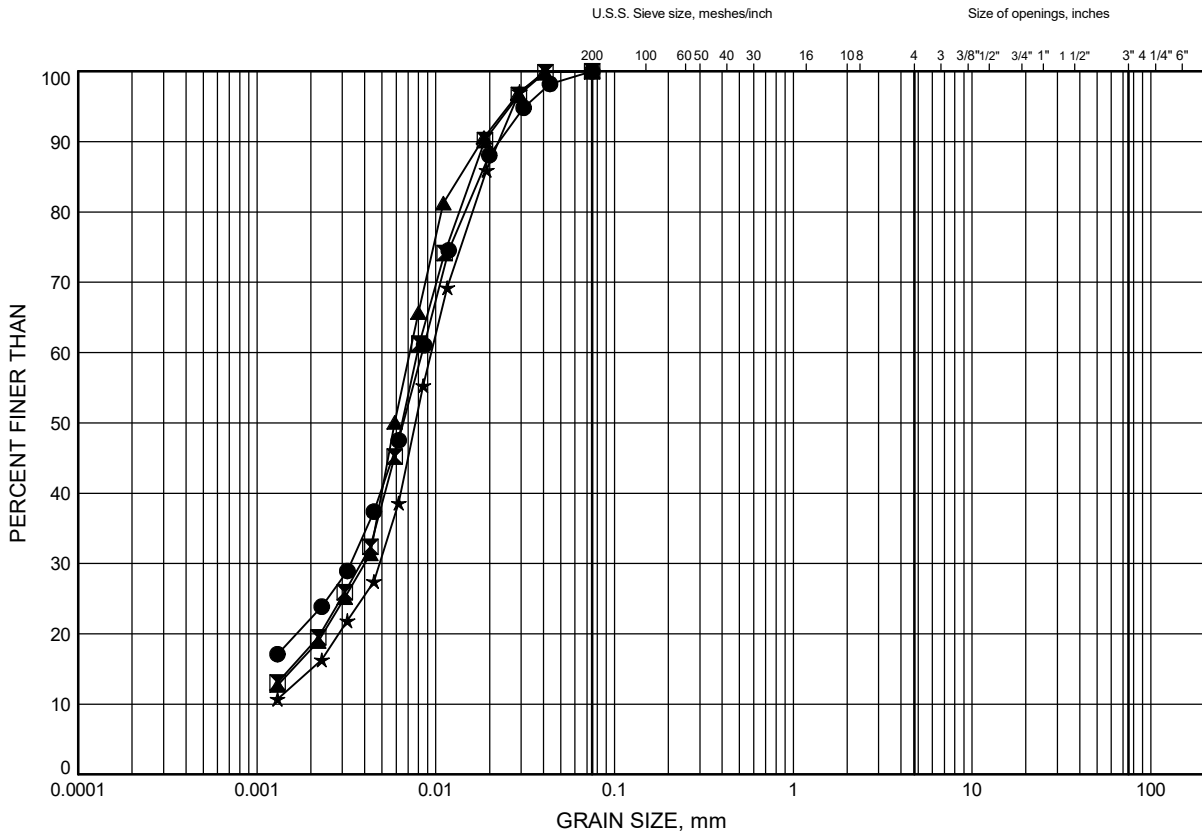


Prep'd MFA  
Chkd. MM

# Whitewood Creek GRAIN SIZE DISTRIBUTION

FIGURE B2

## Silt / Clayey Silt



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-01	2.6	276.9
⊠	17-02	6.4	276.1
▲	17-03	4.9	277.0
★	17-04	1.8	276.7

Date May 2017  
W.P. \_\_\_\_\_

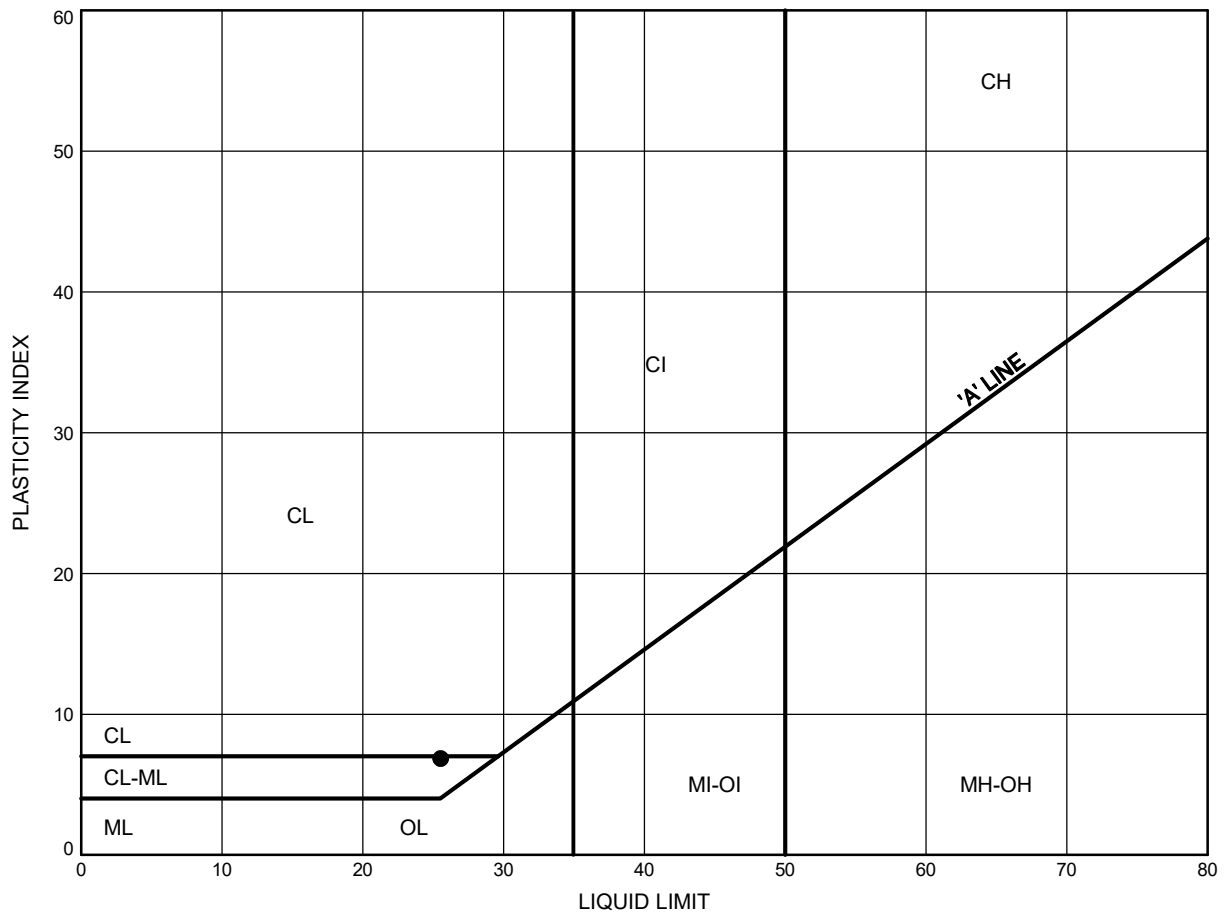


Prep'd MFA  
Chkd. MM

# Whitewood Creek ATTERBERG LIMITS TEST RESULTS

FIGURE B3

Silt / Clayey Silt



## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-02	6.4	276.1

Date May 2017  
W.P. ....

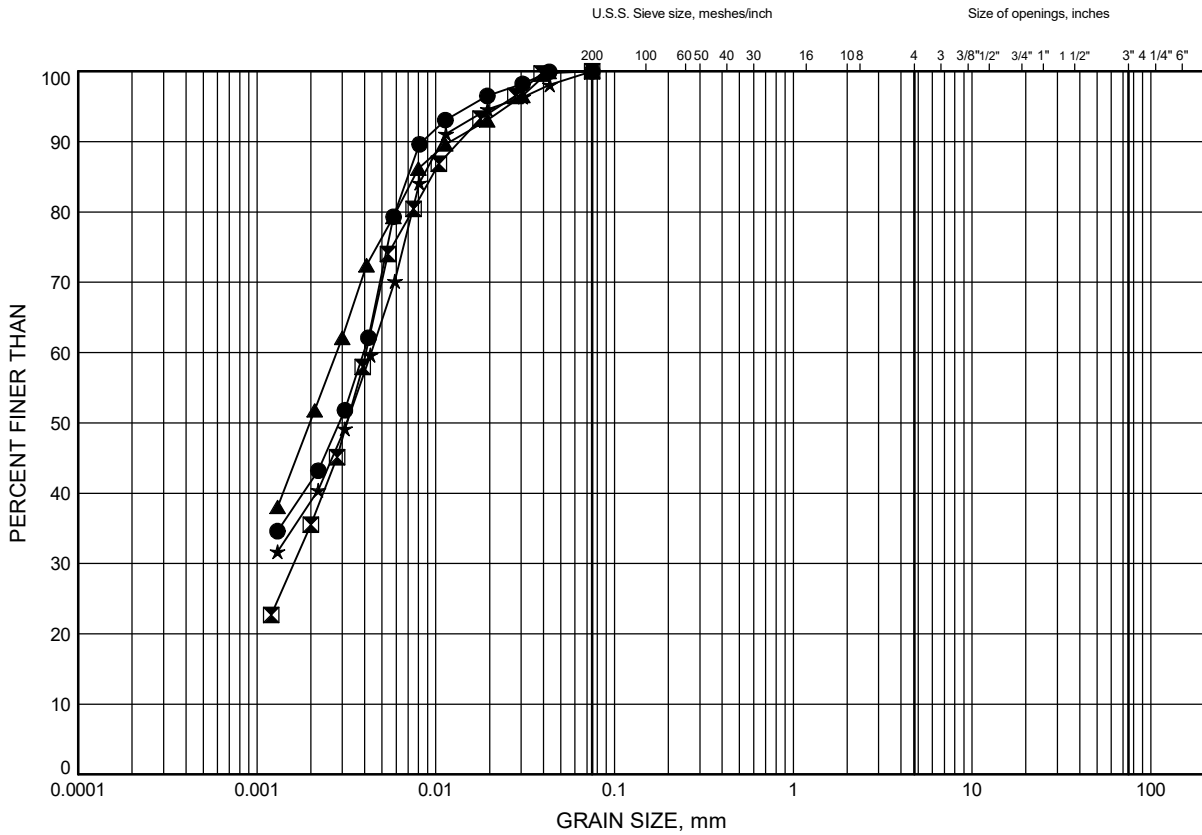


Prep'd MFA  
Chkd. MM

# Whitewood Creek GRAIN SIZE DISTRIBUTION

FIGURE B4

## Silty Clay



### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-01	6.4	273.1
⊠	17-02	11.0	271.5
▲	17-03	9.4	272.4
★	17-04	6.4	272.1

Date May 2017  
W.P.

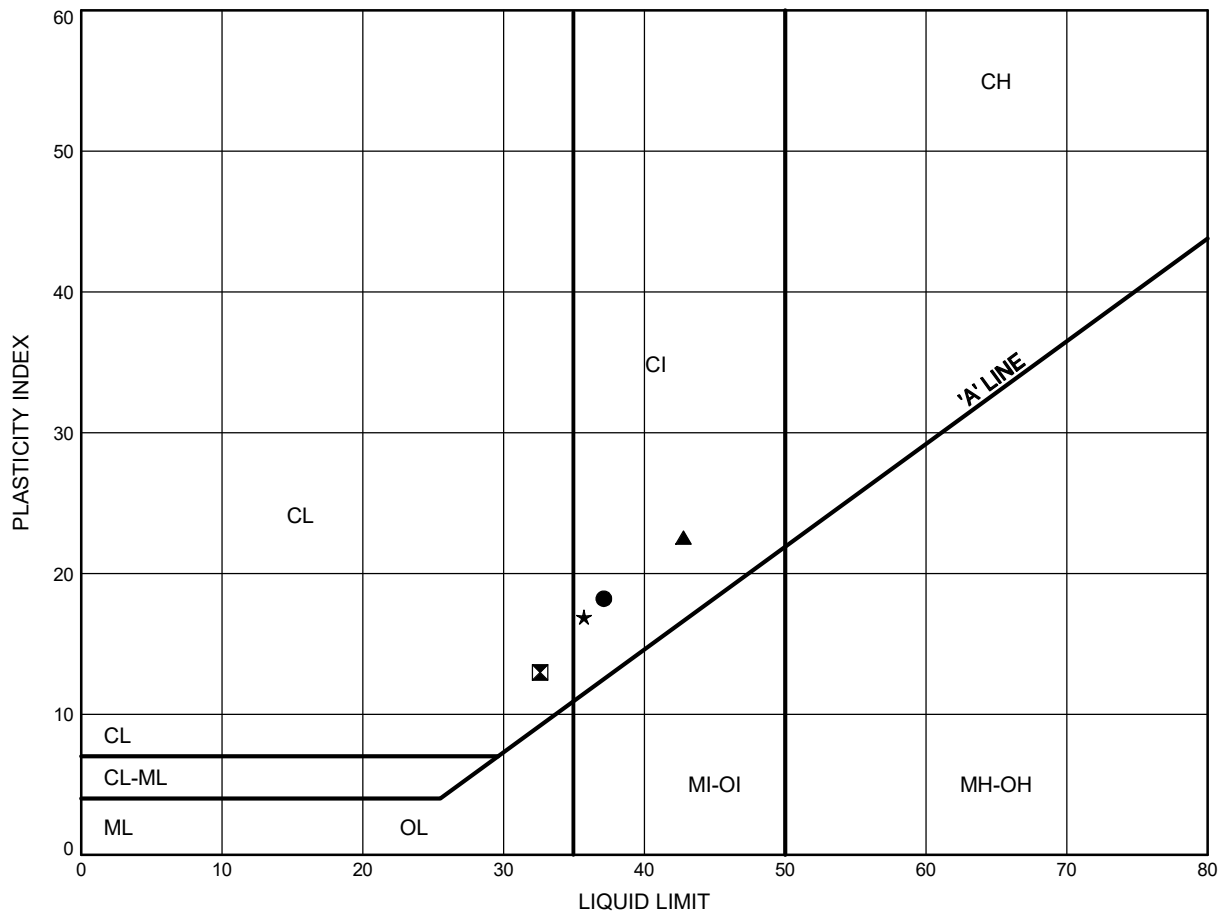


Prep'd MFA  
Chkd. MM

# Whitewood Creek ATTERBERG LIMITS TEST RESULTS

FIGURE B5

Silty Clay



## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-01	6.4	273.1
⊠	17-02	11.0	271.5
▲	17-03	9.4	272.4
★	17-04	6.4	272.1

Date May 2017  
W.P. \_\_\_\_\_



Prep'd MFA  
Chkd. MM





## ASTM D5731-08

<b>Date Drilled:</b>	06-Apr-17
<b>Date Tested:</b>	01-May-17
<b>Tester:</b>	GA
<b>Reviewed by:</b>	CZ

[illegible]



# ASTM D5731-08

Date Drilled:	07-Apr-17
Date Tested:	01-May-17
Tester:	GA
Reviewed by:	CZ

[illegible]

**SGS Canada Inc.**

P.O. Box 4300 - 185 Concession St.  
Lakefield - Ontario - K0L 2H0  
Phone: 705-652-2000 FAX: 705-652-6365

**Project : 17840**

19-April-2017

**Thurber Engineering Ltd.**

Attn : Mark Farrant

103, 2010 Winston Park Drive  
Oakville, ON  
L6H 5R7,

Phone: 905-829-8666 x 228  
Fax:

**Date Rec. :** 12 April 2017  
**LR Report:** CA13544-APR17  
**Reference:** 17840 Mark Farrant


**Copy:** #1

## CERTIFICATE OF ANALYSIS

### Final Report

Analysis	1: Analysis Start Date	2: Analysis Start Time	3: Analysis Approval Date	4: Analysis Approval Time	5: MDL	7: Whitewood Creek Culvert	8: Sitch Creek Culvert Hwy 588
Sample Date & Time						06-Apr-17 18:00	09-Apr-17 18:30
Temperature Upon Receipt [°C]					---	13.0	13.0
pH [no unit]	13-Apr-17	08:53	17-Apr-17	14:39	0.05	7.46	7.25
Conductivity [uS/cm]	13-Apr-17	08:53	17-Apr-17	14:39	2	129	90
Resistivity (calculated) [Ohms.cm]	17-Apr-17	16:09			---	7750	11100
Redox Potential [mV]	12-Apr-17	13:31	13-Apr-17	11:41	---	295	303
Chloride [mg/L]	12-Apr-17	16:30	13-Apr-17	12:41	0.04	7.3	3.0
Sulphate [mg/L]	12-Apr-17	16:30	13-Apr-17	12:41	0.04	3.2	3.0
Sulphide [mg/L]	13-Apr-17	10:15	17-Apr-17	10:30	0.006	< 0.006	< 0.006

Temperature of Sample upon Receipt: 13 degrees C  
Cooling Agent Present: No  
Custody Seal Present: No

  
Deanna Edwards, B.Sc, C.Chem  
Project Specialist  
Environmental Services, Analytical

**SGS Canada Inc.**

P.O. Box 4300 - 185 Concession St.

Lakefield - Ontario - KOL 2H0

Phone: 705-652-2000 FAX: 705-652-6365

**Project :** 17840**LR Report :** CA13544-APR17

## Method Descriptions

Parameter	Units	SGS Method Code	Reference Method Code
Anions by IC	mg/L	ME-CA-[ENV]IC-LAK-AN-001	EPA300/MA300-Ions1.3
Conductivity	uS/cm	ME-CA-[ENV]EWL-LAK-AN-006	SM 2510
pH	no unit	ME-CA-[ENV]EWL-LAK-AN-006	SM 4500
Redox Potential	mV		SM 2580
Sulphide by SFA	mg/L	ME-CA-[ENV]SFA-LAK-AN-008	SM 4500



**SGS Canada Inc.**

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**Project :** 17840

**LR Report :** CA13544-APR17

## 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: DIO0140-APR17												
Chloride	0.04	mg/L	<0.04		4	20	100	80	120	104	75	125
Sulphate	0.04	mg/L	<0.04		2	20	98	80	120	107	75	125
Conductivity - QCBatchID: EWL0169-APR17												
Conductivity	2	uS/cm	2		3	10	97	90	110	NA		
pH - QCBatchID: EWL0169-APR17												
pH	0.05	no unit	NA		0		100			NA		
Redox Potential - QCBatchID: EWL0152-APR17												
Redox Potential	no	mV	NA		4	20	104	80	120	NA		
Sulphide by SFA - QCBatchID: SKA0110-APR17												
Sulphide	0.006	mg/L	<0.006		ND	20	81	80	120	NV	75	125



**SGS Canada Inc.**

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## Thurber Engineering Ltd

Attn : Cory Zanatta

2010 Winston Park Dr  
Oakville, ON  
L6H 5R7,

Phone: 905-829-8666 x 240

Fax:

**Project :** 17742/17840

24-May-2017

**Date Rec. :** 17 May 2017

**LR Report:** CA14528-MAY17

**Reference:** 17742/17840 Cory Zanatta

**Copy:** #1

# CERTIFICATE OF ANALYSIS

## Final Report

Analysis	1: Analysis Start Date	2: Analysis Start Time	3: Analysis Approval Date	4: Analysis Approval Time	5: 17840 17-04 SS4	6: 17840 17-08 SS6	7: 17840 17-06 SS7	8: 17840 17-03 SS5
Sample Date & Time					15-May-17	15-May-17	15-May-17	15-May-17
Temperature Upon Receipt [°C]	---	---	---	---	10.0	10.0	10.0	10.0
Corrosivity Index [none]	24-May-17	13:45	24-May-17	13:45	7.5	4.5	7.5	4.0
Soil Redox Potential [mV]	18-May-17	19:36	19-May-17	14:01	139	152	272	237
Sulphide [%]	23-May-17	12:52	23-May-17	13:09	0.67	0.53	0.51	< 0.02
% Moisture (wet wt) [%]	23-May-17	10:42	23-May-17	10:44	19.3	19.8	9.9	17.9
pH [no unit]	19-May-17	14:44	24-May-17	13:14	8.73	8.22	8.51	8.55
Chloride [µg/g]	19-May-17	12:04	23-May-17	11:42	16	5.9	15	25
Sulphate [µg/g]	19-May-17	12:04	23-May-17	11:42	54	68	200	61
Conductivity [uS/cm]	19-May-17	14:44	24-May-17	13:14	76	92	173	109
Resistivity (calculated) [Ohms.cm]	19-May-17	14:44	24-May-17	13:14	13200	10900	5780	9170



**SGS Canada Inc.**

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Phone: 705-652-2000 FAX: 705-652-6365

**Project :** 17742/17840

**LR Report :** CA14528-MAY17

<b>Analysis</b>	<b>9: 17840 17-02 SS6</b>	<b>10: 17840 17-07 SS7</b>	<b>11: 17792 17-03 SS3</b>	<b>12: 17792 17-02 SS4</b>
Sample Date & Time	15-May-17	15-May-17	15-May-17	15-May-17
Temperature Upon Receipt [°C]	10.0	10.0	10.0	10.0
Corrosivity Index [none]	7.5	7.5	2.0	1.0
Soil Redox Potential [mV]	200	256	278	315
Sulphide [%]	0.05	0.39	< 0.02	< 0.02
% Moisture (wet wt) [%]	18.9	14.1	20.1	10.9
pH [no unit]	8.68	8.47	7.40	6.03
Chloride [µg/g]	55	59	260	66
Sulphate [µg/g]	110	200	8.3	32
Conductivity [uS/cm]	157	200	384	150
Resistivity (calculated) [Ohms.cm]	6370	5000	2600	6670

Temperature of Sample upon Receipt: 10 degrees C

Cooling Agent Present: Yes

Custody Seal Present: No

Corrosivity Index is based on the American Water Works 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, B.Sc, C.Chem**

**Project Specialist**

**Environmental Services, Analytical**



**SGS Canada Inc.**

P.O. Box 4300 - 185 Concession St.

Lakefield - Ontario - K0L 2H0

Phone: 705-652-2000 FAX: 705-652-6365

**Project :** 17742/17840

**LR Report :** CA14528-MAY17

### Method Descriptions

Parameter	SGS Method Code
Anions by IC	ME-CA-[ENV]IC-LAK-AN-001
Carbon/Sulphur	ME-CA-[ENV]ARD-LAK-AN-020
Conductivity	ME-CA-[ENV]EWL-LAK-AN-006
Metals Prep	ME-CA-[ENV]ARD-LAK-AN-013
pH	ME-CA-[ENV]EWL-LAK-AN-001





**SGS Canada Inc.**

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Phone: 705-652-2000 FAX: 705-652-6365

**Project :** 17742/17840

**LR Report :** CA14528-MAY17

## 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 (%)
					%	Low				High	Low	
Anions by IC - QCBatchID: DIO0347-MAY17												
Chloride	0.4	µg/g	<0.4		12	20	97	80	120	97	75	125
Sulphate	0.4	µg/g	<0.4		5	20	97	80	120	86	75	125
Carbon/Sulphur - QCBatchID: ECS0026-MAY17												
Sulphide	0.02	%	<0.02		ND	20	117	80	120			
Conductivity - QCBatchID: EWL0361-MAY17												
Conductivity	2	uS/cm	< 2		0	10	96	90	110	NA		
pH - QCBatchID: EWL0361-MAY17												
pH	0.05	no unit	NA		0		100			NA		



## **Appendix C**

### **Selected Site Photographs**



**Photograph 1 – Whitewood Creek Culverts, East End (Outlet)**



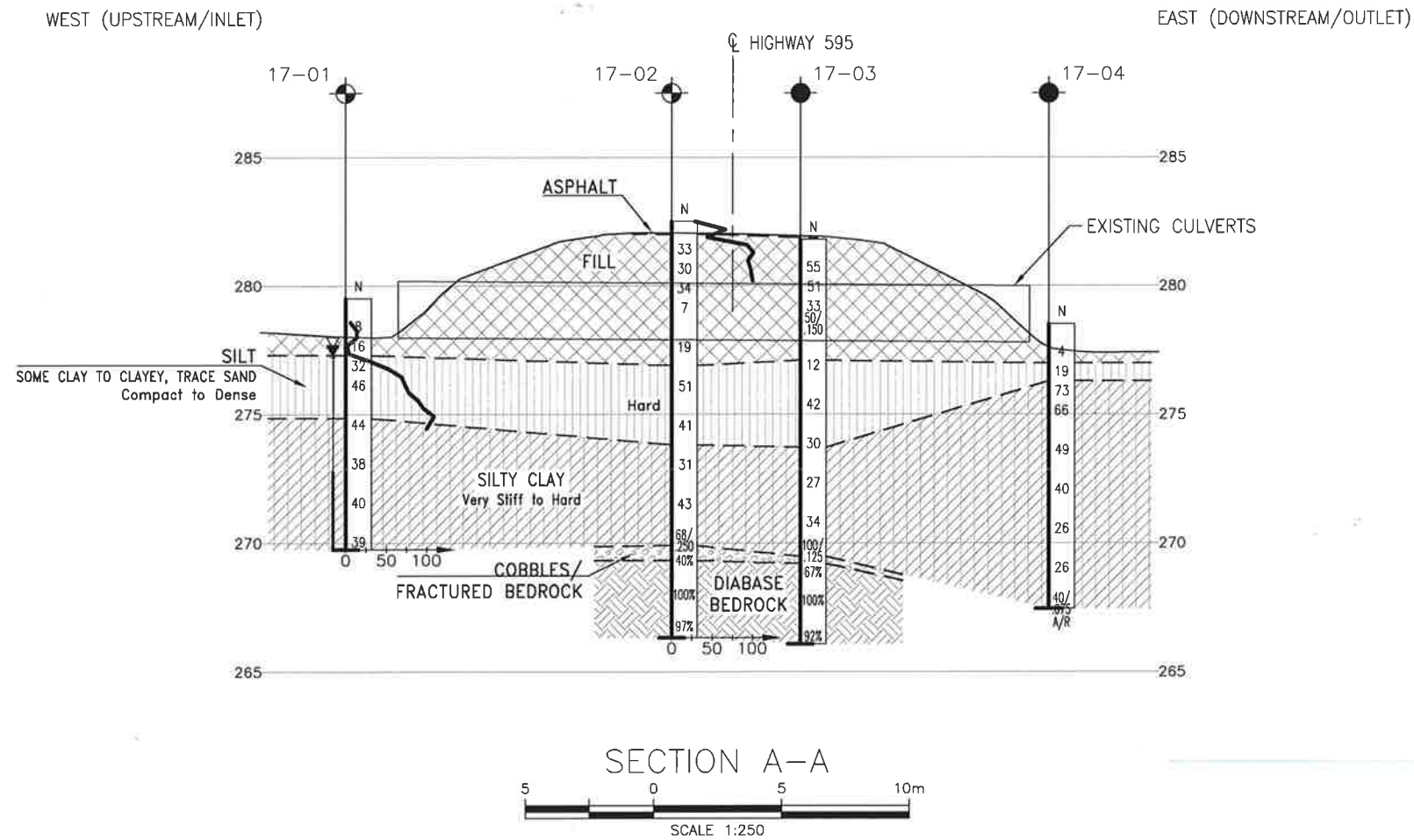
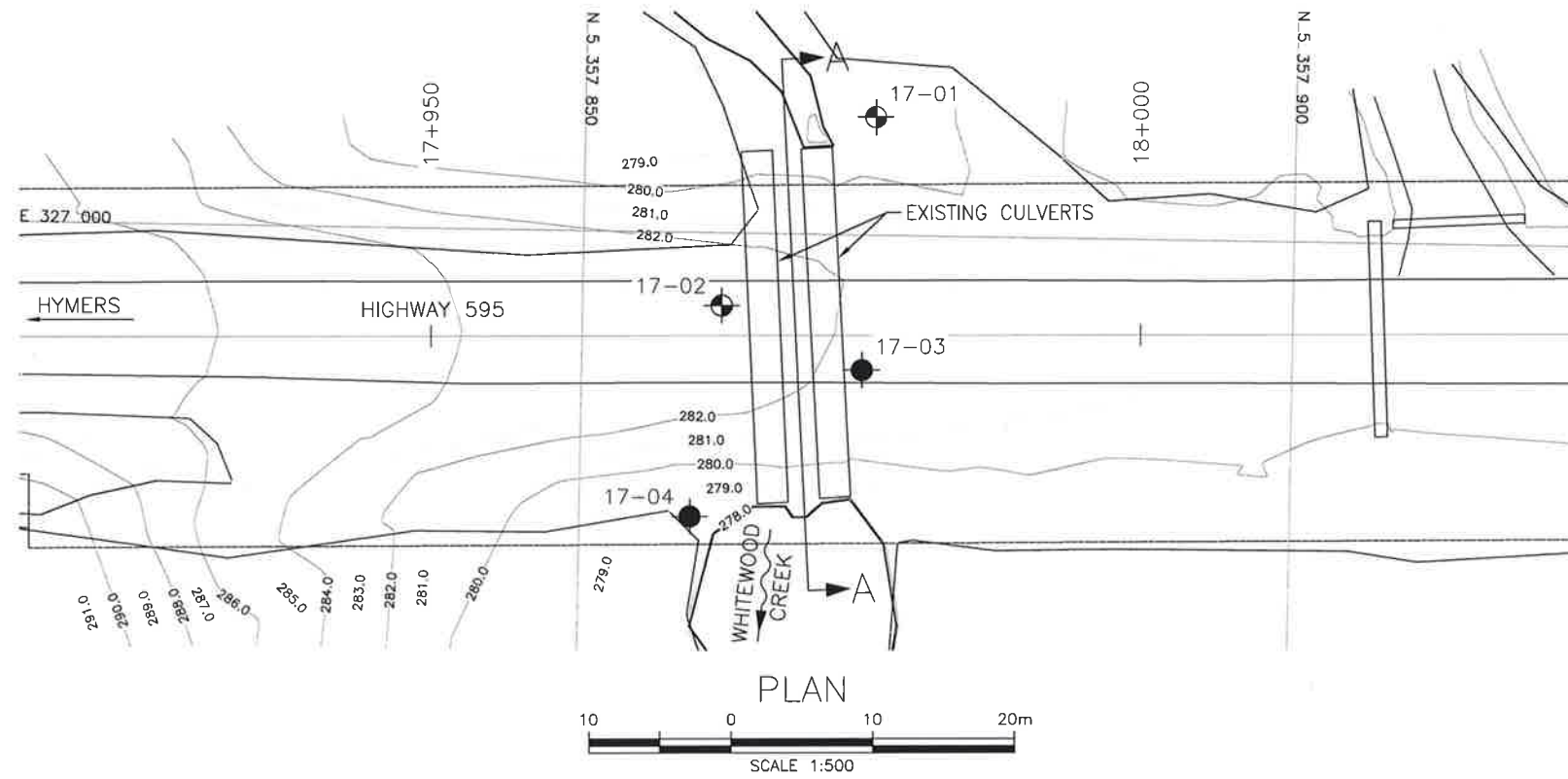


**Photograph 2 – Whitewood Creek Culverts, West End (Inlet)**



## **Appendix D**

### **Borehole Locations and Soil Strata Drawing**



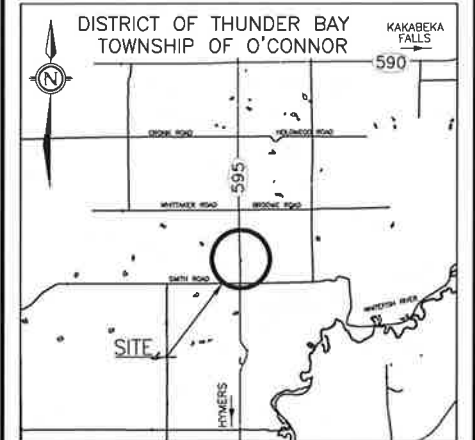
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No  
WP No








HIGHWAY 595  
WHITEWOOD CREEK  
CULVERTS  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



## 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
17-01	279.5	5 357 870.4	326 991.7
17-02	282.5	5 357 859.8	327 005.1
17-03	281.8	5 357 869.7	327 009.5
17-04	278.5	5 357 857.8	327 020.0

-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-228**

[illegible]



## **Appendix E**

### **Comparison of Foundation Alternatives**



### COMPARISON OF FOUNDATION ALTERNATIVES

Corrugated Steel Pipe (CSP) Culvert	Concrete Box Culvert	Concrete Open Footing Culvert
<u>Advantages:</u> i. Ease of construction. ii. Segmented pipes can accommodate some potential differential settlement along culvert axis.	<u>Advantages:</u> i. Relatively rapid installation and less disturbance to subgrade soils if precast segments are used. ii. Segmental option can accommodate some potential differential settlement along culvert axis.	<u>Advantages:</u> i. Conventional construction. ii. Possibly less disturbance of creek channel / less environmental issues such as those involving spawning fish species.
<u>Disadvantages:</u> i. Multiple pipes may be needed to meet hydraulic design (capacity) requirements. ii. Temporary roadway protection system is required.	<u>Disadvantages:</u> i. More expensive than a CSP culvert. ii. Relatively large excavation required to install culvert. iii. Temporary roadway protection system required.	<u>Disadvantages:</u> i. Greater potential for differential settlement. ii. Deeper excavation and potentially longer dewatering requirements. iii. More extensive roadway protection is required compared to the other two options. iv. More disturbance of creek.
<b>FEASIBLE</b>	<b>FEASIBLE</b>	<b>NOT RECOMMENDED</b>





## **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 422
- OPSS.PROV 501
- 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.803.010
- OPSD.810.010

## **2. Suggested Wording for NSSP**

- Suggested Text for NSSP on “Obstructions”

“Excavations and installation of cofferdams and roadway protection systems could encounter obstructions such as cobbles and boulders embedded in the fill and native soils, or shallow bedrock. Such obstructions may impede excavation progress and/or sheetpile installation. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions to achieve the design depths.”

- Suggested Text for NSSP on “Groundwater and 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.