



THURBER ENGINEERING LTD.

FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17 TWINNING, RENFREW AREA
CULVERT 10+890 WBL, MCNAB/BRAESIDE
WP 4068-09-00 / ASSIGNMENT NO. 4018-E-0009

Geocres No.: 31F-217

Report to:

Ministry of Transportation Ontario

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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

Thurber Engineering Ltd. (Thurber) has been engaged by the Ministry of Transportation Ontario (MTO) under Assignment No. 4018-E-0009 to carry out Foundation Investigations to support the design of the Highway 17 Twinning Project which extends from Scheel Drive westerly to 3 km west of Bruce Street in the Renfrew area.

This report addresses the proposed culvert located at Station 10+890 McNab/Braeside Township near Renfrew, Ontario. The existing Highway 17 alignment will become the future Highway 17 eastbound lanes and new westbound lanes will be constructed. This new culvert is required to convey an unnamed tributary of Bonnechere River below an embankment supporting the proposed Highway 17 westbound lanes.

This section of the report presents the factual findings obtained from a foundation investigation completed at the future culvert structure at Station 10+890. Thurber carried out the investigation under Ministry of Transportation (MTO) Assignment No. 4018-E-0009.

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

No previous foundation investigation information for the subject culvert was available within the online Geocres Library.

2 SITE DESCRIPTION

2.1 General

The site is currently undeveloped and located approximately 80 m north of the existing Highway 17 alignment and 800 m west of Goshen Road. For project purposes, Highway 17 is herein described as oriented east-west.



The land use adjacent to the site generally consists of undeveloped, forested and marshy areas with nearby agriculture. The terrain is relatively flat with a minimal downward slope towards the small, unnamed tributary of the Bonnechere River. The ground surface at the proposed culvert location was wet and covered with tall grass. At the time of the field investigation, the creek was within a channel with a width of approximately 1.5 to 2.0 m and a water depth of approximately 0.3 to 0.4 m. The topography suggests flow through the new culvert would be from south to north.

The existing Highway 17 south of the site consists of a two-lane undivided highway with gravel shoulders and a posted speed limit of 90 km/hr. The AADT for this existing section of Highway 17 near the site had a reported AADT of 13,200 in 2012.

Photographs showing the existing conditions in the area of the site at the time of the field investigation are included in Appendix D for reference.

2.2 Site Geology

Based on published geological information in *The Physiography of Southern Ontario* by Chapman and Putnam (1984), the site lies within the physiographic region known as the Ottawa Valley Shallow Till and Rock Ridges. Surficial mapping by Ontario Geological Survey (OGS) indicates the site to be comprised of either coarse-textured glaciomarine deposits, organic deposits or Precambrian bedrock. Base mapping by the OGS indicates the bedrock in the area is early felsic plutonic rock consisting of granodiorite, tonalite, monzogranite, syenogranite, derived gneisses and migmatites.

3 SITE INVESTIGATION AND FIELD TESTING

The current site investigation and field-testing program was carried out between October 2nd to 7th, 2019. The field investigation consisted of advancing 3 boreholes identified as Boreholes CV-16 through CV-18. Prior to commencement of drilling, utility clearances were obtained in the vicinity of the borehole locations.

The locations of the boreholes were surveyed by Thurber for both location and elevation with a Trimble Catalyst DA1 antenna with centimeter accuracy. The northing, easting and elevation of the boreholes are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A, the individual Record of Borehole sheets in Appendix B, and in Table 3-1 below. The site is located within MTM Zone 9.

The current investigation was carried out using a track-mounted CME 45 drill rig equipped with hollow-stem augers and rotary diamond drilling equipment.

Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). In situ vane shear testing was completed in cohesive soils with an MTO 'N' sized vane. Bedrock was cored following ASTM Standard D6032-08 in all boreholes with NQ size coring equipment. Bedrock core samples were stored in core boxes for transport.

Table 3-1: Borehole Summary

Borehole No.	Drilled Location	Northing (Latitude)	Easting (Longitude)	Ground Surface Elevation (m)	Termination Depth (m)
CV-16	Proposed Culvert Inlet	5 034 110.1 (45.446883)	297 393.7 (-76.594681)	157.4	5.6
CV-17	Proposed HWY17 WBL C/L	5 034 122.9 (45.446998)	297 395.0 (-76.594665)	157.3	5.8
CV-18	Proposed Culvert Outlet	5 034 137.4 (45.447128)	297 409.1 (-76.594485)	157.3	5.1

A standpipe piezometer, 19 mm in diameter, was installed in Boreholes CV-16. The installation details are illustrated on the respective Record of Borehole sheets provided in Appendix B. The boreholes were backfilled in accordance with MOE requirements (O.Reg 903, as amended). The piezometer will be decommissioned by Thurber following its use as part of a hydrogeological study.

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

4 LABORATORY TESTING

Laboratory testing was selected in accordance with the current MTO Guideline for Foundation Engineering Services, Section 5. Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples. At least 25% of the recovered soil samples were subjected to grain size distribution analysis and Atterberg limits tests, where appropriate. The testing was carried out to MTO and ASTM standards. Chemical analysis for determination of pH, conductivity, resistivity, sulphide, sulphate and chloride was carried out on one soil sample from Borehole CV-16. All cored bedrock was logged and total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) was measured on each core.

The results of the geotechnical tests are summarized on the Record of Borehole sheets included in Appendix B and all laboratory results are presented on the figures included in Appendix C.

5 GENERAL DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix B and the Borehole Location and Soil Strata Drawing included in Appendix A. A general description of the stratigraphy based on the conditions encountered in the boreholes is given in the following sections. However, the factual data presented on the Borehole Records takes precedence over the Soil Strata Drawing and the general description. It must be



recognized that the soil, rock and groundwater conditions may vary between and beyond borehole locations. Soil classification is in accordance with ASTM D2487. Cohesive soils are described per current MTO protocols.

In general terms, the site was found to consist of a thin veneer of organics at surface over deposits consisting of sand to silty sand and clayey silt over gneiss bedrock.

5.1 Organic Silt

A thin veneer of organic material consisting of an organic silt with varying amounts of gravel was encountered at surface in Boreholes CV-16 and CV-18 with a thickness of 25 mm and 0.8 m, respectively.

One SPT test conducted within the organic unit gave an N-value of 6 indicating a loose relative density. The moisture content of one sample tested was 95%.

5.2 Sand to Silty Sand (SM) with gravel

A native deposit of sand to silty sand with varying amounts of gravel was encountered below the organic material in Boreholes CV-16 and CV-18 and at surface in Borehole CV-17. Wood fragments were observed in the upper portion of the sand unit in Borehole CV-17. Frequent cobbles and boulders were noted below a depth of 0.8 m (elev. 156.6 and 156.5 m) in Boreholes CV-16 and CV-17, respectively, and throughout the deposit in Borehole CV-18. The layer ranged in thickness from 0.7 to 2.5 m and extended to a base elevation ranging from 154.8 to 155.8.

SPT tests conducted in this layer gave N-values ranging from 3 to greater than 100 blows for 178 mm of penetration indicating a relative density ranging from very loose to very dense, however, this refusal represents the bedrock surface rather than the relative density of the soil deposit. The soil generally ranges from very loose to dense in relative density.

The moisture content of this unit generally ranged from 14 to 55% with one test reading of 1% from a sample consisting of primarily broken boulder fragments. The results of grain size distribution testing carried out on one sample of this material indicate the material consists of approximately 33% gravel, 45% sand and 22% fines, as illustrated on Figure C1 in Appendix C.

5.3 Clayey Silt (CL)

A glaciomarine native deposit of clayey silt was encountered below the sand deposit in Borehole CV-16. The deposit extended to a depth of 2.8 m with an underside elevation of 154.6 m.

SPT tests conducted within this cohesive unit gave N-values of weight of hammer and 35. An in situ shear vane test in the clayey silt indicated an undrained shear strength of approximately 21 kPa indicating a soft consistency. Sensitivity was measured to be 3.3.

The moisture content of the clayey silt samples tested were 28 and 33%. The results of one grain size analysis tests conducted on a sample of this material indicated the material to consist of



approximately 0% gravel, 3% sand, 63% silt and 34% clay, as illustrated on Figures C2 in Appendix C.

The results of Atterberg Limits testing carried out on one sample of this material indicate the material to have a liquid Limit of 26, Plasticity Limit of 15 and a Plasticity Index of 11, as illustrated on Figure C3 in Appendix C. The laboratory results indicate that the material to have low plasticity (CL). It should be noted in accordance with the MTO Guideline for Foundation Engineering Services (May 2019) the cohesive deposit could be described as a “clayey silt”.

5.4 Bedrock

Bedrock was proven by coring in all boreholes. The bedrock encountered consisted of slightly weathered to fresh, strong, foliated gneiss to granitic gneiss ranging in colours of red, pink, grey and black. Clay infilled fractures were noted in Run 2 and 3 between a depth of approximately 3.0 to 4.7 m (elev. 154.4 to 15.27 m) in Borehole CV-16. Photographs of the bedrock cores are provided in Appendix C. The following table summarizes the rock core quality:

Summary of Rock Core Quality

Parameter	Range	Average
Total Core Recovery (TCR), %	43 – 100	87
Solid Core Recovery (SCR), %	15 – 100	68
Rock Quality Designation (RQD), %	0 – 100	45
Fracture Index	1 to >10	-

Based on the RQD value range, the bedrock is classified as poor to excellent quality. Based on field observations, the bedrock is strong.

A summary of the bedrock surface information is provided in Table 5-1 below:

Table 5-1: Summary of Bedrock Depth/Elevation

Borehole No.	Depth to Bedrock Surface (mbgs)	Bedrock Surface Elevation (m)
CV-16	2.8	154.6
CV-17	2.5	154.8
CV-18	1.5	155.8

5.5 Groundwater

One standpipe piezometer with a diameter of 19 mm was installed in Borehole CV-16. Groundwater levels recorded in the piezometer are presented in Table 5-2 below:

Table 5-2: Summary of Groundwater Levels

Borehole No.	Bottom of Screen Elevation (m)	Depth (m)	Groundwater Elevation (m)	Date of Measurement
CV-16	154.8	0.1 above g.s.*	157.5	2019.11.26
		0.2 above g.s.*	157.6	2020.04.21

*g.s. = ground surface

Due to the relatively thin soil strata encountered at the site, the piezometer screen was installed across the interface between the sand and underlying clayey silt and therefore represents a generalized indication of the groundwater level in the subsoils at the site.

At the time of the field investigation, the creek water level was measured at an approximate elevation of 157.0 m.

These observations are considered short term and it should be noted that the groundwater level at the time of construction may be different and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation.

5.6 Analytical Testing

One sample of the native sand was submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate, sulphide and chloride concentrations, resistivity and conductivity. The analysis results are summarized in Table 5-3. Copies of the test results are provided in Appendix C.

Table 5-3: Results of Chemical Analysis

Borehole	CV-16
Sample	SS1
Depth (m)	0 – 0.6
Chloride (µg/g)	24
Sulphate (µg/g)	6
Sulphide (%)	< 0.02
pH (-)	7.55
Resistivity (Ohm-cm)	8,320
Conductivity (µS/cm)	120



6 MISCELLANEOUS

Borehole locations were selected by Thurber relative to existing site features. The as-drilled locations and ground surface elevation of the boreholes were surveyed by Thurber following completion of the field program. The elevation survey was carried out with reference to geodetic elevation benchmarks provided by the MTO.

Eastern Ontario Diamond Drilling of Hawkesbury, Ontario supplied and operated the drilling equipment and carried out the drilling, soil sampling, in-situ testing, piezometer installation and borehole decommissioning. The field investigation was supervised on a full-time basis by Michel Johnston of Thurber. Overall supervision of the investigation program was provided by Justin Gray, P.Eng.

Routine geotechnical laboratory testing was completed by Thurber's laboratory in Ottawa, Ontario. Analytical testing was completed by Paracel Laboratories in Ottawa.

Overall project management and direction of the field program was provided by Fred Griffiths, P.Eng. Interpretation of the factual data and preparation of this report were carried out by Katya Edney, P.Eng., Matt Kennedy, P.Eng., and Fred Griffiths, P.Eng. The report was reviewed by P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This section of the report provides an interpretation of the factual data from Part 1 of this report and presents geotechnical recommendations to assist the project team in designing a proposed culvert crossing to be located at Station 10+890 along the proposed Highway 17 realignment. The site is located 800 m west of Goshen Road in McNab/Braeside Township, Renfrew County. At the site, the existing Highway 17 will become the future eastbound lanes while the new future westbound alignment will be located approximately 80 m north of the existing alignment.

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. Contractors 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 following sections provide geotechnical recommendations for the construction of foundation elements for the proposed culvert. The discussions and recommendations presented in this report are based on the information provided by the Ministry of Transportation of Ontario (MTO) and on the factual data obtained during the course of this investigation.

The proposed culvert is required to convey an unnamed tributary of the Bonnechere River from south to north under an embankment supporting the proposed Highway 17 westbound lanes. The existing elevation at site is approximately 157.3 m and the proposed final grade of Highway 17 at St. 10+890 is an approximate elevation of 159.8 m. At the time of the field investigation, the creek consisted of a channel width of approximately 1.5 to 2.0 m and a water depth of approximately 0.3 to 0.4 m.

The site was found to be underlain by a thin veneer of organics at surface over deposits consisting of sand to silty sand and clayey silt over gneiss bedrock at shallow depth. It is noted that the water



level in the standpipe piezometer was at an elevation of 157.6 m on April 21, 2020 and the creek water level was measured at an approximate elevation of 157.0 m.

No previous foundation investigation information for the subject culvert was available within the online Geocres Library.

7.1 Proposed Structure

For project purposes, Highway 17 is herein described as oriented east-west.

The 2003 Stormwater Management and Drainage Report by NCE in support of the Preliminary Design Report for this project indicates a culvert is present beneath the existing highway at approximately 10+993. The culvert is described as a 1.6 m by 1.0 m CSPA. The existing culverts in the study area were inspected by WSP in 2019 with Culvert 006700170038 indicated to be present at Station 11+000 on the existing alignment. The culvert was measured as a 1.6 m by 1.05 m SPCSPA with a length of 33.52 m and a depth of cover of 1.2 m. No action was recommended for the existing culvert in the 2019 WSP study.

At the time of preparation of this Foundation Investigation and Design Report, per the Terms of Reference for this assignment, the proposed structure for the westbound lanes will consist of a concrete, closed-bottom, box culvert (CBC) with a width of 3.0 m and a height of 1.5 m. It is anticipated that the invert will be near the existing ground surface elevation of 157.3 m.

7.2 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6-19.

In accordance with CHBDC, the analysis and design of the structure takes into consideration the importance of the structure and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which in this case is the Ministry of Transportation, Ontario (MTO).

It is understood that the new culvert structure is being designed to the “Major Route” importance category.

This project has been assigned Typical Consequence Classification, in accordance with Section 6.5.1 of the CHBDC. Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances.

The degree of site and prediction model understanding for this site has been assessed to be typical understanding (Section 6.5.3 of CHBDC).

8 SEISMIC CONSIDERATIONS

8.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). The seismic hazard for this site has been obtained from the GSC online calculator for a nearby culvert site at Station 14+660. The data includes a peak ground acceleration (PGA), peak ground velocity (PGV) and the 5% spectral response acceleration values ($S_a(T)$) for the *reference* ground condition (Site Class C) for a range of periods (T) and for a range of return periods including 475-year, 975-year and 2475-year events. The GSC seismic hazard calculated data sheet for this site is included in Appendix F.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA). The PGA at this site for a *reference* Site Class C with a 2% probability of exceedance in 50 years (2475-year event) is 0.227g. This value is to be scaled by the $F(PGA)$ based on the site-specific Site Class.

8.2 CHBDC Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy below the foundations.

Based on the average undrained shear strengths measured below the anticipated culvert foundation elevation, the site is classified as a Seismic Site Class C in accordance with Table 4.1 of the CHBDC. As per Table 4.8 of the CHBDC, Site Class C yields a PGA_{ref} of 0.182 and $F(PGA)$ of 1.0 for the site. These values give a factored PGA of 0.227g.

8.3 Seismic Liquefaction Potential

The susceptibility of the cohesive soils encountered at this site to experience liquefaction/cyclic softening was assessed following the Boulanger and Idriss (2007)ⁱ criteria using measured undrained shear strengths. The results of the analysis indicate the organic silt is susceptible to cyclic mobility. Removal of this material from within the footprint of the embankment is recommended. It should be replaced with compacted granular fill.

The susceptibility of the cohesionless soils at the site to experience liquefaction was assessed using the SPT data following the simplified method for cohesionless soil as outlined in Boulanger and Idriss (2014)ⁱⁱ. Upon completion of dewatering as outlined in Section 11.3, treatment of the very loose to loose sandy material within the footprint of the embankment is recommended. It should be compacted in place to 92% of Standard Proctor maximum dry density.

9 STRUCTURE FOUNDATION ALTERNATIVES

9.1 Foundation Alternatives

Selection of the culvert type must consider the proposed construction procedures, staging requirements, geotechnical resistance available in the foundation soils, depth to suitable bearing stratum and post-construction settlement criteria. From a geotechnical perspective, the following culvert types were considered:

- Closed-Bottom Culvert (Box)

A precast, segmental, closed-bottom, box culvert is considered a feasible option from a foundation engineering perspective. Precast sections, rather than cast-in-place construction, can be installed expediently with less potential for disturbance of the founding soils during installation.

- Circular Pipes (Concrete, HDPE, Steel)

A pipe culvert is a technically feasible alternative from a foundation engineering perspective, but since the proposed pipe must meet the required flow capacity and hydraulic requirements, this option may not be feasible from an overall design standpoint given the limited cover which would be available.

- Open-Bottom Culvert (Box, Arch)

The construction of an open-bottom culvert will have greater construction concerns due to the high water table, artesian conditions and requirement for greater excavation depths to construct the culvert footings to satisfy frost depth requirements. The use of an open-bottom culvert would require greater dewatering efforts when compared to other culvert options.

A comparison of these alternatives, based on their respective advantages and disadvantages, is included in Appendix F. It is not considered practical to support a culvert on deep foundations at this site and therefore this option is not presented in this report.

9.2 Construction Methodology

At the time of the field investigation, the creek consisted of a channel width of approximately 1.5 to 2.0 m and a water depth of approximately 0.3 to 0.4 m. Excavations will likely extend below the water level of the creek. An adequate and effective dewatering plan including surface water management, cofferdams, creek diversion and excavation dewatering will be required to enable excavation to the required founding elevation and construction of the foundations in the dry (See Section 11.3).

At the time of preparation of this report, a construction staging plan has not yet been developed. The foundation recommendations presented herein have been prepared based on the

assumption that construction of the culvert will be carried out during the construction of the new westbound Highway 17 alignment with no requirement for temporary roadway protection. Given the anticipated settlement from the new embankment, a preload period should be anticipated.

9.3 Recommended Approach

A closed-bottom, box culvert is recommended at this site. It is anticipated that construction for the westbound lanes would be carried out while traffic remains on the existing alignment. A preload period, three months in length, would be required prior to carrying out the open cut excavation for the culvert installation (See Section 10.5.2). A temporary culvert will be required during the preload period.

A pipe culvert would also be considered a feasible alternative. Construction staging would be similar to that for the closed-bottom box culvert option.

10 FOUNDATION DESIGN RECOMMENDATIONS

From a foundation engineering perspective, a closed-bottom, concrete, box culvert is recommended.

• Proposed top of pavement westbound Highway 17	159.8 m
• Assumed invert of proposed culvert	157.3 m
• Proposed elevation of base of concrete box	157.0 m
• Creek level, at time of site visit	157.0 m
• Groundwater elevation, at time of site visit	157.6 m
• Bedrock Surface	154.6 to 155.8 m

10.1 Culvert Foundation Bearing Resistances

It is anticipated that the subgrade soils within the culvert footprint will be subjected to the additional loads from the proposed embankment with a height of approximately 2.5 m. Further discussion on the potential settlement of the southern cohesive subgrade soils is provided in Section 10.5. The subgrade should be prepared as described in Section 10.2.

The recommended geotechnical resistances for a 3.0 to 4.0 wide (exterior) pre-cast closed-bottom, box culvert with the underside of culvert base slab at or below approximate elevation 157.0 m, installed on a bedding layer with a minimum thickness of 0.3 m placed on a prepared subgrade are as follows:

- Factored Geotechnical Resistance at ULS of 165 kPa
- Factored Geotechnical Resistance at SLS of 100 kPa

The factored geotechnical resistances include the following factors:



- Consequence factor (Ψ) of 1.0 (as per CHBDC Table 6.1)
- Geotechnical resistance factors (as per CHBDC Table 6.2):
 - $\phi_{gu} = 0.5$ (static analysis; typical degree of understanding)
 - $\phi_{gs} = 0.8$ (static analysis; typical degree of understanding)

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be reduced in accordance with CHBDC Clause 6.10.3 and Clause 6.10.4. Foundation settlement, based on the supplied SLS resistance, is expected to be less than 25 mm for culverts constructed on subgrades prepared with good workmanship and in accordance with Section 10.2 below.

Resistance to lateral forces/sliding resistance between the precast concrete and the underlying Granular 'A' bedding (Section 10.3) should be evaluated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.45. A reduction factor of 0.8 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the culvert and Granular A. An unfactored coefficient of friction of 0.35 can be assumed for the interface between the Granular 'A' and the clay. A reduction factor of 0.6 (as per CHBDC Table 6.2) should be used to estimate the sliding resistance between the Granular A and the clay subgrade.

Surface water diversion and dewatering will be required to place the bedding material and install the culvert in the dry (Section 11.3).

10.2 Subgrade Preparation, Bedding and Backfilling

All organics, soft or loose deposits, disturbed soils, alluvial deposits and deleterious materials must be stripped from the footprint of the culvert foundation to expose competent native subgrade material at or below the desired founding elevations. Organic silt with a thickness of approximately 25 mm and 800 mm was encountered near the south and north ends of the proposed culvert during the drilling investigation and must be removed from beneath the embankment footprint. Upon completion of dewatering as outlined in Section 11.3, the surface of the very loose to loose sandy material within the embankment should be compacted in place to 92% of Standard Proctor maximum dry density. The bearing resistances provided above assume that organic material is replaced and the very loose to loose sand deposits, where encountered at the subgrade level within the culvert footprint are compacted appropriately. The subexcavation should extend down to the competent, native inorganic soils and should be backfilled with well-compacted Granular A, as required.

Any soft or organic materials at the subgrade level should be sub-excavated and replaced with granular fill consisting of OPSS.PROV 1010 Granular A material, compacted as per OPSS.PROV 501, as soon as practical to protect the subgrade from disturbance during construction. In order to provide a more uniform foundation subgrade condition for the culvert, a minimum 300 mm thick layer of well compacted bedding material conforming to OPSS.PROV 1010 Granular A requirements must be provided under the base of the culvert as per OPSS 422 and OPSD 803.010.



Protection of the subgrade should include installation of a Class II, non-woven geotextile with a maximum FOS of 150 μm (OPSS 1860) installed beneath the Granular A material. The geotextile should be placed as soon as possible after preparation of the final subgrade level and in accordance with OPSS.PROV 902. An NSSP is provided in Appendix H to include in the contract documents to alert the Contractor of the potential for encountering sensitive foundation soils at the site.

Compaction must be carried out under dry and stable conditions. It is noted that construction will extend below groundwater elevation. Creek diversion and dewatering will be required to prepare the subgrade in the dry. Please refer to Section 11.3 for additional comments on groundwater and surface water control.

Structural backfill adjacent to the culvert should consist of OPSS Granular A or Granular B Type II meeting the specifications of OPSS.PROV 1010 and SP110S06, placed and compacted in accordance with OPSS.PROV 501. Heavy compaction equipment used adjacent to the culvert must be restricted in accordance with OPSS.PROV 501. Care must be exercised when compacting the fill adjacent to and above the culvert in order not to damage the culvert.

10.3 Frost Penetration Depth

The depth of frost penetration at this site is estimated to be 1.9 m (as per OPSD 3090.101); shallow foundations should be founded at or below this depth or provided with equivalent insulation. Closed-bottom box culverts are not typically provided with frost protection.

Please refer to the pavement design report for frost taper recommendations for the pavement.

10.4 Backfill and Lateral Earth Pressures

Structural backfill material should consist of Granular A or Granular B Type II meeting the OPSS.PROV 1010 and SP110S06 specifications. Large scale direct shear box testing on samples of Granular A and Granular B Type II from numerous nearby aggregate sources was completed for this project. The results indicate that for design of structural backfill for this project, an internal angle of friction of 40 degrees can be used for Granular B Type II and quarry-sourced Granular A in this area provided the vertical pressure on the material is less than 150 kPa (Geocres Memorandum 31F-213). An Operational Constraint will be required in the contract restricting the source of Granular A to quarries (see Appendix H). Throughout this report, the term "Granular A" is defined as "Quarry-Source Granular A" unless specifically described as "Pit-Source Granular A".

The backfill must be in accordance with OPSS 902 and placed to the extents shown on OPSD 3101.150. The backfill should be compacted and compaction equipment to be used adjacent to the structure must be restricted in accordance with OPSS.PROV 501.

The design of the walls should incorporate a subdrain as shown in OPSD 3101.150.



Lateral earth pressure parameters provided in Table 10-1 and Table 10-2 in the sections below are based on the assumptions that the wall is vertical and the backfill is fully drained so that there are no unbalanced hydrostatic pressures above the permanent groundwater level. If adequate drainage cannot be confirmed, the potential for buildup of unbalanced hydrostatic pressures should be considered in design.

Where walls are vertical and back slopes are horizontal, the corresponding coefficients provided in Table 10-1 and Table 10-2 should be used. For other backfill and wall geometries, Thurber will need to calculate the appropriate earth pressure coefficients once the final geometry is confirmed.

10.4.1 Static Lateral Earth Pressure

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC. Under drained conditions the lateral earth pressure is generally given by the following expression:

$$\sigma_h = K * (\gamma h + q)$$

where:

σ_h	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see table below) (K_a for yielding walls, K_o for non-yielding walls)
γ	=	unit weight of retained soil (see table below), use submerged unit weight below groundwater level
h	=	depth below top of fill where pressure is computed (m)
q	=	value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC. Typical earth pressure coefficients for quarry-sourced OPSS Granular A and OPSS Granular B Type II backfill are shown in Table 10-1.

Table 10-1: Static Earth Pressure Coefficients

Condition	Quarry Sourced OPSS Granular A and Granular B Type II $\phi = 40^\circ, \gamma = 22.8 \text{ kN/m}^3$	Pit Sourced OPSS Granular A $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.36	0.43
Coefficient of Active Earth Pressure, K_A (Unrestrained Wall)	0.22	0.27

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. The movement required can be assessed from Table C6.12 of the Commentary to the CHBDC. Active earth pressures should be used for any wingwalls or unrestrained walls. For rigid structures, at-rest horizontal earth pressures would apply for design.

10.4.2 Combined Static and Seismic Lateral Earth Pressure

In accordance with Clause 6.14 of the CHBDC, retaining structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. The following recommendations are per Section C6.14 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with:

- $k_h = \frac{1}{2} * F(PGA) * PGA$, for structures that allow 25 to 50 mm of movement, and
- $k_h = F(PGA) * PGA$, for non-yielding walls

The coefficients of horizontal earth pressure for combined static and seismic loading presented in Table 10-2 may be used. The provided earth pressure coefficients are based on a Seismic Site Class C and a PGA with a 2% probability of exceedance in 50 years of 0.227g (Geological Survey of Canada – Fifth Generation) and a $F(PGA)$ of 1.0 as per Table 4.8 of the CHBDC.

Table 10-2: Combined Static and Seismic Earth Pressure Coefficients

Condition	Quarry Sourced OPSS Granular A and Granular B Type II	Pit Sourced OPSS Granular A
	$\phi = 40^\circ, \gamma = 22.8 \text{ kN/m}^3$	$\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.35	0.42
Coefficient of Active Earth Pressure, K_A (Unrestrained Wall)	0.28	0.34

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soils profile.

$$\sigma_h = K * \gamma * d + (K_{AE} - K_A) * \gamma * (H - d)$$

where:

σ_h	=	lateral earth pressure at depth d (kPa)
d	=	depth below the top of the wall (m)
K	=	static earth pressure coefficient (K_A for yielding walls, K_o for non-yielding walls)
γ	=	unit weight of retained soil, use submerged unit weight below groundwater level
K_{AE}	=	combined static and seismic earth pressure coefficient
H	=	total height of the wall (m)

10.5 Embankment Fill

10.5.1 Embankment Stability

As per the typical cross sections for the proposed Highway 17 shown in Figure 7.3, the embankment will consist of rock fill material with a proposed slope of 1.25H:1V. In some locations, slope flattening materials (earth fill) may be placed at 6H:1V on the rock fill slopes. Embankments should be constructed in accordance with OPSS.PROV 206.

Slope stability for proposed embankment of the new Highway 17 westbound lanes was evaluated using GeoStudio 2020 Slope/W software for limit equilibrium analysis. Input parameters for the analysis are based on the SPT N values, undrained shear strength and the results of laboratory testing and are summarized on each of the slope stability model output figures included in Appendix G. The following additional parameters and assumptions were used in the analyses:

- The soil stratigraphy is based on the nearest boreholes. The surficial organic silt layer must be removed and replaced with Granular A from beneath the embankment footprint and to 2 m past the embankment toe.
- The maximum fill height is approximately 2.5 m at Station 10+890.
- The recommendations provided herein are based on the strength parameters of quarry source Granular A material.
- Side slopes of 1.25H:1V for Rock Fill were analyzed for two scenarios: with and without slope flattening at 6H:1V. Slope flattening will meet the requirements of OPSD 202.010 and 202.020.
- Traffic loading has been included in the analyses by assuming a surcharge of 17 kPa.
- The pavement structure on the embankment will consist of hot mix over a Granular O base layer over a Granular B Type II subbase layer with a total thickness of 540 mm.
- A site adjusted PGA value of 0.114 g, equal to $\frac{1}{2}$ of the site adjusted PGA value (0.227 g), was used for seismic analysis, as per Section 4.4.3.3, of the CHBDC and outlined in Section 8.2 above.



Copies of the output from the stability analyses are provided in Appendix G. Each output figure shows the slope geometry, groundwater conditions, soil stratigraphy and soil strength parameters utilized in the analysis.

The stability analyses generated the following factor of safety values for the proposed embankment design:

Condition	Case	Rock Fill, 1.25H:1V No Slope Flattening	Rock Fill, 1.25H:1V Slope Flattening at 6H:1V
Permanent	Short Term (Undrained)	1.6	2.9
	Long Term (Drained)	1.5	3.0
Temporary (Traffic Loading)	Short Term (Undrained)	1.6	2.7
Temporary (Seismic Loading)	Pseudo-Static (Undrained)	1.3	1.6

Table 6.2 of Section 6.9.1 of the CHBDC requires minimum Factors of Safety of 1.5 and 1.3 for embankments in permanent and temporary conditions respectively for a typical degree of understanding. All of the static results presented above meet or exceed the target Factors of Safety.

Table 6.3 in Section 6.14.4.1 of the CHBDC indicates a minimum seismic resistance factor of 0.95 for force-based design and 1.0 for performance-based design, thus a target Factor of Safety of 1.0 to 1.1 for this temporary condition with a typical degree of understanding is appropriate for the pseudo-static seismic analysis. All of the pseudo-static results presented above, meet or exceed the target Factor of Safety for seismic design. It is noted that some displacement of the embankment can occur where the pseudo-static Factor of Safety is less than 1.3. However, as noted in the table above this criterion has also been satisfied for all cases.

The proposed embankment slopes satisfy all of the static and pseudo-static slope stability requirements.

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankments constructed with slope flattening, see Section 11.4 below.

10.5.2 Embankment Settlement

The future westbound lanes will have a fill height of about 2.5 m adjacent to the proposed culvert. Settlement of the future highway embankment needs to be considered not only in terms of pavement performance on the approaches but also in selection and design of the culvert foundations.

An assessment of the time dependent settlement that would result from construction of the proposed Highway 17 WBL embankment was carried out using Rocscience's Settle^{3D} modelling



software with a Boussinesq stress distribution. The soil stratigraphy was defined based on borehole data at CV-16 and the water table was defined based on piezometer readings. It is noted that engineering judgment and experience were used to select the material properties based on the stress range anticipated due to loading.

The following has been assumed for the embankment geometry:

- Height = 2.5 m
- Length = 200 m
- Platform Width = 13.5 m from outside of rounding to outside of rounding
- Side slopes = 1.25H:1V

It is recommended that slope flattening not be used in the area within 50 m of the culvert to reduce the culvert length and to reduce the magnitude of settlement.

At the proposed embankment consisting of approximately 2.5 m of additional fill, a total settlement of approximately 50 mm is expected to occur in the clayey silt within one year following fill placement, with 95% of the settlement occurring within 1 to 2 months. In areas of the site where the clayey silt is not present, the anticipated embankment settlement would be significantly less and could result in overall differential settlements of up to 40 to 50 mm along the culvert alignment, assuming the subgrade is prepared as recommended above.

The magnitude of the embankment compression constructed with rockfill and granular materials is in the order of 0.5% of the embankment height and is expected to occur following fill placement.

MTO guidelines for settlement of freeway approach embankments within structure transition zones over a period of 20 years after paving is outlined below:

- 25 mm within 20 m of the structure;
- 50 mm from 20 to 50 m from the structure;
- 75 mm from 50 to 75 m from the structure; and
- 100 mm greater than 75 m from the structure.

Based on these guidelines, the maximum total embankment settlement of the westbound embankment exceeds these criteria. Therefore, it is recommended that a full height preload and a temporary CSP culvert be constructed and left in place for a duration of approximately 3 months to ensure that post-construction settlement meets the above guidelines.

Since the consolidation settlement described above will take place in the clayey silt, the preload footprint could be limited to areas underlain by this stratum. However, due to the potential for variability of the thickness of the clayey silt between the boreholes and ease of construction, it is recommended that the full height preload be constructed along the entire culvert alignment to reduce potential for differential settlements and post-construction performance issues.



A settlement monitoring program should be incorporated into the Contract to allow monitoring of the actual rate and magnitude of the embankment settlement and determination of the end of preload.

Following completion of the preload period, the preload material would then be excavated to remove the temporary CSP and construct the permanent highway culvert.

Alternatively, the culvert could be designed with a camber to structurally accommodate the anticipated settlement and/or oversized to allow sufficient hydraulic capacity after undergoing the settlement. In this case a preload period is not required however, a two month delay prior to paving would be recommended.

10.6 Cement Type and Corrosion Potential

Chemical analysis for determination of pH, water soluble sulphate, sulphides, chloride concentrations, resistivity and electrical conductivity was carried out on samples of the native materials. The analysis results are summarized in Section 5.6 and a copy of the test results is provided in Appendix C.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Section 5.6 were compared with Table 3.2 of the MTO Gravity Pipe Design Guideline and indicate a very low corrosive environment. The test results provided in Section 5.6 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with the soil and groundwater at the site. The sulphate results in were compared with Table 3 of Canadian Standards Association Standards A23.1-19 (CSA A23.1) and indicate a low degree of sulphate attack potential on concrete structures at this site.

The corrosive effects of road de-icing salts should also be considered.

11 CONSTRUCTION CONSIDERATIONS

11.1 Temporary Excavations

All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of OHSA, the clayey silt and sandy soils above the groundwater table are considered Type 3. All soils below the groundwater table are considered Type 4. Side slopes for excavations through more than one soil type must be entirely based on the highest soil type number.

Excavations for the footings must be carried out in accordance with OPSS 902 and will extend into the underlying native deposits (sand to silty sand). Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. Stockpiling or surface surcharge should not be allowed on the embankment or side slopes.



Although not anticipated, at locations where there are space restrictions, the excavations could be carried out within a protection system. Further discussion on temporary protection systems (TPS) is presented in Section 11.2.

11.2 Temporary Protection Systems

Although not anticipated, Temporary Protection Systems (TPS) could be used for excavation support or groundwater control, they must be implemented in accordance with OPSS.PROV 539 and designed for Performance Level 2. Given the presence of potential obstructions at this site, installation of interlocking sheet piles may be difficult and, therefore, a soldier pile and lagging system may be preferred. Due to the relatively shallow bedrock, the soldier piles may need to be predrilled and socketed into the underlying bedrock to achieve suitable fixity for the shoring system.

When assessing the design of the roadway protection, the Contractor should consider the potential obstructions such as cobbles and boulders that were encountered during drilling operations as well as the nominal artesian conditions that were noted during groundwater measurements. Lateral support may need to be enhanced by socketing the soldier piles into bedrock and/or by using bracing or rakers. Due to the presence of cobbles and boulders in the site soils, it may be necessary to predrill for the soldier piles. Suggested wording for an NSSP for obstructions is included in Appendix H.

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

Lateral earth pressure coefficients, under fully mobilized conditions, that can be used in design for the structural backfill are provided in Table 10-1. The lateral earth pressure coefficients for the underlying native soils are given below for a vertical wall and a horizontal backslope:

Native clayey silt:

γ	=	17.5 (kN/m ³ bulk unit weight of soil, to be adjusted below water)
K_A	=	0.36
K_P	=	2.8
s_u	=	20 kPa

Native silty sand to sand:

γ	=	18.5 (kN/m ³ bulk unit weight of soil, to be adjusted below water)
K_A	=	0.33
K_P	=	3.0

If the backslope behind, or if the ground surface in front of the temporary protection systems is not horizontal, the lateral earth pressure parameters provided above do not apply and recalculation of the earth pressure parameters will be required.



The design of roadway protection is the responsibility of the Contractor. All protection systems should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to construction equipment and operations.

11.3 Surface and Groundwater Control

Culvert construction, subgrade preparation and placement and compaction of granular bedding must be carried out in the dry; it is recommended that the groundwater be lowered to 0.5 m below the final sub-excavation elevation. Furthermore, surface runoff will tend to seep into and accumulate into the excavations. The Contractor must control groundwater, perched groundwater and surface water flow at the site to permit the replacement of the culvert in a dry and stable excavation.

Subgrade preparation, placement and compaction of granular bedding, and culvert construction must be carried out with a properly designed dewatering system to control groundwater and creek/surface water and may include coffer dams, ditch diversion, pumping etc. The creek diversion should be placed outside the construction area. The dewatering system will be required to remain operational and effective until the temporary excavations are backfilled and then should be decommissioned and removed.

Artesian conditions were encountered during the drilling investigation and must be accounted for during the dewatering operations. Excavation is not to proceed until the groundwater table has been lowered to the recommended 0.5 m below culvert subgrade elevation.

The dewatering system could consist of sump pumps in conjunction with a sandbag cofferdam or a more robust system such as an interlocking sheet piles, depending on the creek flow at the time of construction. If sheet piles are selected, it should be noted that bedrock is shallow and the presence of cobbles and boulders in the granular fill may impeded their penetration.

The design of dewatering systems is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the system in accordance with SP No. FOUN0003 which amends OPSS 902. A preconstruction survey is not considered to be required, thus Designer Fill-In ** in this SP should be "NA".

The water level will fluctuate and the minimum groundwater elevation for the site at the time of the excavation should be taken as the expected high water level defined in SP517F01 and SP FOUN0003. Given the presence of artesian groundwater conditions it is recommended that the dewatering system design engineer requirement be invoked in SP517F01. Excavation base instability due to artesian conditions must be considered for the site. In addition, the potential for bottom heave due to the presence of cohesive soils needs to be assessed.

It is anticipated that filtered sump pumps will likely be sufficient to extract water from the excavation for the culverts. The groundwater level within the work zone should be lowered by pumping from sumps to a minimum of 500 mm below the underside of the planned excavation base prior to each stage of excavation and compaction.



Further assessment of the dewatering requirements and the need for a Permit to take Water (PTTW) should be carried out by specialists experienced in this field.

It is noted that a Hydrogeological Investigation and Design Report is under preparation for the Highway 17 Twinning Project. Please refer to that document for additional discussion on dewatering with respect to this assignment.

11.4 Erosion and Scour Control

The Contractor should provide silt fences and erosion control blankets as per OPSS.PROV 805 throughout the duration of construction to prevent transport of silt/sediment. Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. Slope vegetation should be established as soon as possible after completion of the embankment fills in order to limit surficial erosion.

Typically, rock protection should be provided over all earth surfaces subjected to flowing water in accordance with OPSS.PROV 511. Treatment at the outlet should be in accordance with OPSD 810.010. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

Scour protection shall be provided at the culvert inlet and outlet areas. Design of the erosion and scour protection measures must consider hydrologic and hydraulic factors and shall be carried out by specialists experienced in this field.

To minimize the potential for piping and erosion around the inlet of the new culvert, a clay seal may be used. The clay seal must extend to approximately 300 mm above the high water level and laterally for the width of the granular material, and have a minimum thickness of 500 mm. The material requirements should be in accordance with OPSS.PROV 1205. A geosynthetic clay liner could be considered for use as a clay seal.

Liaison between the Foundations Consultant, Structural Engineer and Hydraulic/Drainage Engineer will be required in design to ensure that scour protections is adequately addressed.

12 CONSTRUCTION CONCERNS

The planned construction methodology includes open cut excavations for the installation of foundation elements of a new culvert. A potential construction concern may include, but is not necessarily limited to:

- Dewatering and control of groundwater seepage during excavation. Minor artesian conditions in the clayey silt layer were noted during groundwater measurements.
- The soils which will be exposed beneath a culvert bedding layer are sensitive and readily disturbed. A suggested Notice to Contractor is provided in Appendix H.

- Mitigation of the settlement induced by the new westbound embankment will require a preload or a structure designed to accommodate the movements. An instrumentation and monitoring program will need to be implemented to assess the progress of the preload. Given the limited project length, the monitoring program would include approximately six settlement rods located on the new alignment with a nominal spacing of 25 m. The base plates should be installed prior to fill placement and the rods will require extension as fill is placed around them. The top of the settlement rods should be surveyed every week during preload construction and every two weeks for the anticipated 3 month preload period. The installation of the monitoring equipment and surveying would typically be carried out by the Contractor, with the results evaluated by the Contract Administration team. The required specifications and drawings will be prepared once project direction concerning the construction schedule and available preload duration has been established.

The successful performance of the structure installations will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations will be required as per OPSS.PROV 902 during construction to confirm that the foundation recommendations are correctly implemented, and material specifications are met.



13 CLOSURE

Engineering analysis and preparation of this report was carried out by Mrs. Katya Edney, P.Eng., Matt Kennedy, P.Eng. and Dr. Fred Griffiths, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundation Projects.

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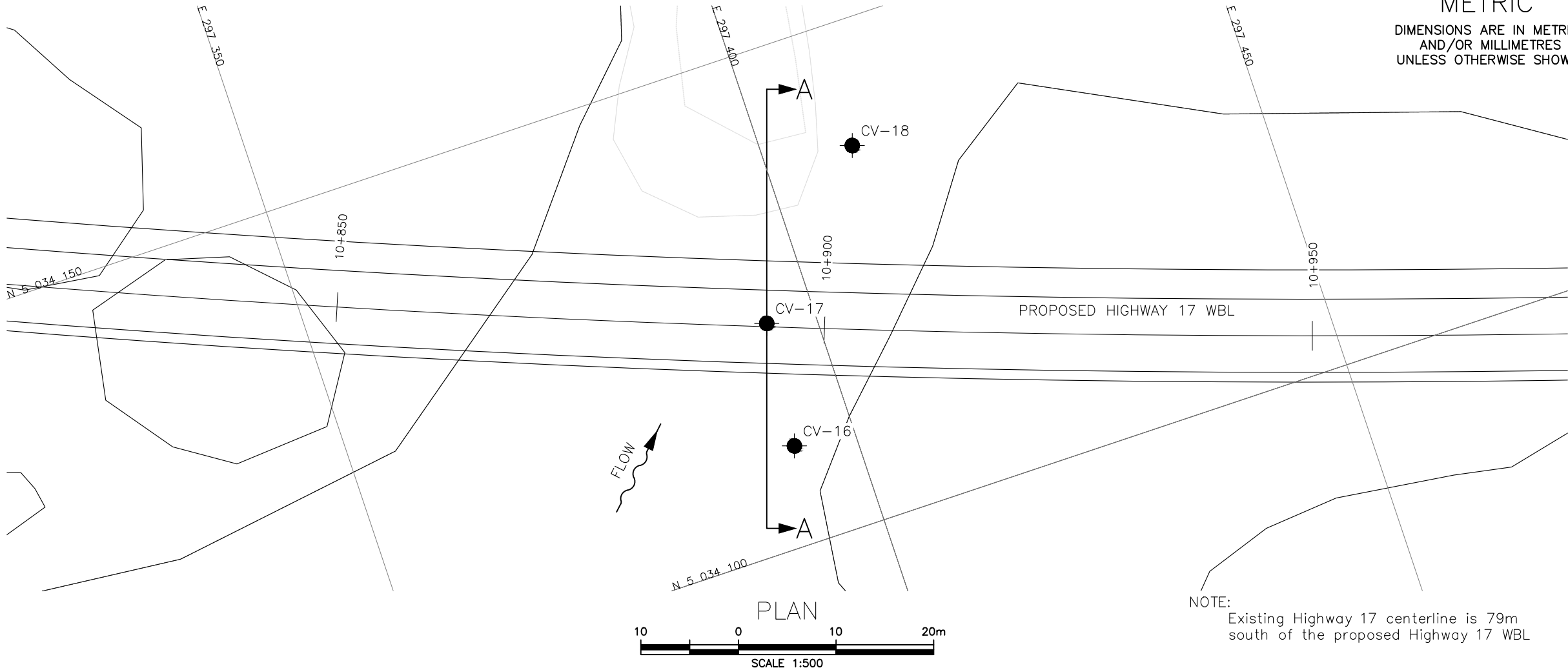
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ⁱ Boulanger, R. W. and Idriss, I. M. (2007). Evaluation of cyclic softening in silts and clays, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, 133(6), 641-652.

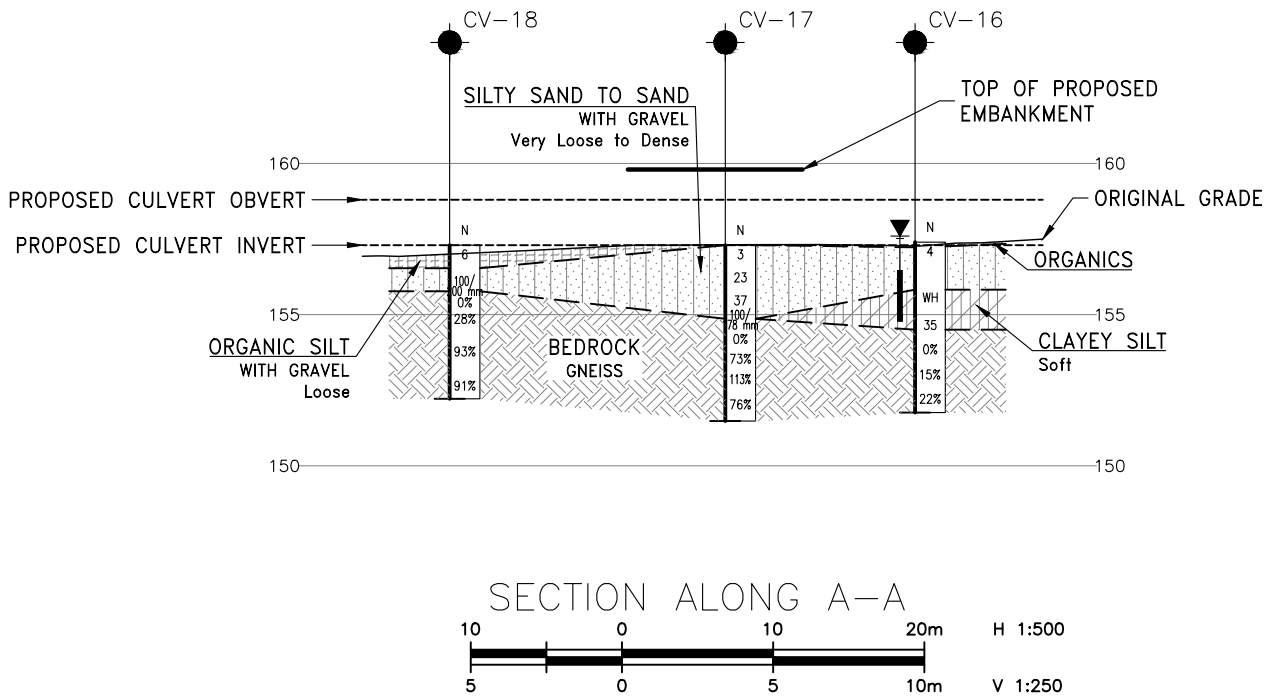
ⁱⁱ Boulanger, R. W., and Idriss, I. M. (2014). CPT and SPT based liquefaction triggering procedures, Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.

Appendix A.

Borehole Location Plan and Stratigraphic Drawings



NOTE:
Existing Highway 17 centerline is 79m
south of the proposed Highway 17 WBL



CONT No
WP No 4068-09-00

HIGHWAY 17 TWINNING
CULVERT STA. 10+890 WBL

BOREHOLE LOCATIONS AND SOIL STRATA

KEYPLAN

LEGEND

●	Borehole
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
CV-16	157.4	5 034 110.1	297 393.7
CV-17	157.3	5 034 122.9	297 395.0
CV-18	157.3	5 034 137.4	297 409.1

-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. Structural elements, surface details and features are for conceptual illustration.

3) Coordinate system is MTM NAD 83 Zone 9.

GEOCRES No. 31F-217

REVISIONS

DATE	BY	DESCRIPTION
DESIGN KE	CHK MJK	CODE
DRAWN AN	CHK KE	SITE

LOAD	DATE	JUL 2021
STRUCT	DWG 1	

FILENAME: H:\Drafting\24000\24726\BHP-Cul Sta 10+890.dwg
PLOTDATE: 7/30/2021 4:01 PM

Appendix B.

Record of Borehole Sheets



SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of fragments of decayed organic matter
Till	unstratified glacial deposit which may include particles ranging in sizes from clay to boulder
Fill	material below the surface identified as placed by humans (excluding buried services)

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

Desiccated	having visible signs of weathering by oxidization of clay materials, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

RECOVERY:

For soil samples, the recovery is recorded as the length of the soil sample recovered.

N-VALUE:

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 63.5 kg hammer falling 0.76 m, required to drive a 50 mm O.D. split spoon sampler 0.3 m into undisturbed soil. For samples where insufficient penetration was achieved and N-value cannot be presented, the number of blows are reported over the sampler penetration in millimetres (e.g. 50/75).

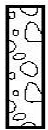
DYNAMIC CONE PENETRATION TEST (DCPT):

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an "A" size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The DCPT is used as a probe to assess soil variability.

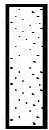


STRATA PLOT:

Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders
Cobbles
Gravel



Sand



Silt



Clay



Organics



Asphalt



Concrete



Fill



Bedrock

TEXTURING CLASSIFICATION OF SOILS

Classification	Particle Size
Boulders	Greater than 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	Less than 0.002 mm

SAMPLE TYPES

SS	Split spoon samples
ST	Shelby tube or thin wall tube
DP	Direct push sample
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ etc.	Rock core sample obtained with the use of standard size diamond coring equipment

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	Undrained Shear Strength (kPa)
Very Soft	12 or less
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

NOTE: Clay sensitivity is defined as the ratio of the undisturbed strength over the remolded strength.

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

Descriptive Term	SPT "N" Value
Very Loose	Less than 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	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	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clayey silts of low plasticity, gravelly clays, sandy clays.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note - W_L = Liquid Limit



EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

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 0.1 m in length or larger, as a percentage of total core length
Unconfined Compressive Strength: (UCS)	Axial stress required to break the specimen.
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

DISCONTINUITY SPACING

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

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)
Extremely Strong	Greater than 250
Very Strong	100 – 250
Strong	50 – 100
Medium Strong	25 – 50
Weak	5 – 25
Very Weak	1 – 5
Extremely Weak	0.25 – 1

RECORD OF BOREHOLE No CV-16

1 OF 1

METRIC

WP# 4068-09-00 LOCATION Lat: 45.446883°, Long: -76.594681°
Culvert 10+890 MTM z9: N 5 034 110.1 E 297 393.7 ORIGINATED BY MJJ
HWY 17 BOREHOLE TYPE CME 45 Trackmount, HSA/NW/NQ Coring COMPILED BY JP
DATUM Geodetic DATE 2019.10.04 - 2019.10.04 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
157.4	Ground Surface															
0.0 0.1	Organics (25 mm)															
	SAND Very Loose to Loose Brown to Grey - frequent cobbles/boulders from 0.8 m to 1.5 m		1	SS	4											
155.8																
1.6	CLAYEY SILT (CL) Soft Grey		2	SS	WH											0 3 63 34
154.6			3	SS	35											
2.8	GNEISS BEDROCK Slightly weathered, red and black, strong, moderately to highly fractured - clay infilled fractures from elev. 154.4 to 152.7 m		1	RUN												RUN #1 TCR=43% SCR=34% RQD=0%
			2	RUN												RUN #2 TCR=100% SCR=52% RQD=15%
			3	RUN												RUN #3 TCR=67% SCR=92% RQD=22%
151.8																
5.6	End of Borehole Monitoring well consists of 19 mm diameter Schedule 40 PVC pipe with a 1.5 m slotted screen WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2019.11.26 0.1 above g.s. 157.5 2020.04.21 0.2 above g.s. 157.6															

DOUBLE LINE 24726 CULVERT 10+890.GPJ 2012TEMPLATE(MTO).GDT 1/5/21

RECORD OF BOREHOLE No CV-17

1 OF 1

METRIC

WP# 4068-09-00 LOCATION Lat: 45.446998°, Long: -76.594665°
Culvert 10+890 MTM z9: N 5 034 122.9 E 297 395.0 ORIGINATED BY MJJ
HWY 17 BOREHOLE TYPE CME 45 Trackmount, HSA/NW/NQ Coring COMPILED BY JP
DATUM Geodetic DATE 2019.10.02 - 2019.10.03 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
157.3	Ground Surface							20	40	60	80	100			
0.0	SAND Very Loose Grey - wood fragments		1	SS	3		157								
156.5															
0.8	SILTY SAND (SM) with gravel Compact to Dense Brown to Grey - frequent cobbles		2	SS	23		156								33 45 22 (SI+CL)
			3	SS	37										
154.8			4	SS	100/		155								
2.5	GNEISS BEDROCK Slightly weathered to fresh, grey, foliated, strong		1	RUN	178 mm		154								RUN #1 TCR=53% SCR=18% RQD=0%
			2	RUN											RUN #2 TCR=100% SCR=96% RQD=73%
			3	RUN			153								RUN #3 TCR=100% SCR=100% RQD=100%
			4	RUN			152								RUN #4 TCR=97% SCR=93% RQD=76%
151.5															
5.8	End of Borehole														

DOUBLE LINE 24726 CULVERT 10+890.GPJ 2012TEMPLATE(MTO).GDT 1/5/21

RECORD OF BOREHOLE No CV-18

1 OF 1

METRIC

WP# 4068-09-00 LOCATION Lat: 45.447128°, Long: -76.594485°
Culvert 10+890 MTM z9: N 5 034 137.4 E 297 409.1 ORIGINATED BY MJJ
HWY 17 BOREHOLE TYPE CME 45 Trackmount, NW/NQ Coring COMPILED BY JP
DATUM Geodetic DATE 2019.10.07 - 2019.10.07 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
157.3	Ground Surface													
0.0	ORGANIC SILT with gravel Loose Black		1	SS	6		157						95	
156.5														
0.8	SILTY SAND (SM) with gravel - frequent cobbles and boulders		2	GS			156							
155.8			3	SS100/200 mm										
1.5	GRANITE/GNEISS BEDROCK Pink and grey, slightly weathered to fresh, strong, some foliation, highly to slightly fractured		1	RUN										
			2	RUN			155							
			3	RUN			154							
			4	RUN			153							
152.2														
5.1	End of Borehole													

DOUBLE LINE 24726 CULVERT 10+890.GPJ 2012TEMPLATE(MTO).GDT 1/5/21

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

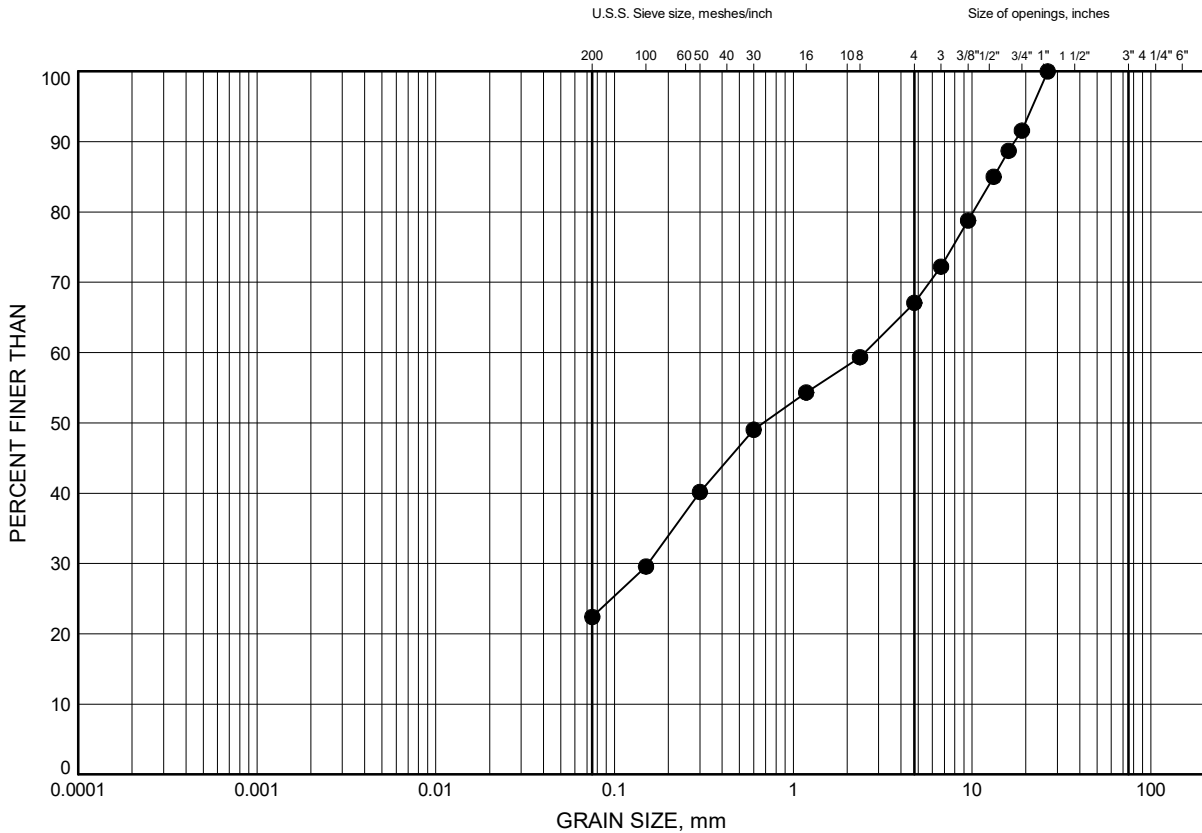
Appendix C.
Laboratory Testing

Appendix C.1
Particle Size Analysis Figures
Atterberg Limit Test Results

Highway 17 Twinning GRAIN SIZE DISTRIBUTION

FIGURE C1

SILTY SAND (SM) with gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CV-17	1.1	156.2

Date June 2020
WP# 4068-09-00

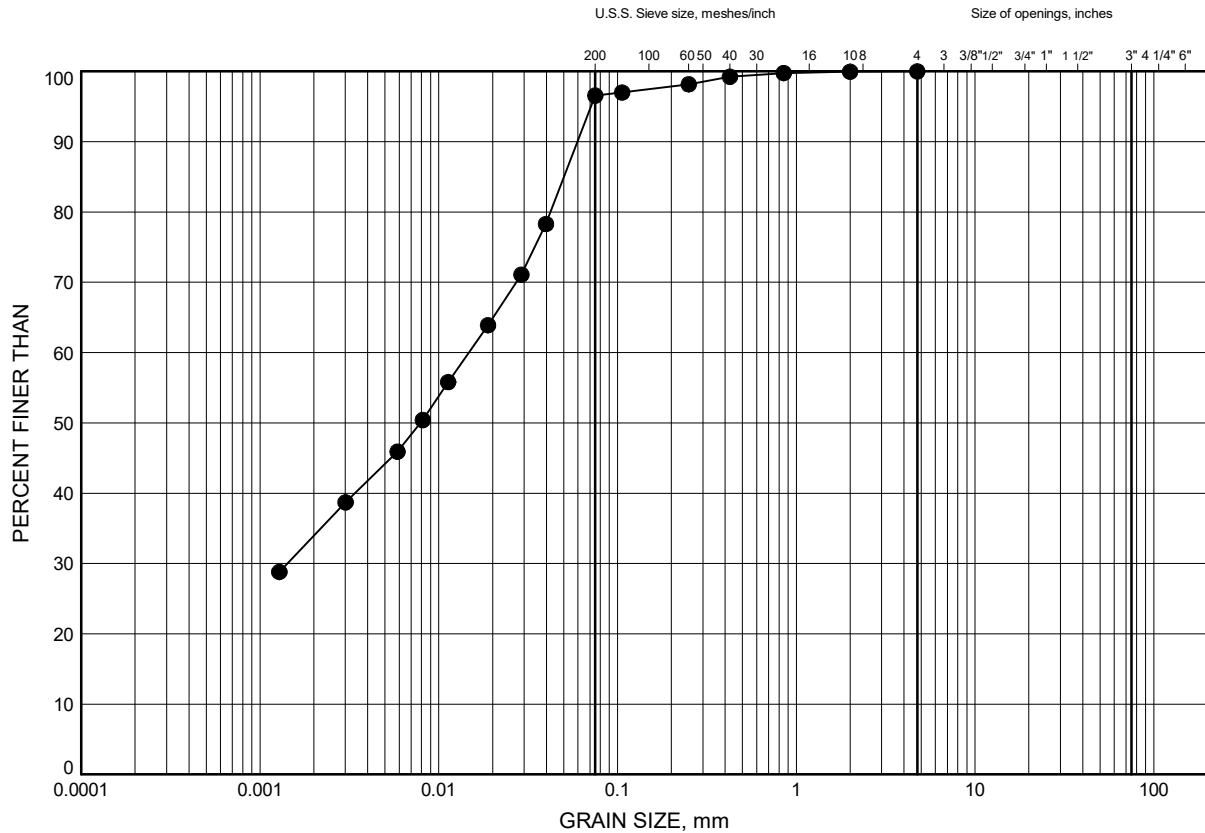


Prep'd KE
Chkd. FG

Highway 17 Twinning GRAIN SIZE DISTRIBUTION

FIGURE C2

CLAYEY SILT (CL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CV-16	1.8	155.6

Date June 2020
WP# 4068-09-00

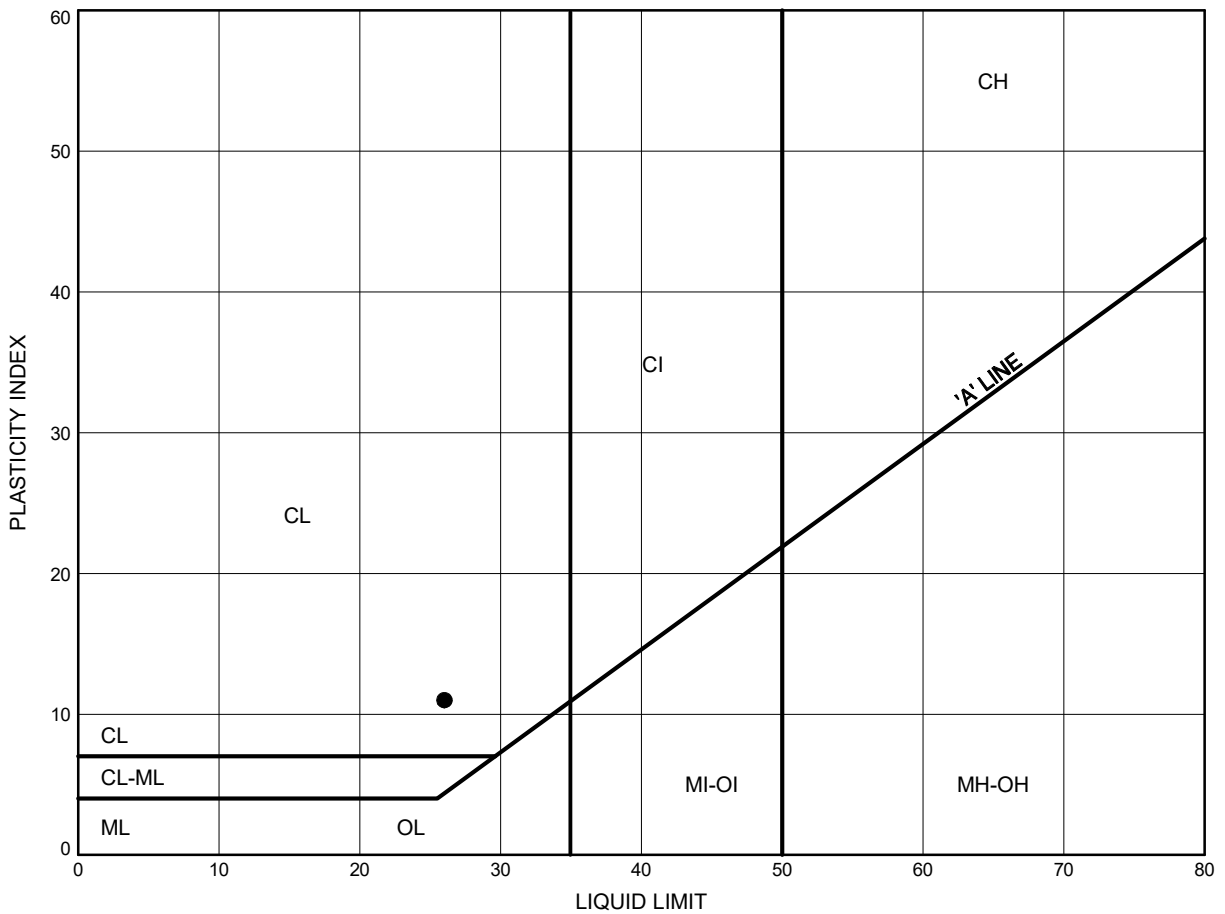


Prep'd KE
Chkd. FG

Highway 17 Twinning ATTERBERG LIMITS TEST RESULTS

FIGURE C3

CLAYEY SILT (CL)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CV-16	1.8	155.6

Date June 2020
 WP# 4068-09-00



Prep'd KE
 Chkd. FG

Appendix C.2
Analytical Testing Results

Certificate of Analysis

Thurber Engineering Ltd.

2460 Lancaster Rd, Suite 104
Ottawa, ON K1B 4S5
Attn: Justin Gray

Client PO: 24726
Project: Hwy 17, Culvert 10+890
Custody: 49176

Report Date: 20-Oct-2019
Order Date: 16-Oct-2019

Order #: 1942186

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID
1942186-01

Client ID
CV-16, SS1, 0'-2'

Approved By:



Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 24726

Report Date: 20-Oct-2019

Order Date: 16-Oct-2019

Project Description: Hwy 17, Culvert 10+890

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	17-Oct-19	17-Oct-19
Conductivity	MOE E3138 - probe @25 °C, water ext	17-Oct-19	18-Oct-19
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	18-Oct-19	19-Oct-19
Resistivity	EPA 120.1 - probe, water extraction	17-Oct-19	18-Oct-19
Solids, %	Gravimetric, calculation	17-Oct-19	17-Oct-19

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 24726

Report Date: 20-Oct-2019

Order Date: 16-Oct-2019

Project Description: Hwy 17, Culvert 10+890

Client ID:	CV-16, SS1, 0'-2'	-	-	-
Sample Date:	04-Oct-19 09:00	-	-	-
Sample ID:	1942186-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	79.0	-	-	-
----------	--------------	------	---	---	---

General Inorganics

Conductivity	5 uS/cm	120	-	-	-
pH	0.05 pH Units	7.55	-	-	-
Resistivity	0.10 Ohm.m	83.2	-	-	-

Anions

Chloride	5 ug/g dry	24	-	-	-
Sulphate	5 ug/g dry	6	-	-	-

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 24726

Report Date: 20-Oct-2019

Order Date: 16-Oct-2019

Project Description: Hwy 17, Culvert 10+890

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Conductivity	ND	5	uS/cm						
Resistivity	ND	0.10	Ohm.m						

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 24726

Report Date: 20-Oct-2019

Order Date: 16-Oct-2019

Project Description: Hwy 17, Culvert 10+890

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	120	5	ug/g dry	123			3.1	20	
Sulphate	323	5	ug/g dry	332			2.8	20	
General Inorganics									
Conductivity	114	5	uS/cm	117			2.6	5	
pH	7.30	0.05	pH Units	7.33			0.4	2.3	
Resistivity	87.7	0.10	Ohm.m	85.5			2.6	20	
Physical Characteristics									
% Solids	87.2	0.1	% by Wt.	87.5			0.4	25	

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 24726

Report Date: 20-Oct-2019

Order Date: 16-Oct-2019

Project Description: Hwy 17, Culvert 10+890

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	215	5	ug/g	123	91.2	82-118			
Sulphate	421	5	ug/g	332	88.8	80-120			

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 24726

Report Date: 20-Oct-2019

Order Date: 16-Oct-2019

Project Description: Hwy 17, Culvert 10+890

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Subcontracted Analysis

Thurber Engineering Ltd.

2460 Lancaster Rd, Suite 104
Ottawa, ON K1B 4S5
Attn: Justin Gray

Tel: (613) 408-6795
Fax: (613) 247-2185

Paracel Report No: **1942186**
Client Project(s): **Hwy 17, Culvert 10+890**
Client PO: **24726**
Reference: **Standing Offer**
CoC Number: **49176**

Order Date: 16-Oct-19
Report Date: 20-Oct-19

Sample(s) from this project were subcontracted for the listed parameters. A copy of the subcontractor's report is attached

Paracel ID	Client ID	Analysis
1942186-01	CV-16, SS1, 0'-2'	Sulphide, solid

**SGS Canada Inc.**

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

Paracel Laboratories

Attn : Dale Robertson

300-2319 St.Laurent Blvd.
Ottawa, ON
K1G 4K6, Canada

Phone: 613-731-9577
Fax:613-731-9064

23-October-2019

Date Rec. : 17 October 2019
LR Report: CA15379-OCT19
Reference: Project#: 1942186

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide %
1: Analysis Start Date		23-Oct-19
2: Analysis Start Time		14:17
3: Analysis Completed Date		23-Oct-19
4: Analysis Completed Time		14:33
5: QC - Blank		< 0.02
6: QC - STD % Recovery		113%
7: QC - DUP % RPD		4%
8: RL		0.02
9: CV-16, SS1, 0'-2'	04-Oct-19	< 0.02

RL - SGS Reporting Limit

Note: Sample was received on last day of 14 day holding time; result may be unreliable.

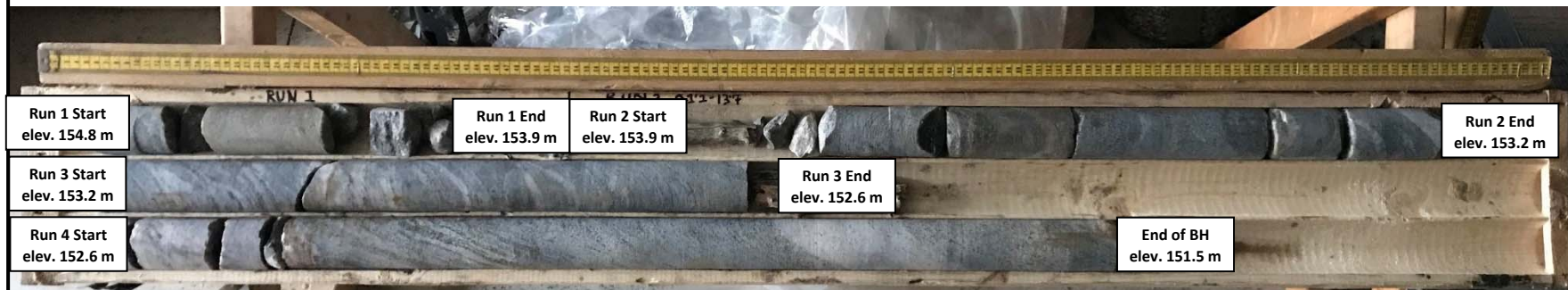
Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

Appendix C.3
Bedrock Photos

Borehole CV-16
Run 1 to 4 (of 4)
Elevation 154.6 m to 151.8 m



Borehole CV-17
Run 1 to 4 (of 4)
Elevation 154.8 m to 151.5 m



THURBER ENGINEERING LTD.

Geotechnical Investigation
HWY 17 Twinning
Renfrew, Ontario

BH CV-17
Project No.: 24726

Borehole CV-18
Run 1 to 3 (of 3)
Elevation 155.8 m to 152.2 m



THURBER ENGINEERING LTD.

Geotechnical Investigation
HWY 17 Twinning
Renfrew, Ontario

BH CV-18
Project No.: 24726

Appendix D.
Site Photographs



Photo 1. Looking southwest on Highway 17 at Station 10+890 (2019/10/02)



Photo 2. Looking south towards existing Highway 17 at Station 10+890 (2019/10/02)

Appendix E.

GSC Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 45.447N 76.598W

User File Reference: Station 10+890

2020-08-07 01:05 UT

Requested by: Thurber Engineering Ltd.

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.356	0.182	0.105	0.032
Sa (0.1)	0.422	0.228	0.137	0.045
Sa (0.2)	0.353	0.197	0.123	0.043
Sa (0.3)	0.268	0.154	0.098	0.035
Sa (0.5)	0.192	0.112	0.072	0.026
Sa (1.0)	0.098	0.059	0.038	0.013
Sa (2.0)	0.048	0.028	0.018	0.005
Sa (5.0)	0.013	0.007	0.004	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.227	0.125	0.076	0.025
PGV (m/s)	0.160	0.090	0.056	0.018

Notes: Spectral ($S_a(T)$, where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information



Natural Resources
Canada

Ressources naturelles
Canada

Canada








Appendix F.

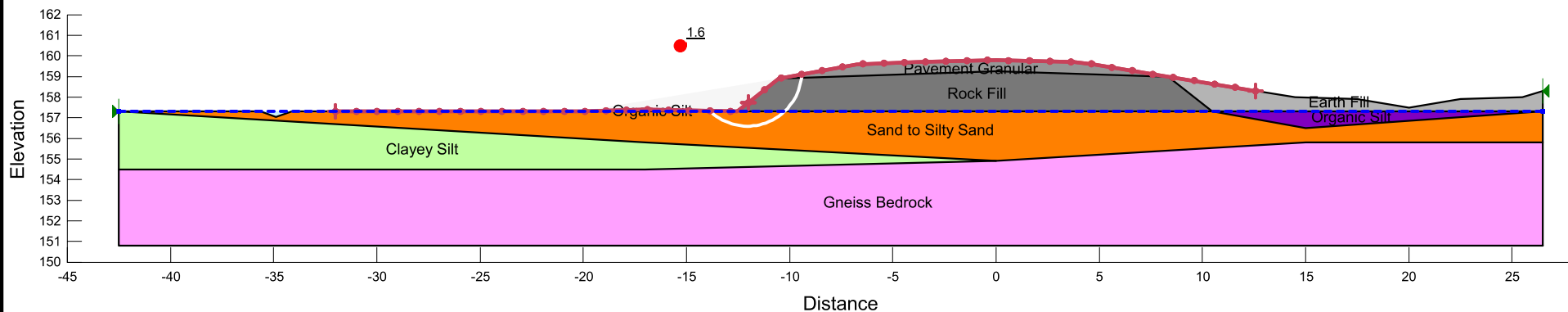
Foundation Comparison

COMPARISON OF ALTERNATIVE FOUNDATION TYPES

Culvert Type	Closed-Bottom Box Culvert	Pipe Culvert	Open-Bottom Culvert
Advantages	<ul style="list-style-type: none"> • Relatively expedient installation if precast units are used. • Smaller magnitude of settlement than open footing culvert due to lower bearing stress on subgrade. 	<ul style="list-style-type: none"> • Relatively expedient pipe installation. • Smaller magnitude of settlement than open footing culvert due to lower bearing stress on subgrade. 	<ul style="list-style-type: none"> • Relatively expedient installation if precast units are used. • Possibility to maintain work zone outside of existing waterway.
Disadvantages	<ul style="list-style-type: none"> • Requires a temporary by-pass to maintain waterflow 	<ul style="list-style-type: none"> • Requires a temporary by-pass to maintain waterflow 	<ul style="list-style-type: none"> • Deepest excavation, increases quantities and dewatering concerns. • Lower geotechnical resistances. • Potential for post construction settlement.
Risks/ Consequences		<ul style="list-style-type: none"> • Design challenges of meeting required flow may require multiple pipes. 	<ul style="list-style-type: none"> • Increased risk of basal instability of footing excavation due to depth of excavation below water table.
Relative Cost	Low to Medium	Low to Medium	Medium
Recommendation	Recommended	Generally Feasible	Not Recommended

Appendix G.
Slope Stability Analysis Figures

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Clayey Silt	Mohr-Coulomb	17.5	20	0
	Earth Fill	Mohr-Coulomb	20	0	30
	Gneiss Bedrock	Bedrock (Impenetrable)			
	Organic Silt	Mohr-Coulomb	16	25	0
	Pavement Granular	Mohr-Coulomb	22	0	35
	Rock Fill	Mohr-Coulomb	20	0	40
	Sand to Silty Sand	Mohr-Coulomb	18.5	0	30



Project
Highway 17 Twinning

Analysis
1A Static (Undrained)

Seismic Coefficient
H: g, V: g








Last Run
07/13/2021, 04:05:05 PM

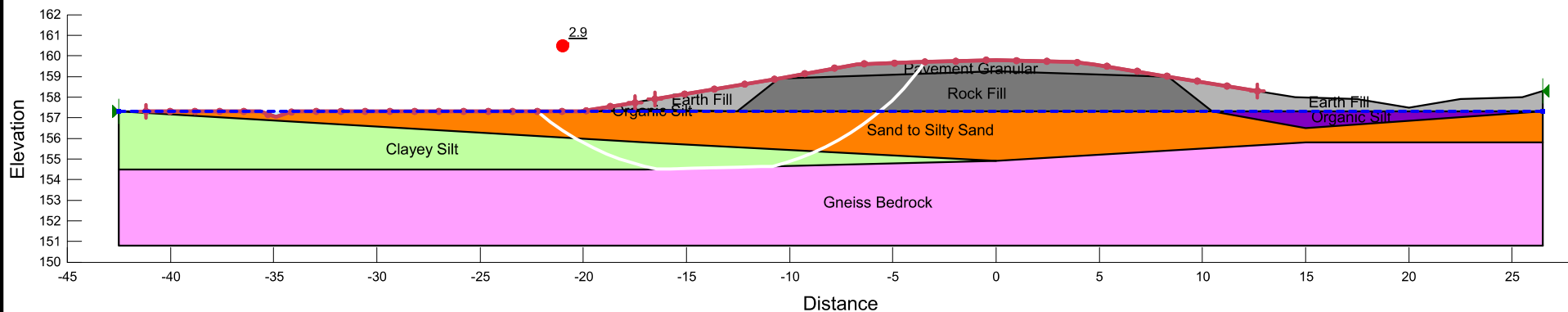
Scale
1:300

Additional Details

Name: Embankment Fill Sta. 10+890
File Name: 10+890 1.25 Rock Fill_20210501.gsz
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m

Fig. G1-1A








Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Clayey Silt	Mohr-Coulomb	17.5	20	0
	Earth Fill	Mohr-Coulomb	20	0	30
	Gneiss Bedrock	Bedrock (Impenetrable)			
	Organic Silt	Mohr-Coulomb	16	25	0
	Pavement Granular	Mohr-Coulomb	22	0	35
	Rock Fill	Mohr-Coulomb	20	0	40
	Sand to Silty Sand	Mohr-Coulomb	18.5	0	30

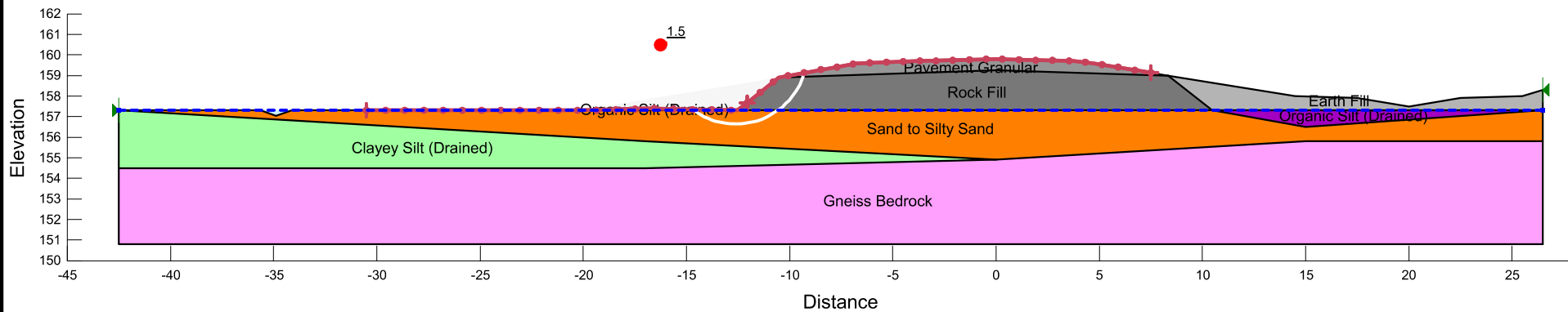


Project Highway 17 Twinning		
Analysis 1B Static (Undrained) - Flattened		
Seismic Coefficient H: g, V: g	Last Run 07/13/2021, 04:05:07 PM	Scale 1:300

Additional Details
Name: Embankment Fill Sta. 10+890
File Name: 10+890 1.25 Rock Fill_20210501.gsz
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m

Fig. G1-1B

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Clayey Silt (Drained)	Mohr-Coulomb	17.5	5	28
	Earth Fill	Mohr-Coulomb	20	0	30
	Gneiss Bedrock	Bedrock (Impenetrable)			
	Organic Silt (Drained)	Mohr-Coulomb	16	2	20
	Pavement Granular	Mohr-Coulomb	22	0	35
	Rock Fill	Mohr-Coulomb	20	0	40
	Sand to Silty Sand	Mohr-Coulomb	18.5	0	30



Project
Highway 17 Twinning

Analysis
2A Static (Drained)

Seismic Coefficient
H: g, V: g








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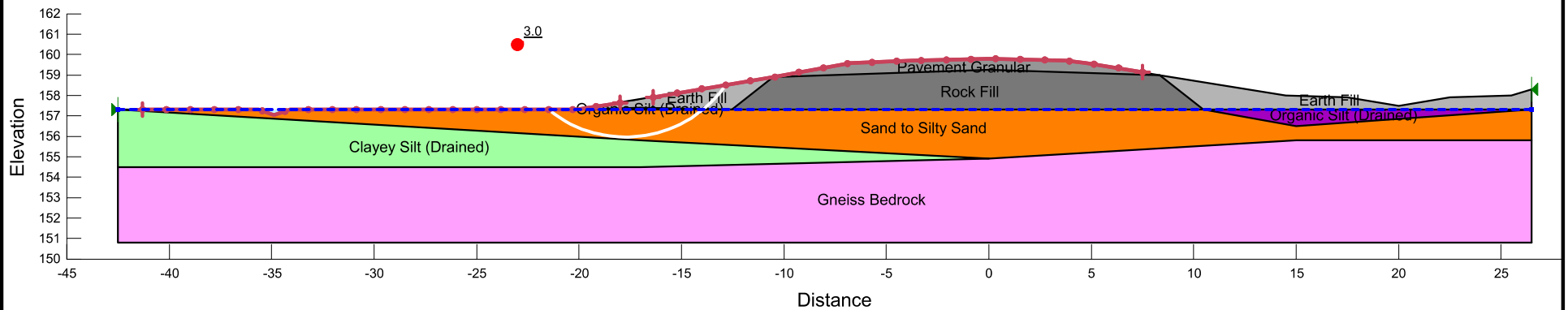
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Additional Details

Name: Embankment Fill Sta. 10+890
File Name: 10+890 1.25 Rock Fill_20210501.gsz
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m

Fig. G1-2A

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Clayey Silt (Drained)	Mohr-Coulomb	17.5	5	28
	Earth Fill	Mohr-Coulomb	20	0	30
	Gneiss Bedrock	Bedrock (Impenetrable)			
	Organic Silt (Drained)	Mohr-Coulomb	16	2	20
	Pavement Granular	Mohr-Coulomb	22	0	35
	Rock Fill	Mohr-Coulomb	20	0	40
	Sand to Silty Sand	Mohr-Coulomb	18.5	0	30



Project

Highway 17 Twinning

Analysis

2B Static (Drained) - Flattened

Seismic Coefficient

H: g, V: g

Last Run

07/13/2021, 04:05:09 PM

Scale

1:300

Additional Details








Name: Embankment Fill Sta. 10+890

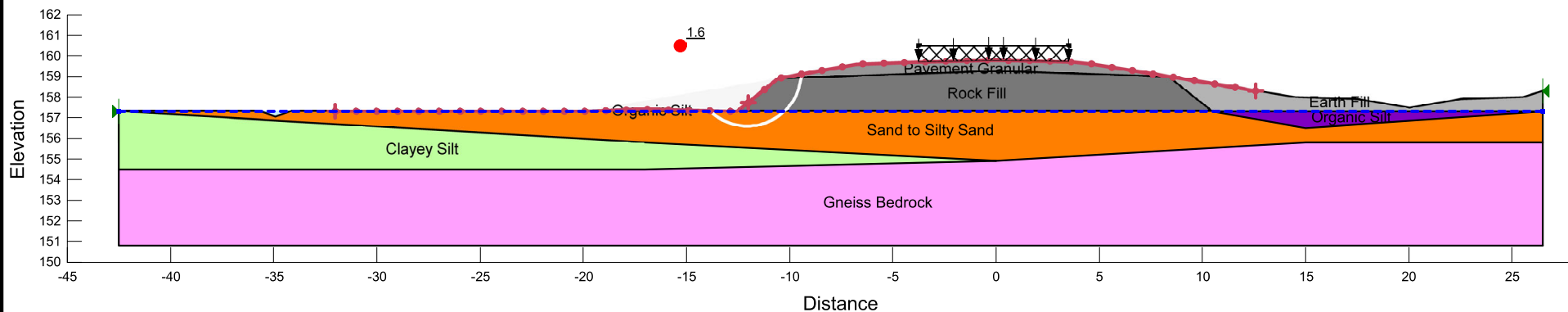
File Name: 10+890 1.25 Rock Fill_20210501.gsz

Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1.52 m

Fig. G1-2B








Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Clayey Silt	Mohr-Coulomb	17.5	20	0
	Earth Fill	Mohr-Coulomb	20	0	30
	Gneiss Bedrock	Bedrock (Impenetrable)			
	Organic Silt	Mohr-Coulomb	16	25	0
	Pavement Granular	Mohr-Coulomb	22	0	35
	Rock Fill	Mohr-Coulomb	20	0	40
	Sand to Silty Sand	Mohr-Coulomb	18.5	0	30

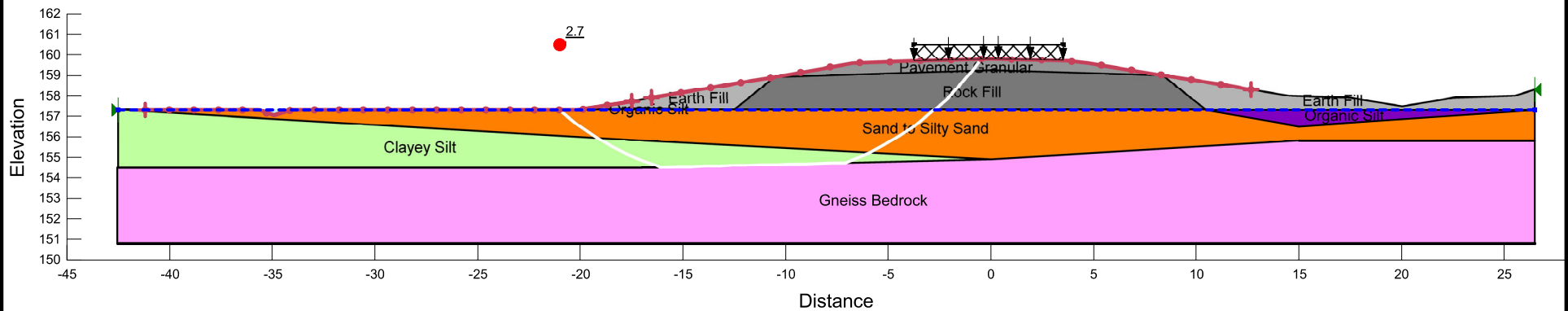


Project Highway 17 Twinning		
Analysis 3A Static Traffic (Undrained)		
Seismic Coefficient H: g, V: g	Last Run 07/13/2021, 04:05:10 PM	Scale 1:300

Additional Details
Name: Embankment Fill Sta. 10+890
File Name: 10+890 1.25 Rock Fill_20210501.gsz
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Surcharge Load: 17 kN/m³

Fig. G1-3A








Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Clayey Silt	Mohr-Coulomb	17.5	20	0
	Earth Fill	Mohr-Coulomb	20	0	30
	Gneiss Bedrock	Bedrock (Impenetrable)			
	Organic Silt	Mohr-Coulomb	16	25	0
	Pavement Granular	Mohr-Coulomb	22	0	35
	Rock Fill	Mohr-Coulomb	20	0	40
	Sand to Silty Sand	Mohr-Coulomb	18.5	0	30

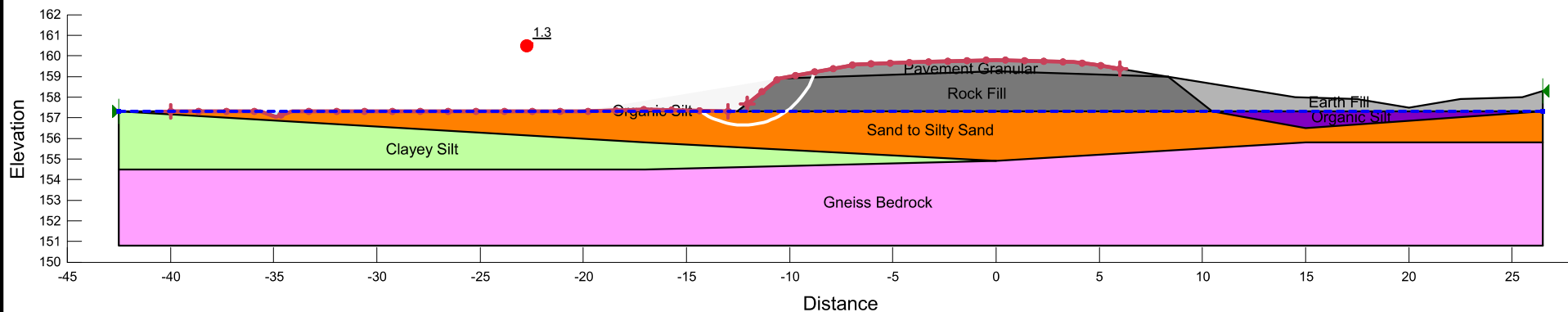


Project Highway 17 Twinning		
Analysis 3B Static Traffic (Undrained) - Flattened		
Seismic Coefficient H: g, V: g	Last Run 07/13/2021, 04:05:11 PM	Scale 1:300

Additional Details
Name: Embankment Fill Sta. 10+890
File Name: 10+890 1.25 Rock Fill_20210501.gsz
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m
Surcharge Load: 17 kN/m³

Fig. G1-3B








Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Clayey Silt	Mohr-Coulomb	17.5	20	0
	Earth Fill	Mohr-Coulomb	20	0	30
	Gneiss Bedrock	Bedrock (Impenetrable)			
	Organic Silt	Mohr-Coulomb	16	25	0
	Pavement Granular	Mohr-Coulomb	22	0	35
	Rock Fill	Mohr-Coulomb	20	0	40
	Sand to Silty Sand	Mohr-Coulomb	18.5	0	30

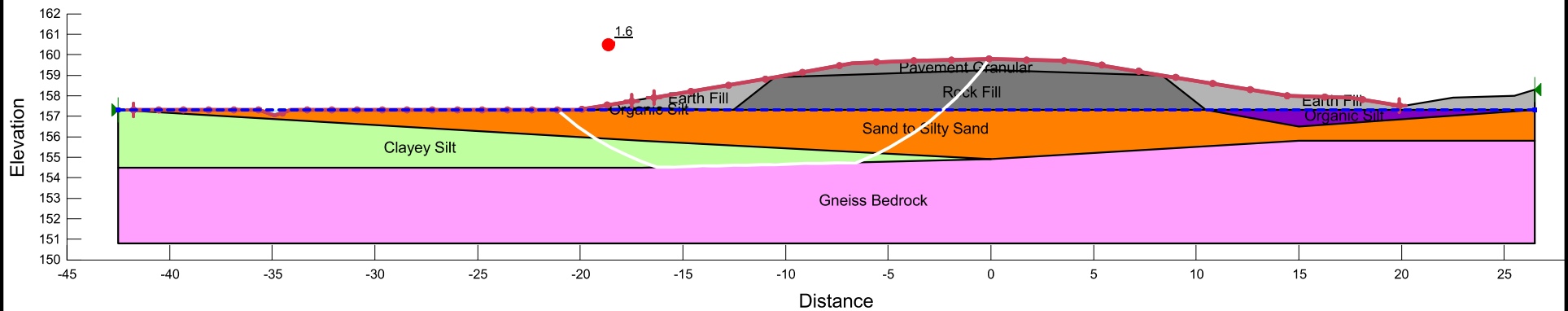


Project		
Highway 17 Twinning		
Analysis		
4A Pseudo Static (Undrained)		
Seismic Coefficient	Last Run	Scale
H: 0.114g, V: g	07/13/2021, 04:05:05 PM	1:300

Additional Details
Name: Embankment Fill Sta. 10+890
File Name: 10+890 1.25 Rock Fill_20210501.gsz
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m

Fig. G1-4A

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Clayey Silt	Mohr-Coulomb	17.5	20	0
	Earth Fill	Mohr-Coulomb	20	0	30
	Gneiss Bedrock	Bedrock (Impenetrable)			
	Organic Silt	Mohr-Coulomb	16	25	0
	Pavement Granular	Mohr-Coulomb	22	0	35
	Rock Fill	Mohr-Coulomb	20	0	40
	Sand to Silty Sand	Mohr-Coulomb	18.5	0	30



Project		
Highway 17 Twinning		
Analysis		
4B Pseudo Static (Undrained) - Flattened		
Seismic Coefficient	Last Run	Scale
H: 0.114g, V: g	07/13/2021, 04:05:06 PM	1:300

Additional Details
Name: Embankment Fill Sta. 10+890
File Name: 10+890 1.25 Rock Fill_20210501.gsz
Method: Morgenstern-Price, Half-Sine
Minimum Slip Surface Depth: 1.52 m

Fig. G1-4B

Appendix H.

List of Referenced Specifications Non-Standard Special Provisions

1. The following Special Provisions and OPSS Documents are referenced in this report:

OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment
OPSD 202.020	Drainage Gap for Slope Flattening on Rock or Granular Embankment
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less than or Equal to 3.0M
OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSS 422	Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 1860	Geotextiles
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheet piling
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1205	Material Specification for Clay Seal
OPSS.PROV 1860	Material Specification for Geotextiles
SP 110S06	Amendment to OPSS 1010, April 2013 Materials
SP 517F01	Amendment to OPSS 517 - Construction Specification for Dewatering
SP FOUN0003	Amendment to OPSS 902 – Dewatering Structure Excavations

2. Suggested wording for NSSPs

“Protection of Sensitive Foundation Soils”

The Contractor is advised that the very loose to loose sand subgrade that will be exposed, and the native clayey silt that may be exposed at the subgrade are moisture sensitive and may become disturbed or otherwise negatively impacted when subjected to construction or personnel traffic, freeze-thaw actions, ingress or ponding water. The Contractor shall be responsible for implementing adequate groundwater control measures to minimize disturbance from construction and personnel traffic on the exposed clayey silt subgrade and allow surface compaction of the native sand subgrade to 92% Standard Proctor maximum dry density.

“Structural Backfill”

Structural backfill for the culvert and retaining walls conforming to OPSS Granular A shall be quarry sourced.

“Notice to Contractor: Obstructions”

The Contractor is hereby notified that the native soils at the site and as inferred from available information should be expected to contain cobbles and boulders. Considerations of these obstructions must be made in the selection of appropriate equipment and procedures for excavations, and installations of temporary protection systems.