



THURBER ENGINEERING LTD.

FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 17 TWINNING, RENFREW AREA
CULVERT 14+666 WBL, MCNAB/BRAESIDE
WP 4068-09-00 / ASSIGNMENT NO. 4018-E-0009

Geocres No.: 31F-218

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) 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 to be located at Station 14+666 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. A new culvert is required to convey an unnamed tributary of Liffey Creek 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 14+666. 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 of the culvert was developed in the course of the investigation. No previous foundation information was available for this site in the Geocres Library.

2 SITE DESCRIPTION

2.1 General

The site is currently undeveloped and located approximately 55 m north of the existing Highway 17 alignment, 1.2 km east of McCallum Drive and 1.1 km west of Miller Road. For project purposes, Highway 17 is herein described as oriented east-west.

The land adjacent to the site generally consists of undeveloped, forested and marshy areas with an agricultural field beyond a recreational trail on an abandoned CNR alignment to the north. The



terrain is relatively flat with slight downward slope towards the small, unnamed tributary of Liffey Creek. The ground surface at the proposed culvert location was wet and covered with tall grass. At the time of the field investigation, no discernible creek bed was present. Standing water was observed near 14+666. 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. Three-cable guiderails are present on both sides. 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 Clay Plains. The Ottawa Valley Clay Plains are characterized primarily by clay plains deposited by the Champlain Sea (Leda Clay) interrupted by ridges of rock or sand.

A Physical Setting Report for the overall project prepared by ERIS and based on Ontario Geological Mapping Indicates that the underlying bedrock at the site is typically granodiorite, tonalite, monzogranite, syenogranite; derived gneisses and migmatites rocks of the Grenville Province.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field-testing program was carried out between September 30th and October 2nd, 2019. The field investigation consisted of advancing 3 boreholes identified as Boreholes CV-19 through CV-21. Prior to commencement of drilling, utility clearances were obtained in the vicinity of the borehole locations.

The locations and elevations of the boreholes were surveyed by Thurber 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.

Table 3-1: Borehole Summary

Borehole No.	Drilled Location	Northing (Latitude)	Easting (Longitude)	Ground Surface Elevation (m)	Termination Depth (m)
CV-19	Proposed Culvert Inlet	5 034 030.9 (45.446201)	301 124.5 (-76.546986)	139.9	11.3
CV-20	Proposed HWY17 WBL C/L	5 034 045.7 (45.446332)	301 130.8 (-76.546906)	140.0	10.8
CV-21	Proposed Culvert Outlet	5 034 059.0 (45.446453)	301 139.7 (-76.546792)	140.1	10.8

The 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.

A standpipe piezometer, 19 mm in diameter, was installed in Borehole CV-21. The installation details are illustrated on the respective Record of Borehole sheet provided in Appendix B. The boreholes were backfilled in accordance with MOE requirements (O.Reg 903, as amended). The piezometer will be utilized as part of a hydrogeological study and subsequently decommissioned by Thurber.

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-21.

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 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 organic silt overlying native clay to clayey silt deposit, which is underlain by a deposit of sandy clayey silt to clayey silty sand till.

5.1 Organic Silt

A thin layer of organic silt with an approximate thickness of 0.6 m was encountered at surface in all borehole locations. SPT tests conducted within the organic silt gave N-values of weight of hammer indicating a relative density of very loose.

5.2 Clay (CI) to Clayey Silt (CL)

A glaciomarine native deposit of clay to clayey silt was encountered below the organic silt at all borehole locations. The deposit extended to a depth of 7.6 m with an underside elevation ranging from 132.3 to 132.5 m. Sand was noted in the upper 0.9 m of the clay deposit in Borehole CV-19.

SPT tests conducted within the cohesive unit gave N-values ranging from weight of hammer to 18. In situ shear vane tests in several locations in the lower clayey silt indicated undrained shear strengths ranging from 59 to greater than 100 kPa indicating a stiff to very stiff consistency. Sensitivity ranged from 4.0 to 8.5.

The moisture content of the clay to clayey silt samples tested ranged from 26 to 41%. The results of six grain size analysis tests conducted on samples of this material are summarized below and are illustrated on Figures C1 in Appendix C.

Summary of Grain Size Distribution Testing – Clay to Clayey Silt

Soil Particle	Percentage (%)
Gravel	0 – 2
Sand	0 – 22
Silt	42 – 65
Clay	32 – 46

The results of Atterberg Limits testing carried out on six samples of this material are summarized below and are illustrated on Figure C3 in Appendix C. The laboratory results indicate that the material is generally a clay of intermediate to low plasticity (CI to CL). In general, the upper portion

of this layer was found to have a higher plasticity than the lower portion. It should be noted in accordance with the MTO Guideline for Foundation Engineering Services (May 2019) the lower cohesive deposit has been described as a “clayey silt” where Atterberg limits tests indicate a CL material.

Summary of Atterberg Limit Testing – Clay to Clayey Silt

Parameter	Value
Liquid Limit	25 – 43
Plastic Limit	15 – 19
Plasticity Index	10 – 24

5.3 Sandy Clayey Silt (CL) to Clayey Silty Sand (SC-SM) (Till)

A native deposit of glacial till was encountered below the clayey silt in all boreholes. The till consists of a mixture ranging from sandy clayey silt to clayey silty sand with traces of gravel. Possible cobbles were noted in Borehole CV-20. All boreholes were terminated within the till at a final depth ranging from 10.8 to 11.3 m (elev. 128.6 to 129.3 m).

SPT tests conducted in this layer gave N-values ranging from 1 to greater than 100 blows for 175 mm of penetration, however, the refusal likely represents the presence of cobbles or boulders rather than the relative density of the soil deposit. This deposit is considered to be non-cohesive in behaviour and loose to dense relative density, typically compact.

The moisture content of this unit ranged from 9 to 18%. The results of grain size distribution testing carried out on three samples of this material are summarized below and are illustrated on Figure C2 in Appendix C.

Summary of Grain Size Distribution Testing – Sandy Clayey Silt to Clayey Silty Sand

Soil Particle	Percentage (%)
Gravel	5 – 8
Sand	41 – 46
Silt	30 – 33
Clay	16 – 19

The results of Atterberg Limits testing carried out on three samples of the fine grained portion of the till are summarized below and are illustrated on Figure C4 in Appendix C. The laboratory results indicate that the fine grained material consists of clay to silty clay of low plasticity (CL to CL-ML). It should be noted in accordance with the MTO Guideline for Foundation Engineering Services (May 2019) the lower cohesive deposit has been described as a “clayey silt” where Atterberg limits tests indicate a CL material.



Summary of Atterberg Limit Testing – Sandy Clayey Silt to Clayey Silty Sand

Parameter	Value
Liquid Limit	16 – 18
Plastic Limit	9 – 10
Plasticity Index	7 – 8

5.4 Groundwater

A standpipe piezometer with a diameter of 19 mm was installed in Borehole CV-21. Groundwater levels recorded in the piezometer are presented in Table 5-1 below:

Table 5-1: Summary of Groundwater Levels

Borehole No.	Bottom of Screen Elevation (m)	Depth (m)	Groundwater Elevation (m)	Date of Measurement
CV-21	129.6	6.4 below g.s.*	133.7	October 1, 2019
		0.7 above g.s.*	140.8	November 26, 2019
		0.7 above g.s.*	140.8	April 21, 2020

*g.s. = ground surface

The groundwater level was recorded in the open Borehole CV-20 prior to backfilling at a depth of approximately 1.7 m (elev. 138.3 m) on October 1st, 2019.

At the time of the field investigation, no discernible creek bed was present. Standing water was observed.

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.



5.5 Analytical Testing

One sample of the native clay 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 5-2. Copies of the test results are provided in Appendix C.

Table 5-2: Results of Chemical Analysis

Borehole	CV-21
Sample	SS6
Depth (m)	3.8 – 4.4
Chloride (µg/g)	8
Sulphate (µg/g)	38
Sulphide (%)	0.08
pH (-)	7.58
Resistivity (Ohm-cm)	5,460
Conductivity (µS/cm)	183



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. 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

Part 2 of the report provides an interpretation of the factual data from Part 1 and presents geotechnical recommendations to assist the project team in designing the foundation for a proposed culvert crossing to be located at Station 14+666 along the proposed Highway 17 westbound lanes. The site is located 1.2 km east of McCallum Drive and 1.1 km west of Miller 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 55 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 Liffey Creek from south to north under a proposed embankment supporting the new Highway 17 westbound lanes. The existing elevation at site is approximately 139.9 m to 140.1 m and the proposed final grade of Highway 17 at St. 14+666 is approximate elevation 144.5 m.

The site was found to be underlain by a surficial organic silt deposit overlying a native clay to clayey silt deposit over a deposit of sandy clayey silt to clayey silty sand till. It is noted that the water level in the standpipe piezometer was at an elevation of 140.8 m on April 21, 2020.



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

The 2003 Stormwater Management and Drainage Report by National Capital Engineering (NCE) in support of the Preliminary Design Report for this project indicates a culvert is present beneath the existing highway at approximately Sta. 14+710. The culvert is described as a 1.05m diameter CSP. This report recommended a replacement CSP with the same diameter.

It is understood from the 2018 RFP that the existing culvert at Sta. 14+710 will be replaced during construction of the new eastbound lanes with a concrete box culvert (CBC) with a span of 3 m and a height of 1.05m.

In 2019, the existing culverts in the study area were inspected by WSP. Culvert 006700170034 was indicated to be present at Sta. 14+723 on the existing alignment; it is assumed that this is the same culvert at Sta. 14+710 discussed above. The culvert was measured as a 1.1 m diameter CSP with a length of 32.57 m and a depth of cover of 3 m. This report recommended a replacement be installed with trenchless methods.

A separate foundation report will be prepared for the eastbound culvert.

The 2003 Stormwater Management and Drainage Report by National Capital Engineering (NCE) in support of the Preliminary Design Report for this project indicates the new culvert under the westbound lanes should be a CBC with a span of 3 m and a height of 1.05m.

7.1 Proposed Structure

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

Per the General Arrangement Drawing dated July 2020, the proposed structural culvert beneath the new westbound lanes is to be a pre-cast, closed-bottom concrete box culvert (CBC) with a length of 13 m, a span of 5.5 m and a height of 4.0 m. Cast-in-place approach slab will extend 6 m from either side of the culvert. The stream bed will be at approximately elevation 140.0 m at the culvert centreline and higher near the culvert walls to allow passage of small mammals. The underside of the base slab is proposed to be at approximate elevation 139.1 m. Retaining walls ranging in length from 6 m to 8.5 m are indicated in all four quadrants. A cut off wall is proposed for the culvert outlet.

It is assumed that the embankment cross-section for the new westbound lanes outside the immediate area of the proposed culvert will be constructed as shown in Figure 7.3 of the Preliminary Design Report for this project with rock fill material at 1.25H:1V. The proposed embankment has a height of approximately 4.5 m above the original ground surface.



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, version CSA S6:19, (CHBDC).

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. 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 E.

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. Based on the average undrained shear strengths measured below the anticipated culvert foundation elevation, the site is classified as a Seismic Site Class D in accordance with Table 4.1 of the CHBDC.



As per Table 4.8 of the CHBDC, Site Class D yields a PGA_{ref} of 0.182 and $F(PGA)$ of 1.135 for the site. These values give a factored PGA of 0.258 g.

8.3 Seismic Liquefaction Potential

The susceptibility of the cohesive soils 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. The cohesionless soils consisting of glacial till are not considered to be susceptible to liquefaction.

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:

- Circular Pipes (Concrete, HDPE, Steel)

Although, from a foundation engineering perspective, a pipe culvert is a technically feasible alternative, the proposed pipe must meet the required flow capacity, navigation and hydraulic requirements as well as provide passage for mammals.

- Open-Bottom Culvert (Box, Arch)

The construction of an open-bottom culvert will have greater construction concerns due to the shallow water table 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 and has the potential for larger settlement following construction when compared to other culvert options.

- Closed-Bottom Culvert (Box)

A pre-cast, 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.



A comparison of these alternatives, based on their respective advantages and disadvantages, is included in Appendix F. It is not considered to be economical or 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, no discernible creek bed was present, however, standing water was observed near 14+666. 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.

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, 3 to 6 months in length, would be required prior to carrying out the open cut excavation for the culvert installation (See Section 10.6.2). 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 concrete box culvert is recommended.

- | | |
|-----------------------------------------------------------|------------------|
| • Proposed top of pavement westbound Highway 17 | 144.5 m |
| • Low point of stream bed in proposed culvert | 140.0 m |
| • Proposed elevation of underside of base slab of culvert | 139.1 m |
| • Groundwater elevation | 140.8 m |
| • Clay/Till interface | 132.3 to 132.5 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 4.5 m. Further discussion on the potential settlement of the subgrade soils is provided in Section 10.6. The subgrade should be prepared as described in Section 10.3.



The recommended geotechnical resistances for a 6.5 m wide (exterior) pre-cast, closed-bottom, box culvert with the underside of culvert base slab at or below approximate elevation 139.1 m, installed on a bedding layer with a minimum thickness of 0.3 m placed on an undisturbed native clay subgrade are as follows:

- Factored Geotechnical Resistance at ULS of 200 kPa
- Factored Geotechnical Resistance at SLS of 100 kPa (provided a preload period is included in the schedule – see Section 10.6)

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 Sections 10.3 and 10.6 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 Wingwalls / Retaining Walls

Culvert wingwalls are proposed to retain embankment backfill, direct water flow into the culvert inlet and protect the soils around the culvert from erosion.

Based on General Arrangement Drawing dated July 2020 (See Appendix F), it is understood that retaining walls are proposed to retain the slopes at the inlet and outlet of the proposed culvert. Preliminary design drawings suggest the concrete retaining walls would be supported on spread footing foundations, which is considered feasible.

Footings for wingwalls should be founded at a depth, when measured perpendicular to the ground surface, that is greater than the depth of frost (i.e. below elevation 138.1 m - see Section 10.4). The walls should be founded on a leveling pad with a minimum thickness of 0.3 m consisting of



Granular A material. The top of the Granular A pad must extend to 0.5 m beyond the outside edge of all sides of the footing and sloped away from the footing at 1H:1V, or flatter.

The geotechnical resistance values provided in Section 10.1 are applicable for a 2 m to 3 m wide shallow footing up to 10 m long on an engineered pad 0.3 m thick placed on native, undisturbed clayey silt/clay. Subgrade preparation recommendations are provided in Section 10.3.

It is noted that wingwalls can be omitted if the culvert structure is lengthened to allow embankment side slopes as described in Section 10.6.

A retained soil system (RSS) for a culvert wingwall or retaining wall is not recommended at this site as it is located within a watercourse and could be affected by fluctuating water levels.

10.3 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 0.6 m was encountered across the area of the proposed culvert during the drilling investigation and must be removed from beneath the culvert, wingwall and embankment footprint. The bearing resistances provided above assume that organic material and very soft to soft clay deposits, where encountered at the subgrade level within the culvert footprint are removed.

Any soft or organic materials at the subgrade level should be sub-excavated and backfilled and compacted as per OPSS.PROV 501 with granular fill consisting of OPSS.PROV 1010 Granular A material 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.

Given the subgrade soils anticipated at the founding level of the replacement culvert, construction equipment should not be permitted to travel on the exposed subgrade. The compaction of granular bedding directly above the subgrade may result in disturbance of the material with pumping of fines into the granular bedding and difficulty achieving the specified degree of compaction. After inspection and approval of the subgrade, protection of the subgrade should include installation of a Class II, non-woven geotextile with a maximum FOS of 150 μ m (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 sensitive nature of the foundation soils.

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 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 the culvert in order not to damage the culvert.

10.4 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.

10.5 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. 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 803.010 for the culvert and OPSD 3101.150 for the retaining walls. 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 wingwalls/retaining walls, where required, must 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 hydrostatic pressures should be considered in design.

Where back slopes are horizontal or 2H:1V, 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.

10.5.1 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 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 $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	2H:1V Slope Behind Wall	Horizontal Surface Behind Wall	2H:1V Slope Behind Wall
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.36	0.52	0.43	0.62
Coefficient of Active Earth Pressure, K_A (Unrestrained Wall)	0.22	0.30	0.27	0.39

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.

10.5.2 Combined Static and Seismic Lateral Earth Pressure

In accordance with Clause 6.14 of the current version of 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 D 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 1.135 as per Table 4.8 of the current version of CHBDC.

Table 10-2: Combined Static and Seismic 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 $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	2H:1V Slope Behind Wall	Horizontal Surface Behind Wall	2H:1V Slope Behind Wall
Coefficient of Active Earth Pressure, K_{AE} (Restrained Wall)	0.37	0.50	0.44	0.60
Coefficient of Active Earth Pressure, K_{AE} (Unrestrained Wall)	0.28	0.44	0.35	0.65

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)

10.6 Embankment Fill

Embankments shall be constructed in accordance with OPSS.PROV 206.

10.6.1 Embankment Stability

Embankment stability has been assessed perpendicular to the highway alignment at two locations:

- As per the General Arrangement Drawing dated July 2020, the proposed embankment at Station 14+658 is approximately 4.5 m in height and consists of structural backfill retained by retaining walls at the culvert inlet and outlet.
- Beyond the retaining walls at Station 14+ 685, typical cross sections for the proposed Highway 17 indicate 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.

Slope stability for proposed embankment of the new Highway 17 westbound lanes and the proposed concrete retaining wall structures were 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 the stability analyses outputs provided in Appendix G. The following additional parameters and assumptions were used in the analysis:

- The soil stratigraphy is based on the nearest boreholes. The organic silt layer must be removed and replaced with Granular A from beneath the embankment footprint and to 2 m past the embankment toe.
- Concrete retaining walls must be founded at or below the frost depth outlined in Section 10.4.
- The maximum fill height is 4.5 m (Station 14+666).
- Although the General Arrangement Drawing indicates the backfill material to consist of Granular B III, 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.
- The draft Pavement design report indicates the pavement structure on rock fill embankments will consist of 190 mm of hot mix over a 150 mm Granular O base layer over a 200 mm Granular B Type II subbase layer with a total thickness of 540 mm. Near the structure, the approach slab will be assumed to be concrete 150 mm thick surfaced with 90 mm of asphalt and will be placed on 300 mm of granulars.
- A site adjusted PGA value of 0.129 g, equal to $\frac{1}{2}$ of the site adjusted PGA value (0.258 g), was used for seismic analysis, as per Section 4.4.3.3, of the CHBDC and outlined in Section 8.2 above.
- A traffic surcharge of 17 kPa has been applied as a temporary load for the rock fill embankment case. In accordance with Section 6.12.5 of the CHBDC the traffic

surcharge does not need to be applied where there is an approach slab supported by a structure.

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:

Table 10-3 Slope Stability Analysis Results for Rock Fill Embankments

Condition	Case	Rock Fill, 1.25H:1V No Slope Flattening	Rock Fill, 1.25H:1V Slope Flattening at 6H:1V
Temporary (traffic loading)	Short Term (Undrained)	1.6 (Fig G1-1)	3.9 (Fig G2-1)
Permanent (no traffic loading)	Long Term (Drained)	1.5 (Fig G1-2)	3.0 (Fig G2-2)
Temporary (includes seismic)	Pseudo-Static Seismic (Undrained)	1.3 (Fig G1-3)	2.1 (Fig G2-3)

The stability analyses generated the following factor of safety values for the proposed retaining wall structures:

Table 10-4 Slope Stability Analysis Results for Retaining Wall Sections

Condition	Case	Northeast and Southwest (no upper slope)	Northwest and Southeast (with upper slope)
Temporary	Short Term (Undrained)	3.4 (Fig G3-1)	3.1 (Fig G4-1)
Permanent (no traffic loading)	Long Term (Drained)	1.9 (Fig G3-2)	1.8 (Fig G4-2)
Temporary (includes seismic)	Pseudo-Static Seismic (Undrained)	2.3 (Fig G3-3)	2.3 (Fig G4-3)

Table 6.2 of the CHBDC for embankment fills with a typical degree of understanding and a Ψ of 1.0 generates minimum Factors of Safety of 1.5 and 1.3 for permanent and temporary conditions respectively. All of the static results presented in Table 10-3 and 10-4 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. Based on these values and Ψ of 1.0, a target Factor of Safety of 1.1 for this temporary condition with a typical degree of understanding is appropriate for the pseudo-static seismic analysis. The pseudo-static result



presented in Table 10-3 above, exceeds 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 Table 10-3 and 10-4 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 embankment slopes, see Section 11.4.

10.6.2 Embankment Settlement

The future westbound lanes will result in a fill height of about 4.5 m across 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 Settle3 modelling software with a Boussinesq stress distribution. The soil stratigraphy was defined based on borehole data 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 = 4.5 m
- Length = 200 m
- Platform Width = 15 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.

Furthermore, an assessment of the time dependent settlement that would result from construction of the proposed concrete retaining walls was carried out utilizing a Boussinesq stress distribution.

The geotechnical parameters used in the settlement analysis were based on an average of the borehole stratigraphies and consolidation test results from similar soils at the Locha Creek Culvert site, approximately 1 km east of the Culvert at 14+666 (Gecores Report No. 31F-212).

The clay stratum was separated into two sub-layers; upper clay and lower clayey silt. Table 10-5 presents the properties used in the Settle3 analysis for the various sub-layers.

Table 10-5: Settle3 Inputs

Clay Layer	Elevation (m)	C _c	C _r	P' _c (kPa)	e _o
Upper Clay	140.0 to 137.7	0.5	0.04	210 to 440	1.26
Lower Clayey Silt	137.7 to 132.4	0.5	0.04	440 to 290	1.26

At the proposed embankment consisting of approximately 4.5 m of additional fill, a total settlement of approximately 70 to 80 mm is expected to occur within one year following fill placement with 95% of the settlement occurring within 6 to 8 months.

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 total embankment settlement of the westbound embankment exceed 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 3 to 6 months to ensure that post-construction settlement meets the above guidelines. The end of preload will need to be confirmed with a settlement monitoring program. 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. For this scenario, the four retaining walls should also be designed to accommodate the anticipated settlements at the wall locations.

10.7 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 5.5 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 5.5 were compared with Table 3.2 of the MTO Gravity Pipe Design Guideline and indicate a low corrosive environment.



The test results provided in Section 5.5 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 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 firm to stiff silty clay and clay soils are considered Type 3. Side slopes for excavations through more than one soil type must be entirely based on the highest soil type number. Unsupported excavations made in Type 3 soils must have side slopes no steeper than 1H:1V from the base of the excavation.

Excavations for the footings must be carried out in accordance with OPSS 902 and will extend into the underlying native deposits (clay to clayey silt). 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. 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. The protection system should be installed at a suitable distance away from the new structures to limit the disturbance to subgrade associated with removal of the protection system following completing of construction. Alternatively, the protection system near the structures could be left in place and cut off in accordance with OPSS.PROV 903 to limit the disturbance of subgrade during removal of the TPS.

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 clay to clayey silt:

$$\begin{array}{rcl} \gamma & = & 17.5 \text{ (kN/m}^3 \text{ bulk unit weight of soil, to be adjusted below water)} \\ K_A & = & 0.36 \\ K_P & = & 2.8 \end{array}$$

Native clayey silty sand till:

$$\begin{array}{rcl} \gamma & = & 18 \text{ (kN/m}^3 \text{ bulk unit weight of soil, to be adjusted below water)} \\ K_A & = & 0.30 \\ K_P & = & 3.3 \end{array}$$

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.

When designing roadway protection systems, the Contractor should consider the potential for obstructions such as cobbles and boulders (inherent in glacial tills) as well as minor artesian conditions that were noted during groundwater measurements in the glacial till layer. Given the presence of potential obstructions at this site, installation of interlocking sheet piles may be difficult. A soldier pile and lagging system is a feasible option. It may be necessary to predrill for the soldier piles. Lateral support may need to be enhanced by socketing the soldier piles into bedrock and/or by using bracing or rakers. Suggested wording for an NSSP for obstructions is included in Appendix H.

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 anticipated that the site will require dewatering to at least 0.5 m below the final excavation or footing level. 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 and retaining walls in dry and stable excavations.

Subgrade preparation, placement and compaction of granular bedding, culvert and retaining wall 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.

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 recommended, 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 sump pumps will likely be sufficient to extract water from the excavation for the culverts. Pumping from within a sandbag cofferdam system is likely sufficient. A sheet pile cofferdam enclosure driven into the foundation clay may also be considered. The groundwater level within the work zone should be lowered by pumping from sumps to a minimum of 0.5 m below the underside of the planned excavation base prior to each stage of excavation.

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 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. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804. Slope vegetation should be established as soon as possible after completion of the embankment fills in order to limit surficial erosion.

Scour protection shall be provided at the culvert inlet and outlet areas. Effective scour and erosion protection should be provided along the waterline, ditches and around footings. Design of the erosion protection measures must consider hydrologic and hydraulic factors and shall be carried out by specialists experienced in this field. Typically, rock protection should be provided over all surfaces with which creek water is likely to be in contact. Treatment at the inlet and outlet of culverts shall be in accordance with OPSD 810.010.

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:

- Construction will extend below the water level in the creek. An adequate and effective surface water management and dewatering plan must be implemented to construct the foundations in the dry.
- Control of groundwater seepage during excavation. Minor artesian conditions in the underlying glacial till layer were noted during groundwater measurements. Due to depth of these artesian conditions, they are not expected to affect the excavation dewatering process.
- The clay which will be exposed beneath a culvert bedding layer or wing wall spread footings is sensitive and readily disturbed. A suggested Notice to Contractor is provided in Appendix H.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the existing soils to support the proposed construction equipment and supplies.
- 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 to 6 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 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, Mrs. Deanna Pizycki, 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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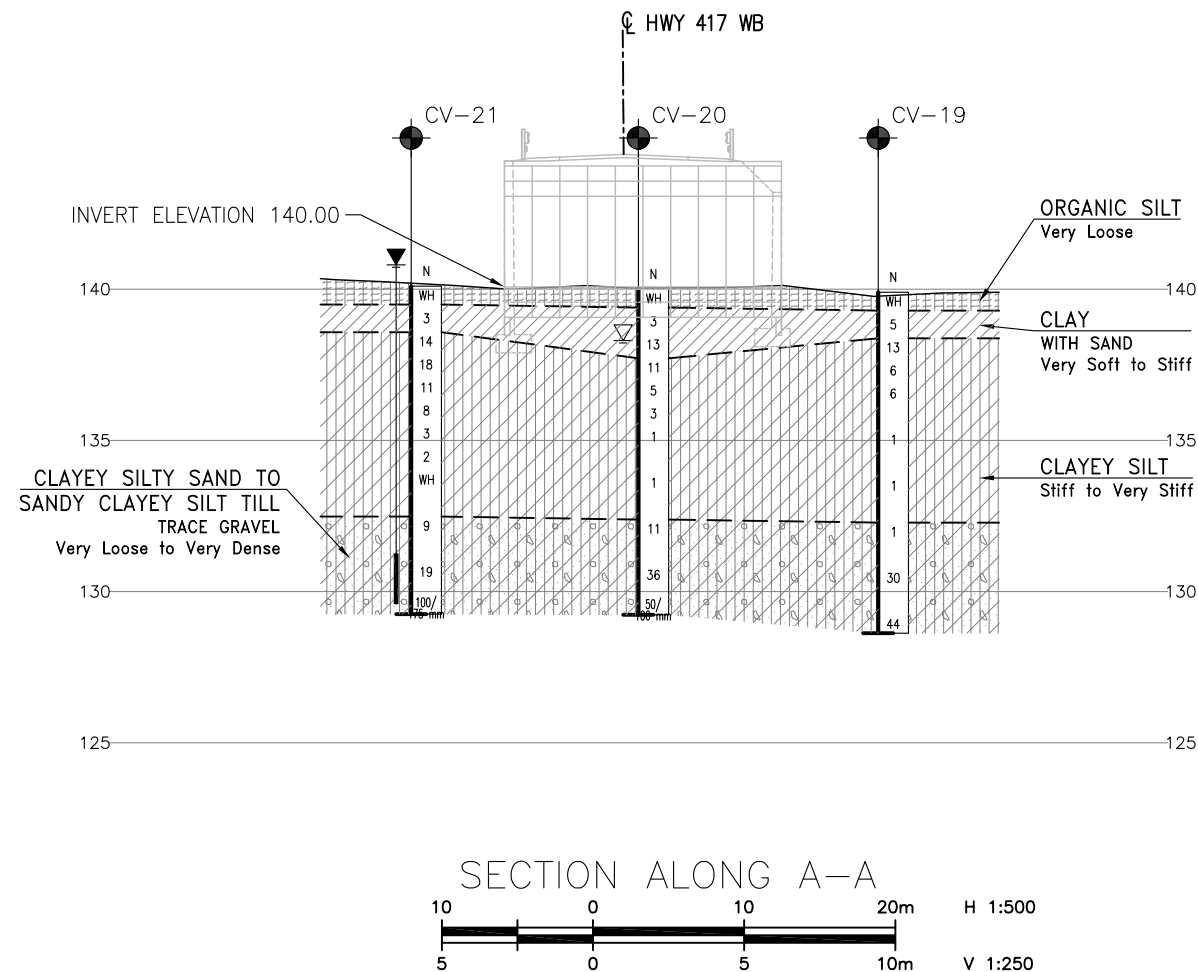
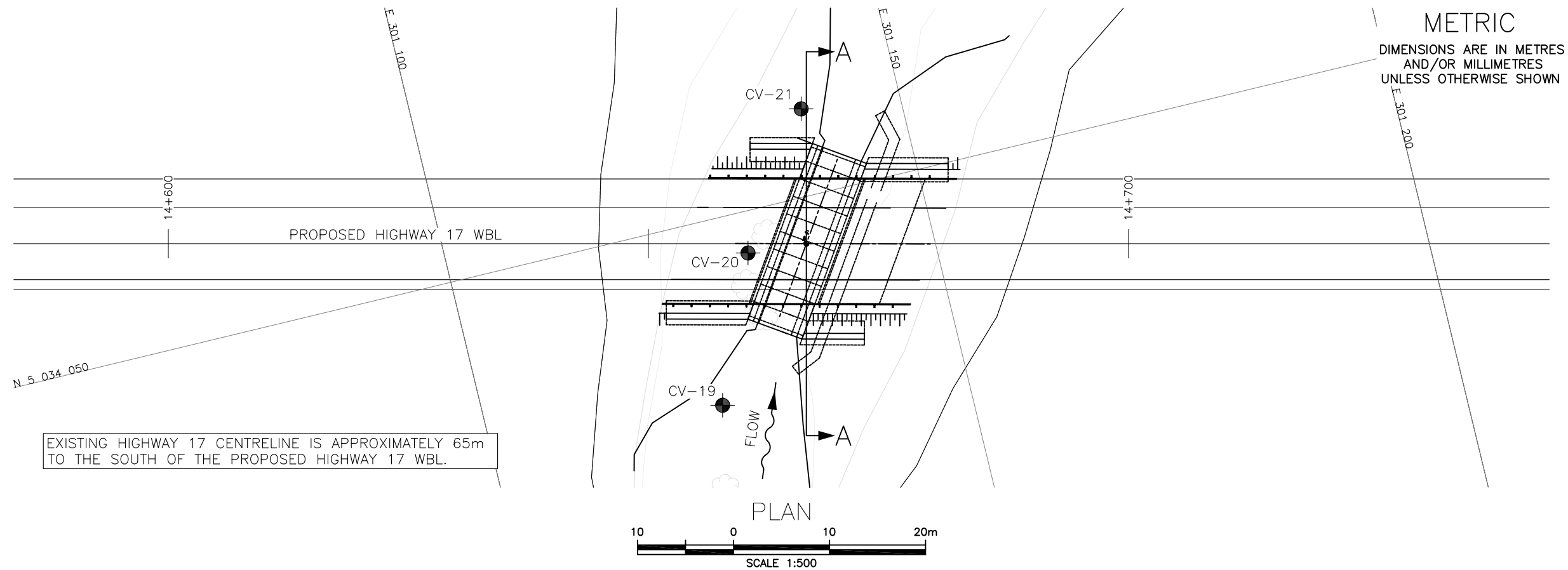
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Appendix A.

Borehole Location Plan and Stratigraphic Drawings



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



CONT No
WP No 4068-09-00





HIGHWAY 17 TWINNING
CULVERT STA. 14+666

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

[illegible]

-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-218

REVISIONS							
	DATE	BY	DESCRIPTION				
DESIGN	KE	CHK -	CODE	LOAD	DATE	JUL 2021	
DRAWN	AN	CHK KE	SITE	STRUCT	DWG	1	

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).

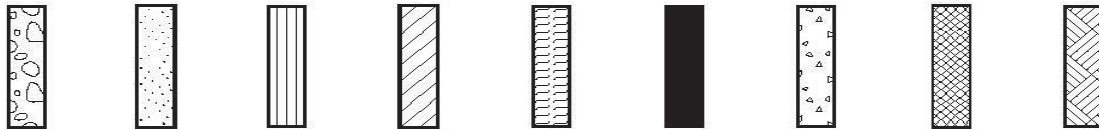
DYNAMIC CONE PENETRATION TEST (DCPT):

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

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.

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 (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 clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean 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

METRIC

[illegible]


+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No CV-19

2 OF 2

METRIC

WP# 4068-09-00 LOCATION Lat: 45.446201°, Long: -76.546986° ORIGINATED BY MJJ
 HWY 17 BOREHOLE TYPE CME 45 Trackmount, HSA COMPILED BY JP
 DATUM Geodetic DATE 2019.10.01 - 2019.10.02 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
	Continued From Previous Page																
128.6	SANDY CLAYEY SILT (CL), trace gravel Very Loose to Dense Grey (TILL)		10	SS	44		129									8 41 32 19	
11.3	End of Borehole																

DOUBLE LINE 24726 CULVERT 14+666 GINT.GPJ 2012TEMPLATE(MTO).GDT 21/4/21

METRIC[illegible]

+ 3, × 3: Numbers refer to Sensitivity

METRIC

SOIL PROFILE						SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT	<div>DYNAMIC CONE PENETRATION RESISTANCE PLOT</div>	<div>PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT</div>	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa						WATER CONTENT (%)
								<div>○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE</div>						
	Continued From Previous Page													
129.2 10.8	CLAYEY SILTY SAND (CL-ML), trace gravel Compact to Very Dense Grey (TILL)		11	SS	50/									
	End of Borehole Water level in open borehole at 1.7 m below ground surface (elevation 138.3) prior to backfilling				100 mm									

RECORD OF BOREHOLE No CV-21

1 OF 2

METRIC

WP# 4068-09-00 LOCATION Lat: 45.446453°, Long: -76.546792°
Culvert 14+666 N 5 034 059.0 E 301 139.7 ORIGINATED BY MJJ
HWY 17 BOREHOLE TYPE CME 45 Trackmount, HSA COMPILED BY JP
DATUM Geodetic DATE 2019.10.01 - 2019.10.01 CHECKED BY FG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W _P W W _L	WATER CONTENT (%)				
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
140.1	Ground Surface							20 40 60 80 100		20 40 60			GR SA SI CL	
0.0	ORGANIC SILT Very Loose		1	SS	WH		140							
139.5														
0.6	CLAY (CI) Very Soft to Firm Grey		2	SS	3		139							0 6 48 46
138.6														
1.5	CLAYEY SILT (CL) Stiff to Very Stiff Grey		3	SS	14		138							
			4	SS	18		137							
			5	SS	11		136							
			6	SS	8		135							
			7	SS	3		134							
			8	SS	2		133							
			9	SS	WH		132							0 1 57 42
132.5														
7.6	CLAYEY SILTY SAND (SC-SM), trace gravel Loose to Very Dense Grey (TILL)		10	SS	9		131							8 45 30 17
			11	SS	19									

Continued Next Page

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

DOUBLE LINE 24726 CULVERT 14+666 GINT.GPJ 2012TEMPLATE(MTO).GDT 21/4/21

RECORD OF BOREHOLE No CV-21

2 OF 2

METRIC

WP# 4068-09-00 LOCATION Lat: 45.446453°, Long: -76.546792°
Culvert 14+666 N 5 034 059.0 E 301 139.7 ORIGINATED BY MJJ
HWY 17 BOREHOLE TYPE CME 45 Trackmount, HSA COMPILED BY JP
DATUM Geodetic DATE 2019.10.01 - 2019.10.01 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	WATER CONTENT (%)								
	Continued From Previous Page							20	40	60	80	100					
129.3	CLAYEY SILTY SAND (SC-SM), trace gravel Loose to Very Dense Grey (TILL)		12	SS	100/		130										
10.8	End of Borehole WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2019.10.01 6.4 133.7 2019.11.26 0.7 above g.s. 140.8 2020.04.21 0.7 above g.s. 140.8				175 mm												

DOUBLE LINE 24726 CULVERT 14+666 GINT.GPJ 2012TEMPLATE(MTO).GDT 21/4/21

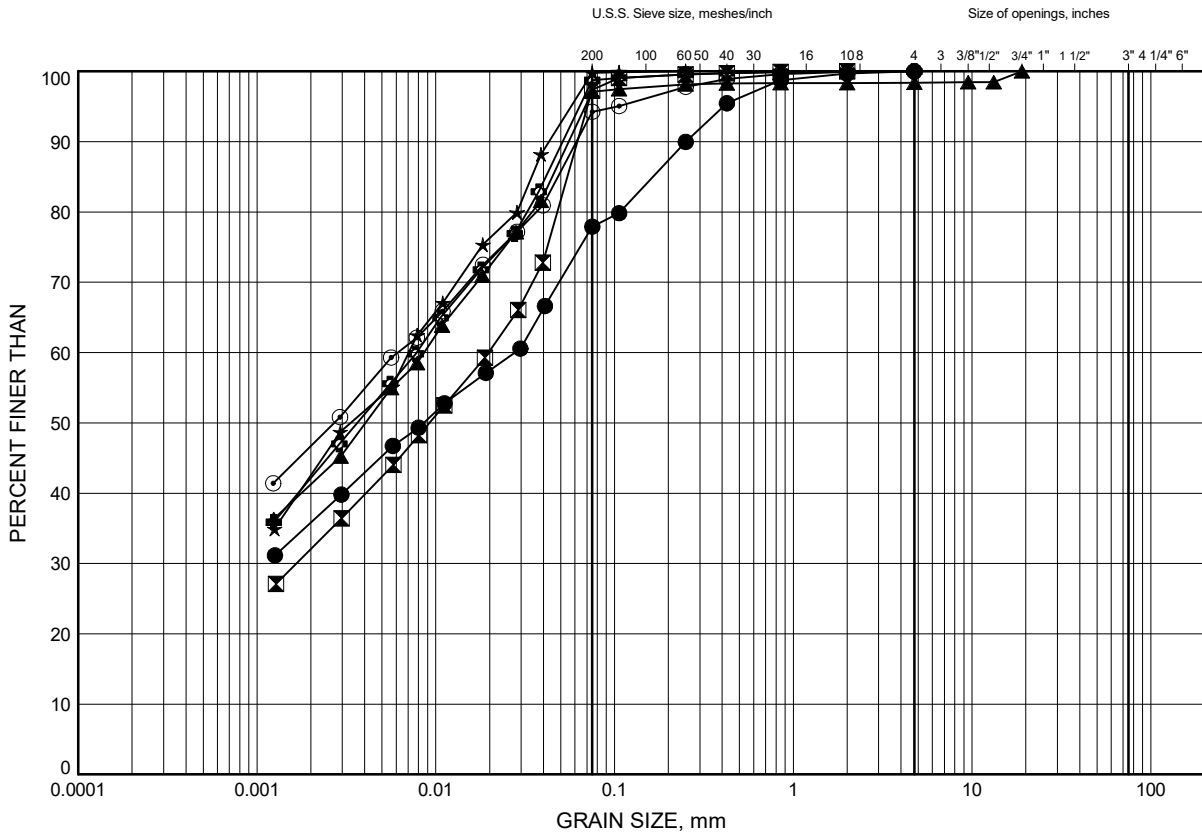
Appendix C.
Laboratory Testing

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

Highway 17 Twinning GRAIN SIZE DISTRIBUTION

FIGURE C1

CLAY (CI) TO 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-19	1.1	138.8
⊠	CV-19	4.9	135.0
▲	CV-20	1.8	138.2
★	CV-20	4.1	135.9
⊙	CV-21	1.1	139.0
⊕	CV-21	6.4	133.7

Date June 2020
WP# 4068-09-00

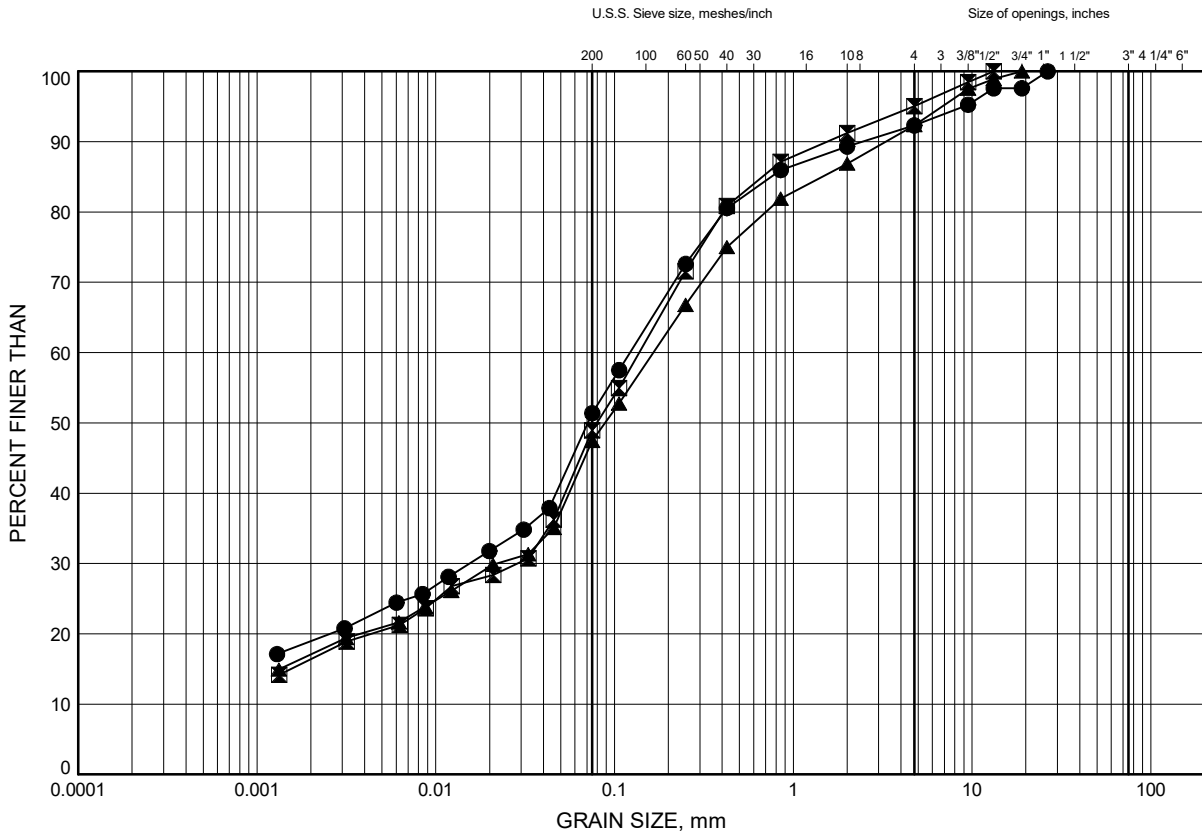


Prep'd KE
Chkd. FG

Highway 17 Twinning GRAIN SIZE DISTRIBUTION

FIGURE C2

SANDY CLAYEY SILT (CL) TO CLAYEY SILTY SAND (SC-SM) (TILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CV-19	11.0	128.9
⊠	CV-20	9.4	130.6
▲	CV-21	7.9	132.2

Date June 2020
WP# 4068-09-00

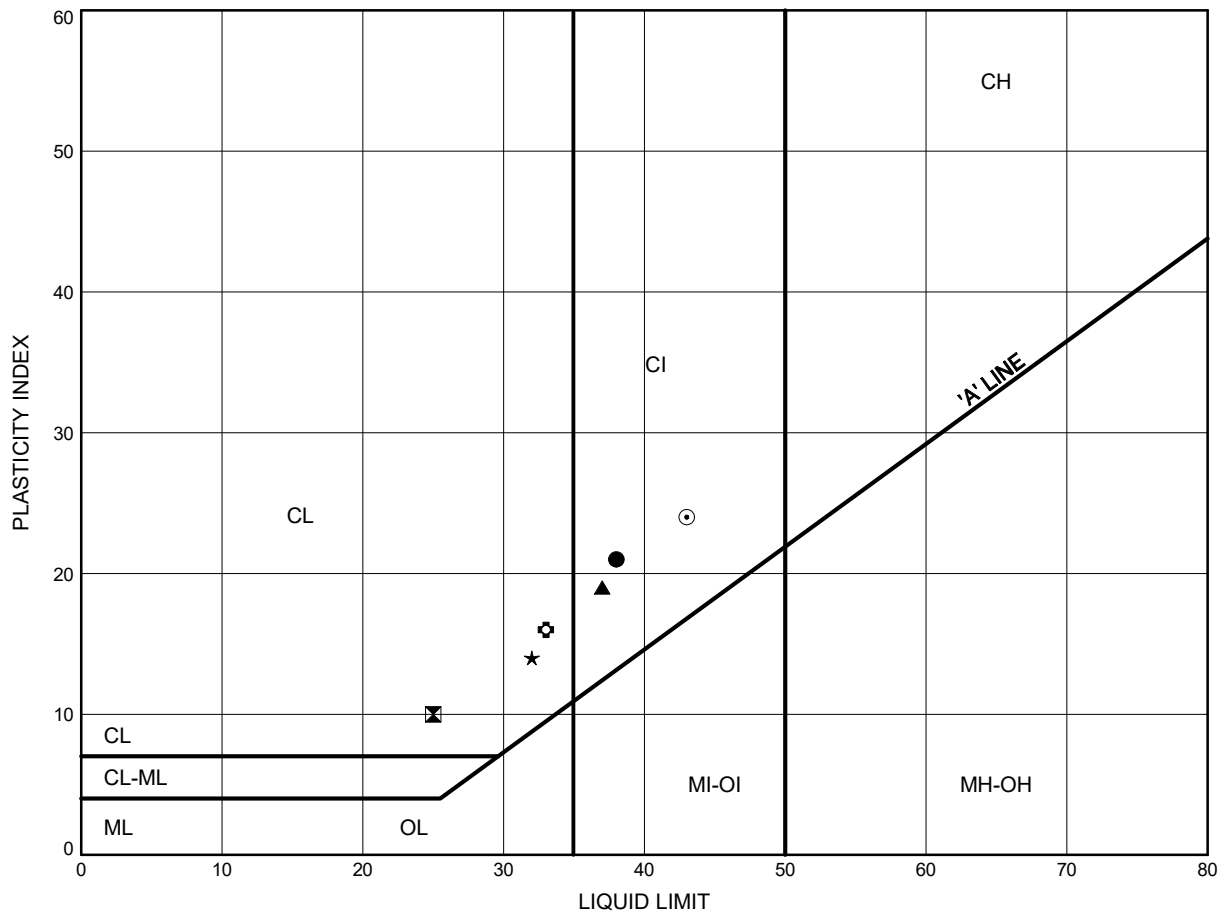


Prep'd KE
Chkd. FG

Highway 17 Twinning ATTERBERG LIMITS TEST RESULTS

FIGURE C3

CLAY (CI) TO CLAYEY SILT (CL)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CV-19	1.1	138.8
⊠	CV-19	4.9	135.0
▲	CV-20	1.8	138.2
★	CV-20	4.1	135.9
⊙	CV-21	1.1	139.0
⊕	CV-21	6.4	133.7

Date June 2020
 WP# 4068-09-00

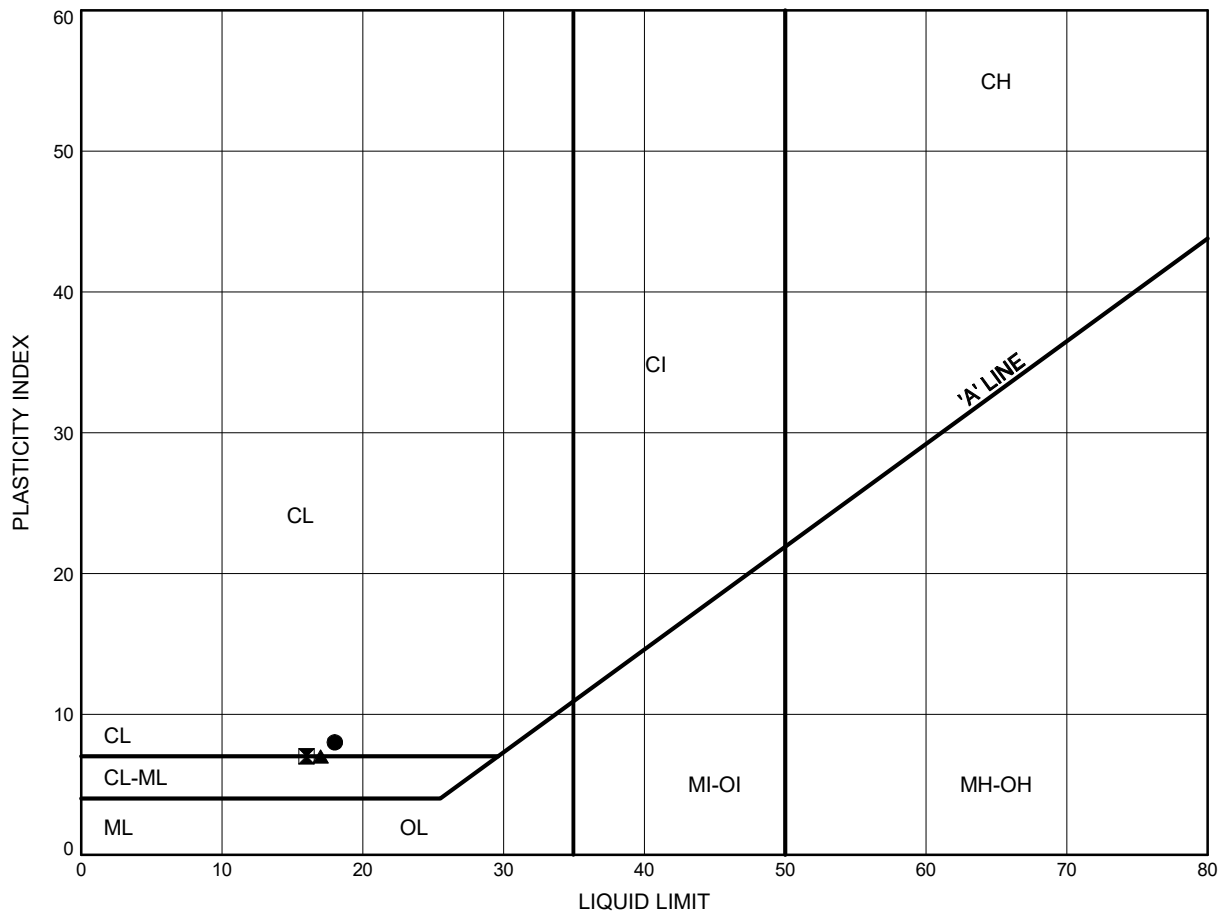


Prep'd KE
 Chkd. FG

Highway 17 Twinning ATTERBERG LIMITS TEST RESULTS

FIGURE C4

SANDY CLAYEY SILT (CL) TO CLAYEY SILTY SAND (SC-SM) (TILL)



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CV-19	11.0	128.9
⊠	CV-20	9.4	130.6
▲	CV-21	7.9	132.2

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 K1B4S5
Attn: Chris Murray

Client PO:
Project: 24726
Custody: 49912

Report Date: 10-Oct-2019
Order Date: 4-Oct-2019

Order #: 1940635

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

Paracel ID
1940635-01

Client ID
CV-21, SS6 (12'6"-14'6")

Approved By:



Mark Foto, M.Sc.
Lab Supervisor

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

Report Date: 10-Oct-2019
Order Date: 4-Oct-2019
Project Description: 24726

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	8-Oct-19	8-Oct-19
Conductivity	MOE E3138 - probe @25 °C, water ext	9-Oct-19	9-Oct-19
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	9-Oct-19	9-Oct-19
Resistivity	EPA 120.1 - probe, water extraction	9-Oct-19	9-Oct-19
Solids, %	Gravimetric, calculation	7-Oct-19	7-Oct-19

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

Report Date: 10-Oct-2019

Order Date: 4-Oct-2019

Project Description: 24726

Client ID:	CV-21, SS6 (12'6"-14'6")	-	-	-
Sample Date:	01-Oct-19 09:00	-	-	-
Sample ID:	1940635-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	79.6	-	-	-
----------	--------------	------	---	---	---

General Inorganics

Conductivity	5 uS/cm	183	-	-	-
pH	0.05 pH Units	7.58	-	-	-
Resistivity	0.10 Ohm.m	54.6	-	-	-

Anions

Chloride	5 ug/g dry	8	-	-	-
Sulphate	5 ug/g dry	38	-	-	-

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

Report Date: 10-Oct-2019

Order Date: 4-Oct-2019

Project Description: 24726

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:

Report Date: 10-Oct-2019

Order Date: 4-Oct-2019

Project Description: 24726

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	502	5	ug/g dry	486			3.1	20	
Sulphate	123	5	ug/g dry	122			0.6	20	
General Inorganics									
pH	7.16	0.05	pH Units	7.13			0.4	2.3	
Resistivity	90.0	0.10	Ohm.m	89.9			0.2	20	
Physical Characteristics									
% Solids	94.3	0.1	% by Wt.	94.2			0.2	25	

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

Report Date: 10-Oct-2019

Order Date: 4-Oct-2019

Project Description: 24726

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	576	5	ug/g	486	90.0	82-118			
Sulphate	229	5	ug/g	122	107	80-120			

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

Report Date: 10-Oct-2019
Order Date: 4-Oct-2019
Project Description: 24726

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 K1B4S5

Attn: Chris Murray

Tel: (613) 247-2121

Fax: (613) 247-2185

Paracel Report No **1940635**

Client Project(s): **24726**

Client PO:

Order Date: 04-Oct-19

Report Date: 10-Oct-19

Reference: **Standing Offer**

CoC Number: **49912**

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
1940635-01	CV-21, SS6 (12'6"-14'6")	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

10-October-2019

Date Rec. : 08 October 2019
LR Report: CA13304-OCT19
Reference: Project#: 1940635

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide %
1: Analysis Start Date		10-Oct-19
2: Analysis Start Time		14:36
3: Analysis Completed Date		10-Oct-19
4: Analysis Completed Time		14:53
5: QC - Blank		< 0.02
6: QC - STD % Recovery		115%
7: QC - DUP % RPD		16%
8: RL		0.02
9: CV-21, SS6 (12'6"-14'6")	01-Oct-19	0.08

RL - SGS Reporting Limit

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

Appendix D.
Site Photographs



Photo 1. Looking south towards existing Highway 17 alignment (2019/10/02)



Photo 2. Looking east near Station 14+666 (2019/10/02)



Photo 3. Standing Water near Sta 14+666 looking east (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: 45.446N 76.547W

User File Reference: Sta. 14+660 Culvert

2020-07-05 14:42 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	0.183	0.106	0.032
Sa (0.1)	0.423	0.229	0.138	0.046
Sa (0.2)	0.354	0.199	0.124	0.044
Sa (0.3)	0.269	0.155	0.099	0.036
Sa (0.5)	0.192	0.113	0.073	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)	0.161	0.091	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
Preliminary General Arrangement**

COMPARISON OF CULVERT FOUNDATION ALTERNATIVES

	Pipe Culvert	Open Bottom Culvert	Closed Box 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 installation if precast units are used. • Possibility to maintain work zone outside of existing waterway. 	<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. • Can facilitate mammal crossing
Disadvantages	<p>Numerous parallel pipes required to provide hydraulic opening equivalent to existing culvert</p> <ul style="list-style-type: none"> • Requires a temporary by-pass to maintain waterflow 	<ul style="list-style-type: none"> • May require protection system for construction of foundations. • Deepest excavation, increases quantities and dewatering concerns. • Lower geotechnical resistances. • Potential for post construction settlement. 	<ul style="list-style-type: none"> • Requires a temporary by-pass to maintain waterflow
Risks/ Consequences	<ul style="list-style-type: none"> • Potential for damage due to settlement 	<ul style="list-style-type: none"> • Increased risk of basal instability of footing excavation due to depth of excavation below water table. • Potential for damage due to settlement 	<ul style="list-style-type: none"> • Potential for damage due to settlement
Relative Cost	Low to Moderate	Moderate	Moderate
Recommendation	Feasible	Not Recommended	Recommended

DRAWING PATH AND NAME: \\mhl\local\data\offices\littles\littles\17\29 - Highway 17 Twinning\Sheet to Miller\10 Design\Structures\Lifey Creek Culvert WB\01 - General Arrangement - Precast Concrete Box Culvert With Wingwalls - Lifey Creek Culvert WB.dwg
LAST UPDATED: Jul 9, 2020-12:43
PRINTED: Jul 9, 2020-12:43

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

DIST. No.	
CONT. No.	XXXX-XXXX
WP. No.	XXX-XXX-XX
HWY 417 WB UNNAMED TRIBUTARY OF LIFFEY CREEK CULVERT GENERAL ARRANGEMENT - OPTION 1A	
SHEET 01	



- GENERAL NOTES:**
- CLASS OF CONCRETE:-----40 MPa MIN.
 - CLEAR COVER TO REINFORCING STEEL:
DECK ----- TOP ----- 50 ± 10
BOTTOM ----- 40 ± 10
REMAINDER ----- 70 ± 20
UNLESS NOTED OTHERWISE.
 - REINFORCING STEEL:
 - REINFORCING STEEL SHALL BE GRADE 400W UNLESS SHOWN OTHERWISE.
 - STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa UNLESS OTHERWISE SPECIFIED.
 - BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.
 - UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL BAR SHALL BE CLASS B.
 - BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12-1, UNLESS INDICATED OTHERWISE.

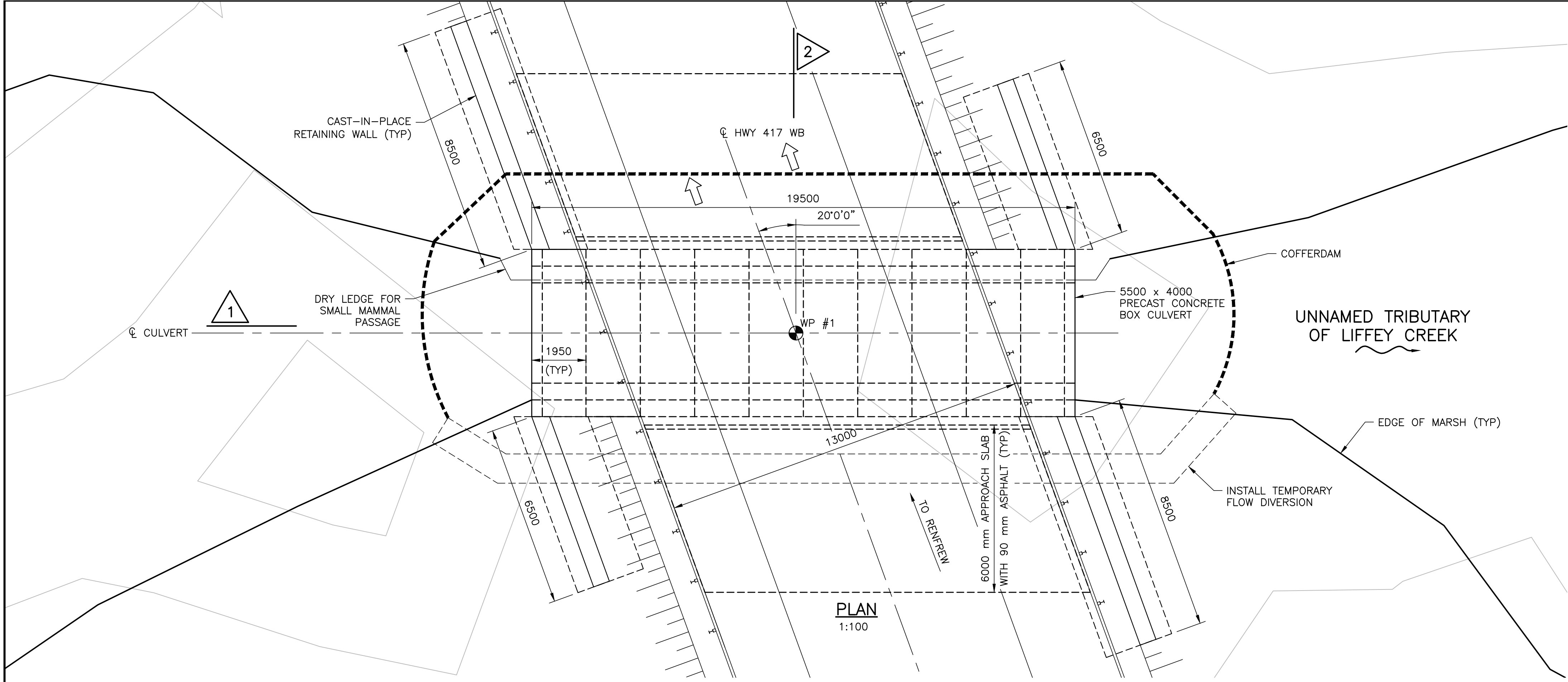
- CONSTRUCTION NOTES:**
- THE CONTRACTOR SHALL CHECK AND IDENTIFY ALL EXISTING UTILITIES WITHIN THE WORK AREA. PRIOR TO THE CONSTRUCTION WORK, THE CONTRACTOR SHALL CARRY OUT ALL NECESSARY PROTECTION AND PRECAUTIONARY MEASURES FOR OR ARRANGE TO DIVERT EXISTING UTILITIES AS MAY BE REQUIRED BY RELEVANT AUTHORITIES.

- APPLICABLE STANDARD DRAWINGS:**
- MTOD 3941.210 FIGURES IN CONCRETE, SITE NUMBER AND DATE LAYOUT
 - OPSD 3370.100 DECK, WATERPROOFING, HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD

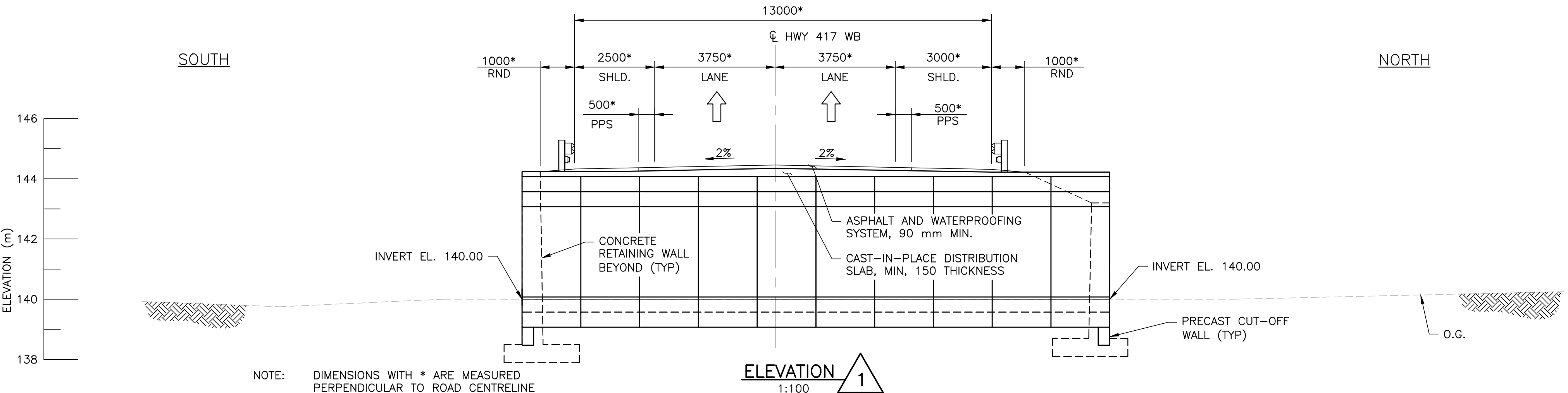
- LEGEND:**
- EARTH
 - GRANULAR BACKFILL

PRELIMINARY

- ABBREVIATIONS:**
- ABUT. - ABUTMENT
 - B - BOTTOM
 - BRGS. - BEARINGS
 - BVCE - BEGINNING OF VERTICAL CURVE ELEVATION
 - BVCS - BEGINNING OF VERTICAL CURVE STATION
 - CL - CENTERLINE
 - C.J. - CONSTRUCTION JOINT
 - CSP - CORRUGATED STEEL PIPE
 - E - EAST
 - EL. - ELEVATION
 - EQ. - EQUAL
 - EVCE - END OF VERTICAL CURVE ELEVATION
 - EVCS - END OF VERTICAL CURVE STATION
 - GRAN. - GRANULAR
 - HWL - HIGH WATER LEVEL
 - HWY - HIGHWAY
 - LOC. - LOCATION
 - N.T.S. - NOT TO SCALE
 - O.G. - ORIGINAL GROUND
 - PPS - PARTIALLY PAVED SHOULDER
 - RD. - ROAD
 - RND - ROUNDING
 - SB - SOUTH BOUND
 - SBGR - STEEL BEAM GUIDE RAIL
 - SHLD. - SHOULDER
 - STA. - STATION
 - T - TOP
 - TYP - TYPICAL
 - T/C - TOP OF CONCRETE
 - T/F - TOP OF FOOTING
 - T/P - TOP OF PAVEMENT
 - U.N.O. - UNLESS NOTED OTHERWISE
 - W - WEST
 - WB - WESTBOUND
 - WBL - WESTBOUND LANE
 - WL - WATER LEVEL
 - WP - WORKING POINT

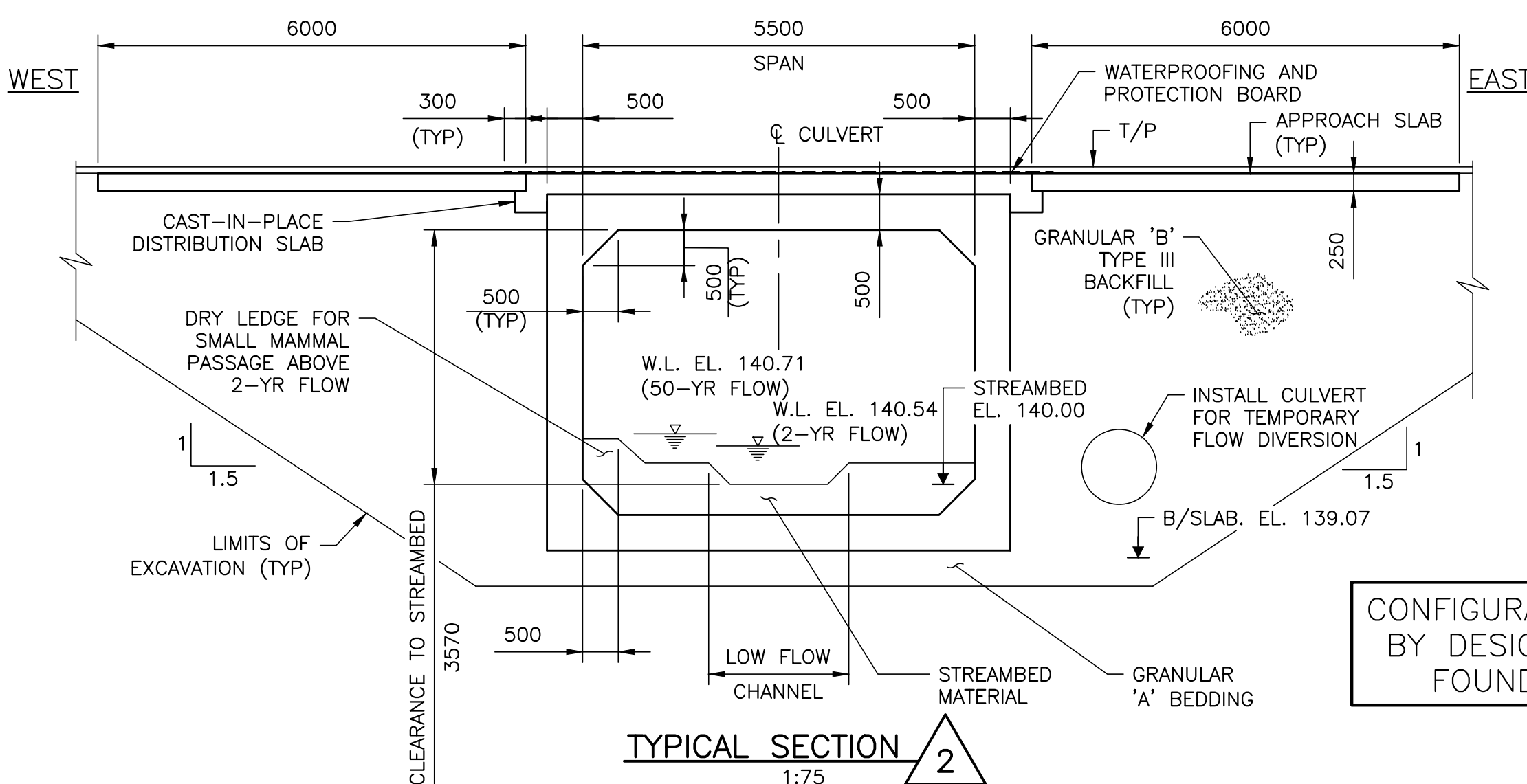


PLAN
1:100



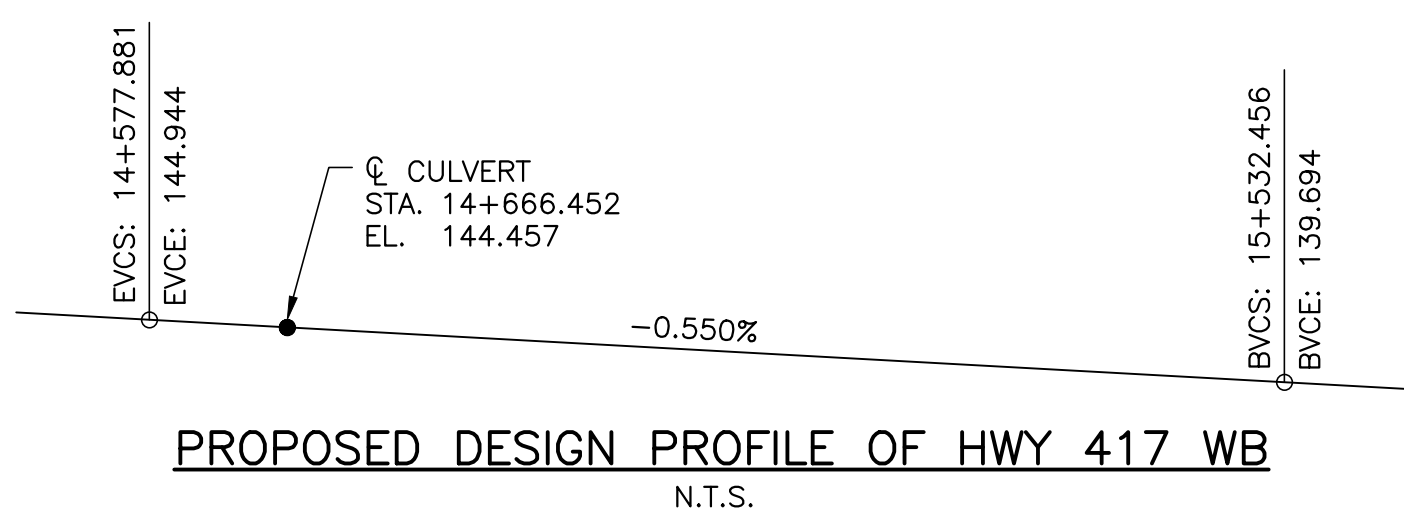
ELEVATION
1:100

NOTE: DIMENSIONS WITH * ARE MEASURED PERPENDICULAR TO ROAD CENTRELINE



TYPICAL SECTION
1:75

CONFIGURATION TO BE CONFIRMED BY DESIGN-BUILDER FOLLOWING FOUNDATION INVESTIGATION










WORKING POINT COORDINATES				
WP #	STATION	ELEVATION	NORTHING	EASTING
1	14+666.452	144.457	5034045.215	301136.926

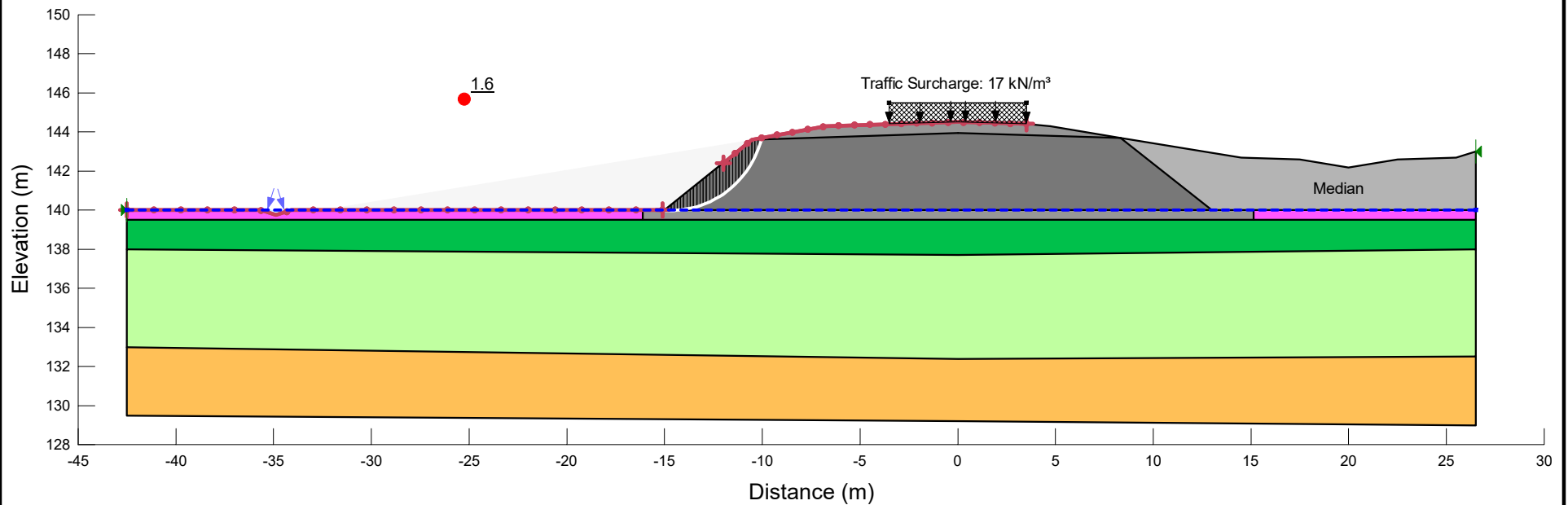
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		DATE	BY	DESCRIPTION
DESIGN	PMA	CHK. JO	CODE CHBDC 2019	LOAD CL-625-QNT
DRAWN	CTB	CHK. PMA	SITE	XX-XXX
				DATE JULY 2020
				DWG. 1

Appendix G.

Slope Stability Analysis Figures

Color	Name	Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22				0	40
	2. EARTH FILL	Mohr-Coulomb	20				0	30
	3. ROCKFILL	Mohr-Coulomb	20				0	42
	4. ORGANIC SILT (TSA)	Mohr-Coulomb	16				25	0
	5. CLAY (TSA)	Mohr-Coulomb	17.5				75	0
	6. CLAYEY SILT (TSA)	S=f(depth)	18	100	-6.4	65		
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18				0	32



Project

Highway 17 Twinning

Analysis

1. Short Term (Undrained)

Seismic Coefficient

H: g, V: g

Last Run

07/20/2021, 06:14:00 AM

Scale

1:315

Additional Details

Name: Embankment Fill Sta. 14+666

Comments: 14+666 Liffey Creek WB








Method: Morgenstern-Price, Half-Sine

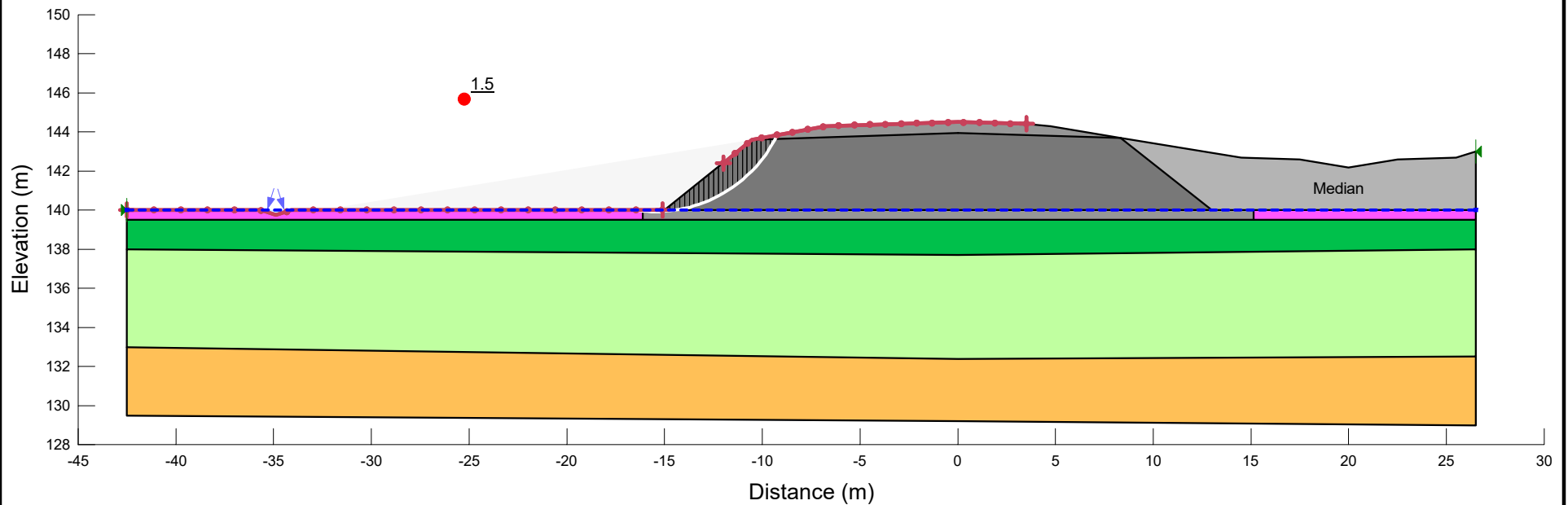
Minimum Slip Surface Depth: 1.52 m

Entry: (-10.028094, 143.69295) m, Exit: (-15.1, 140) m

Center: (-14.725865, 144.81552) m, Radius: 4.8300334 m

Fig. G1-1

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22	0	40
	2. EARTH FILL	Mohr-Coulomb	20	0	30
	3. ROCKFILL	Mohr-Coulomb	20	0	42
	4. ORGANIC SILT (ESA)	Mohr-Coulomb	16	2	20
	5. CLAY (ESA)	Mohr-Coulomb	17.5	5	28
	6. CLAYEY SILT (ESA)	Mohr-Coulomb	18	5	28
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18	0	32



Project

Highway 17 Twinning

Analysis

2. Long Term (Drained)

Seismic Coefficient

H: g, V: g

Last Run

07/20/2021, 06:14:00 AM

Scale

1:315

Additional Details

Name: Embankment Fill Sta. 14+666

Comments: 14+666 Liffey Creek WB








Method: Morgenstern-Price, Half-Sine

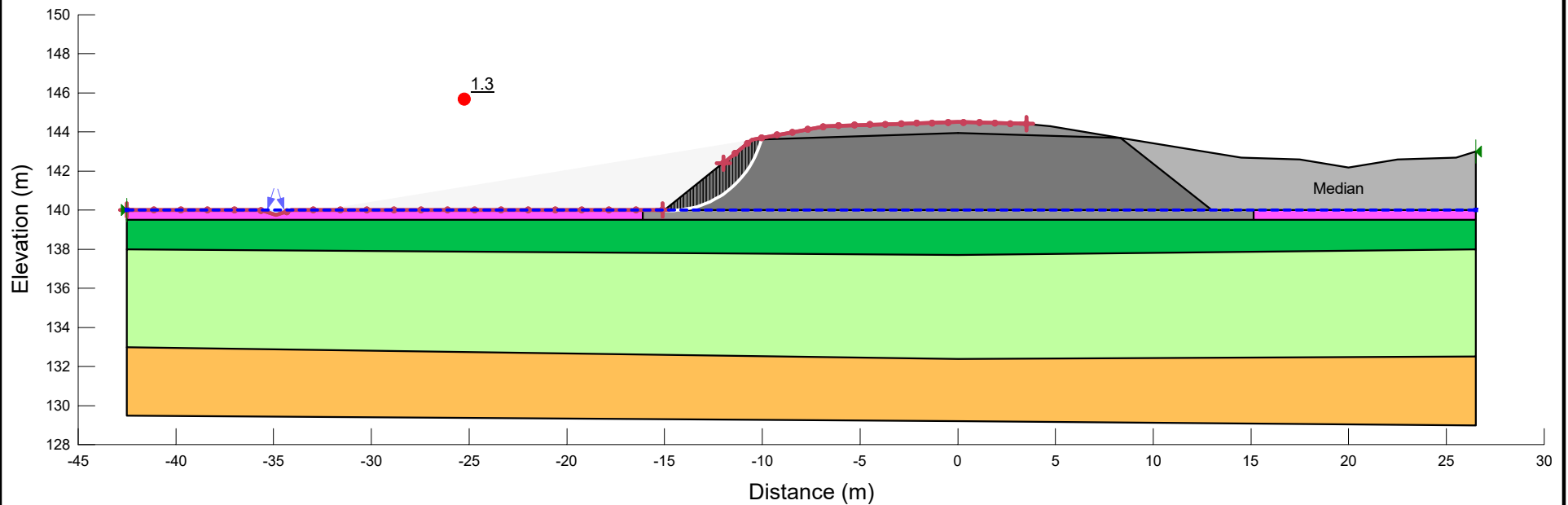
Minimum Slip Surface Depth: 1.52 m

Entry: (-9.2421573, 143.83849) m, Exit: (-16.473936, 140) m

Center: (-15.471562, 146.84315) m, Radius: 6.9161746 m

Fig. G1-2

Color	Name	Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22				0	40
	2. EARTH FILL	Mohr-Coulomb	20				0	30
	3. ROCKFILL	Mohr-Coulomb	20				0	42
	4. ORGANIC SILT (TSA)	Mohr-Coulomb	16				25	0
	5. CLAY (TSA)	Mohr-Coulomb	17.5				75	0
	6. CLAYEY SILT (TSA)	S=f(depth)	18	100	-6.4	65		
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18				0	32



Project

Highway 17 Twinning

Analysis

3. Pseudo-Static Seismic (Undrained)

Seismic Coefficient

H: 0.129g, V: 0g

Last Run

07/20/2021, 06:14:01 AM

Scale

1:315

Additional Details

Name: Embankment Fill Sta. 14+666

Comments: 14+666 Liffey Creek WB








Method: Morgenstern-Price, Half-Sine

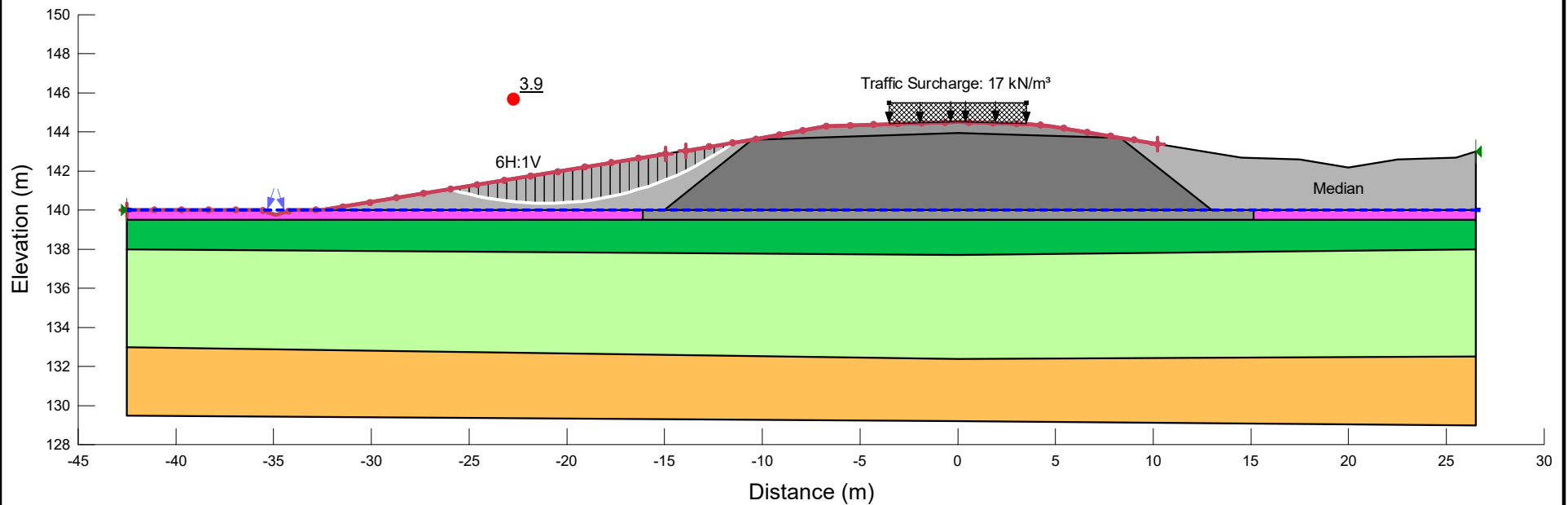
Minimum Slip Surface Depth: 1.52 m

Entry: (-10.028094, 143.69295) m, Exit: (-15.1, 140) m

Center: (-14.725865, 144.81552) m, Radius: 4.8300334 m

Fig. G1-3

Color	Name	Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22				0	40
	2. EARTH FILL	Mohr-Coulomb	20				0	30
	3. ROCKFILL	Mohr-Coulomb	20				0	42
	4. ORGANIC SILT (TSA)	Mohr-Coulomb	16				25	0
	5. CLAY (TSA)	Mohr-Coulomb	17.5				75	0
	6. CLAYEY SILT (TSA)	S=f(depth)	18	100	-6.4	65		
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18				0	32



Project

Highway 17 Twinning

Analysis

4. Short Term (Undrained) - Flattened

Seismic Coefficient

H: g, V: g

Last Run

07/19/2021, 03:15:37 PM

Scale

1:315

Additional Details

Name: Embankment Fill Sta. 14+666

Comments: 14+666 Liffey Creek WB








Method: Morgenstern-Price, Half-Sine

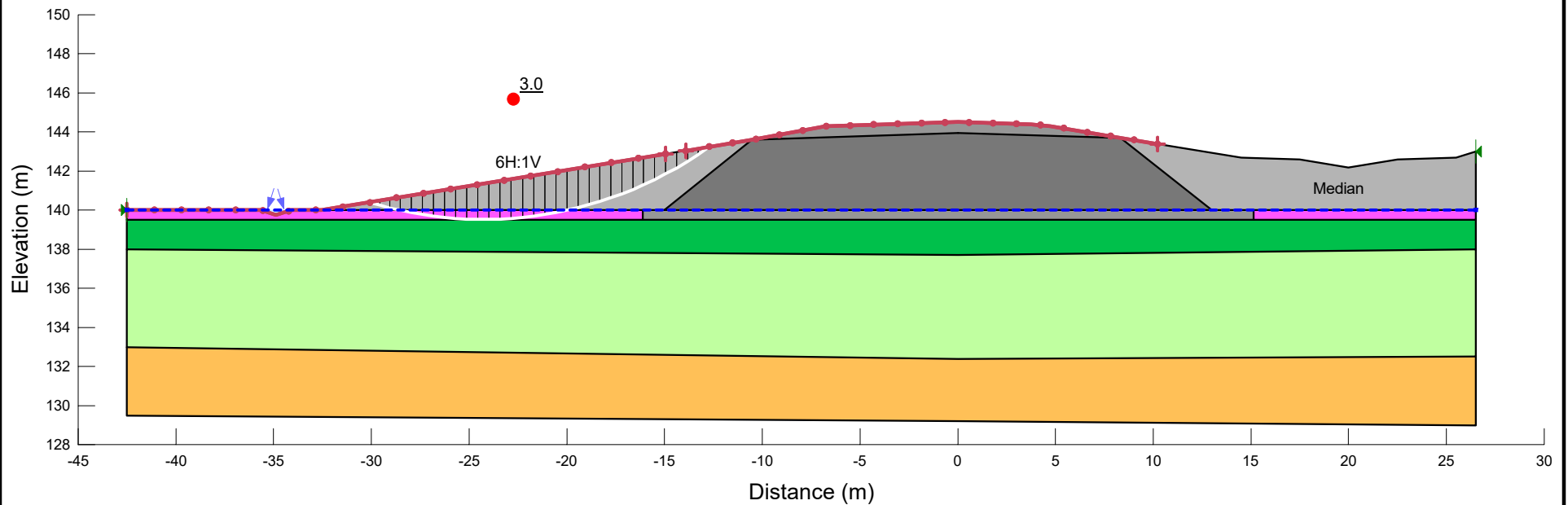
Minimum Slip Surface Depth: 1.52 m

Entry: (-11.507648, 143.44017) m, Exit: (-25.954535, 141.07828) m

Center: (-21.119817, 156.87026) m, Radius: 16.515482 m

Fig. G2-1

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22	0	40
	2. EARTH FILL	Mohr-Coulomb	20	0	30
	3. ROCKFILL	Mohr-Coulomb	20	0	42
	4. ORGANIC SILT (ESA)	Mohr-Coulomb	16	2	20
	5. CLAY (ESA)	Mohr-Coulomb	17.5	5	28
	6. CLAYEY SILT (ESA)	Mohr-Coulomb	18	5	28
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18	0	32



Project

Highway 17 Twinning

Analysis

5. Long Term (Drained) - Flattened

Seismic Coefficient

H: g, V: g

Last Run

07/19/2021, 03:15:36 PM

Scale

1:315

Additional Details

Name: Embankment Fill Sta. 14+666

Comments: 14+666 Liffey Creek WB








Method: Morgenstern-Price, Half-Sine

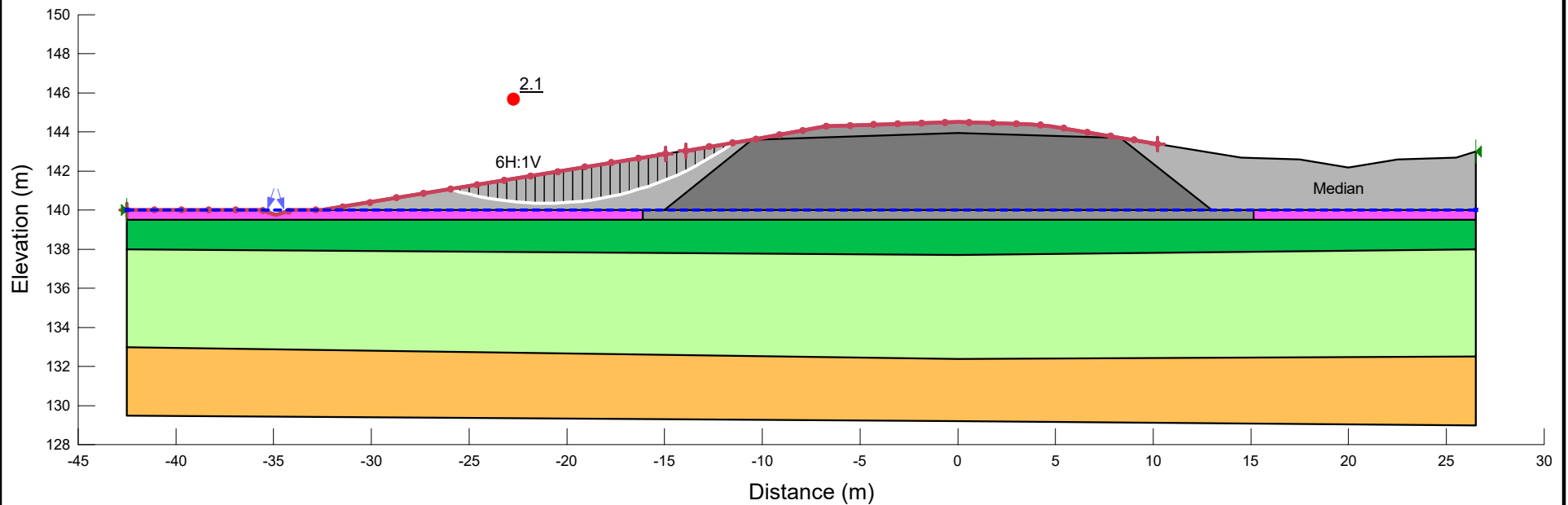
Minimum Slip Surface Depth: 1.52 m

Entry: (-12.708456, 143.24385) m, Exit: (-30.077733, 140.40419) m

Center: (-24.265023, 159.39065) m, Radius: 19.856318 m

Fig. G2-2







Color	Name	Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22				0	40
	2. EARTH FILL	Mohr-Coulomb	20				0	30
	3. ROCKFILL	Mohr-Coulomb	20				0	42
	4. ORGANIC SILT (TSA)	Mohr-Coulomb	16				25	0
	5. CLAY (TSA)	Mohr-Coulomb	17.5				75	0
	6. CLAYEY SILT (TSA)	S=f(depth)	18	100	-6.4	65		
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18				0	32

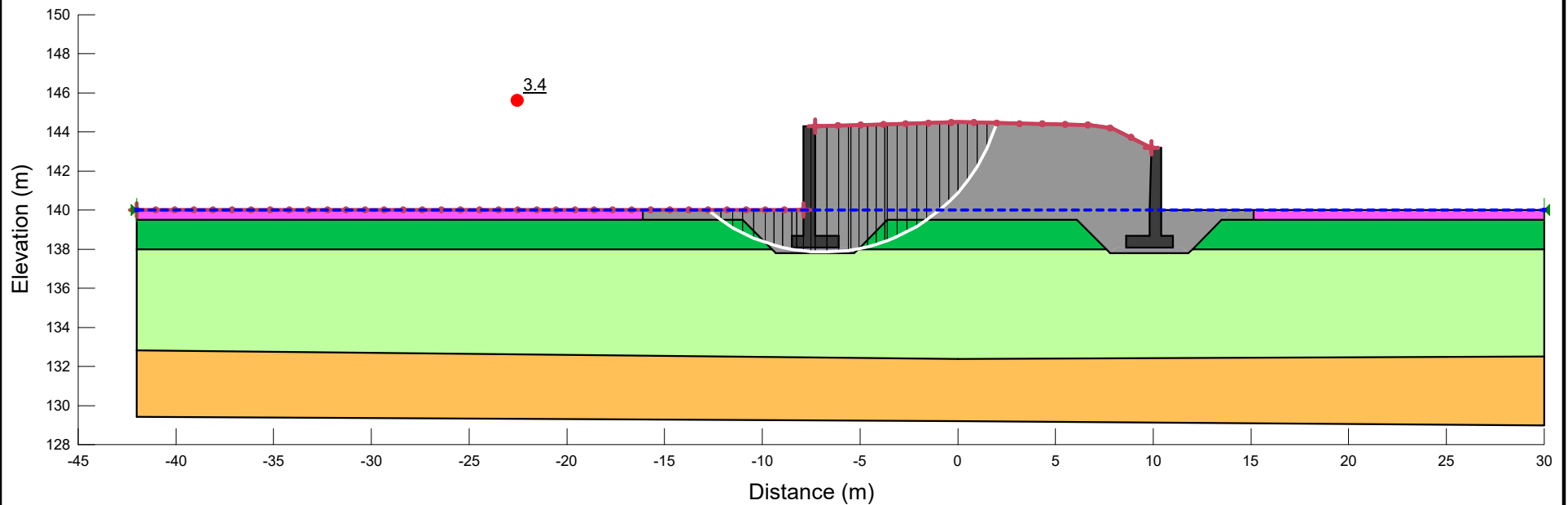


Project		
Highway 17 Twinning		
Analysis		
6. Pseudo-Static Seismic (Undrained) - Flattened		
Seismic Coefficient	Last Run	Scale
H: 0.129g, V: 0g	07/19/2021, 03:15:35 PM	1:315

Additional Details	
Name: Embankment Fill Sta. 14+666	
Comments: 14+666 Liffey Creek WB	
Method: Morgenstern-Price, Half-Sine	
Minimum Slip Surface Depth: 1.52 m	
Entry: (-11.507648, 143.44017) m, Exit: (-25.954535, 141.07828) m	
Center: (-21.119817, 156.87026) m, Radius: 16.515482 m	

Fig. G2-3

Color	Name	Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22				0	40
	4. ORGANIC SILT (TSA)	Mohr-Coulomb	16				25	0
	5. CLAY (TSA)	Mohr-Coulomb	17.5				75	0
	6. CLAYEY SILT (TSA)	S=f(depth)	18	100	-6.4	65		
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18				0	32
	8. CONCRETE	High Strength	24					



Project

Highway 17 Twinning

Analysis

R1. Short Term (Undrained)

Seismic Coefficient

H: g, V: g

Last Run

07/20/2021, 06:14:05 AM

Scale

1:315

Additional Details

Name: Retaining Wall - Horizontal

Comments: 14+666 Liffey Creek WB







Method: Morgenstern-Price, Half-Sine

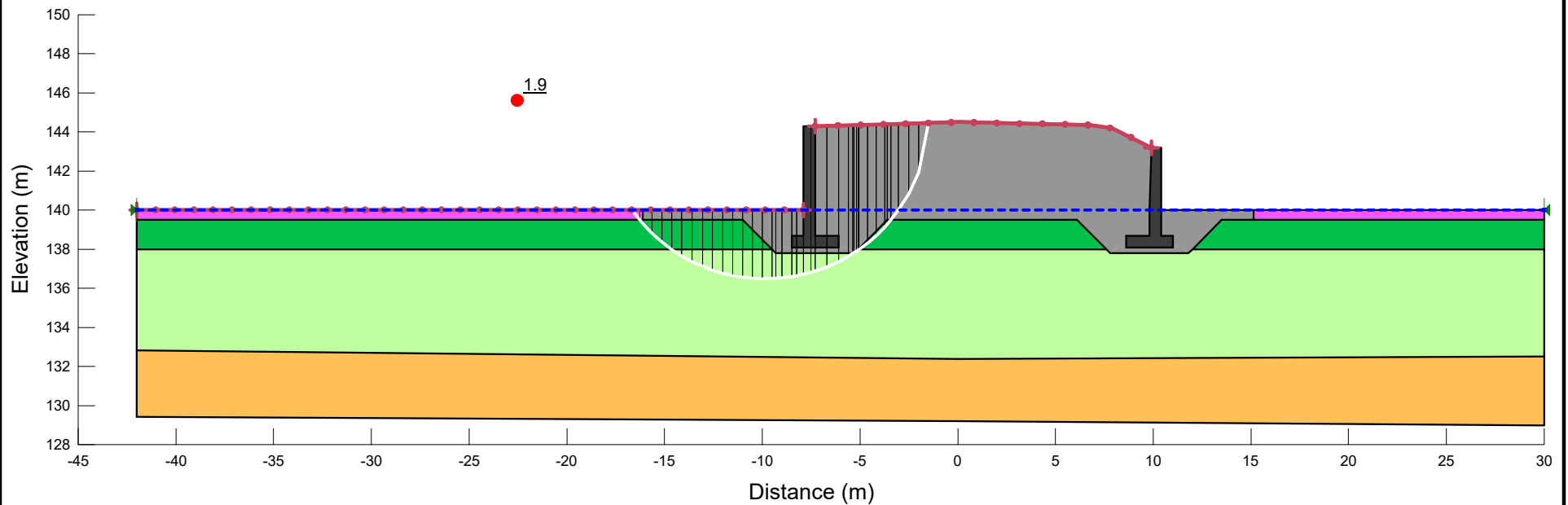
Minimum Slip Surface Depth: 0.1 m

Entry: (2.0046123, 144.45991) m, Exit: (-12.771429, 140) m

Center: (-6.8536364, 147.10094) m, Radius: 9.2435722 m

Fig. G3-1

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22	0	40
	4. ORGANIC SILT (ESA)	Mohr-Coulomb	16	2	20
	5. CLAY (ESA)	Mohr-Coulomb	17.5	5	28
	6. CLAYEY SILT (ESA)	Mohr-Coulomb	18	5	28
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18	0	32
	8. CONCRETE	High Strength	24		



Project

Highway 17 Twinning

Analysis

R2. Long Term (Drained)

Seismic Coefficient

H: g, V: g

Last Run

07/19/2021, 03:15:38 PM

Scale

1:315

Additional Details

Name: Retaining Wall - Horizontal

Comments: 14+666 Liffey Creek WB







Method: Morgenstern-Price, Half-Sine

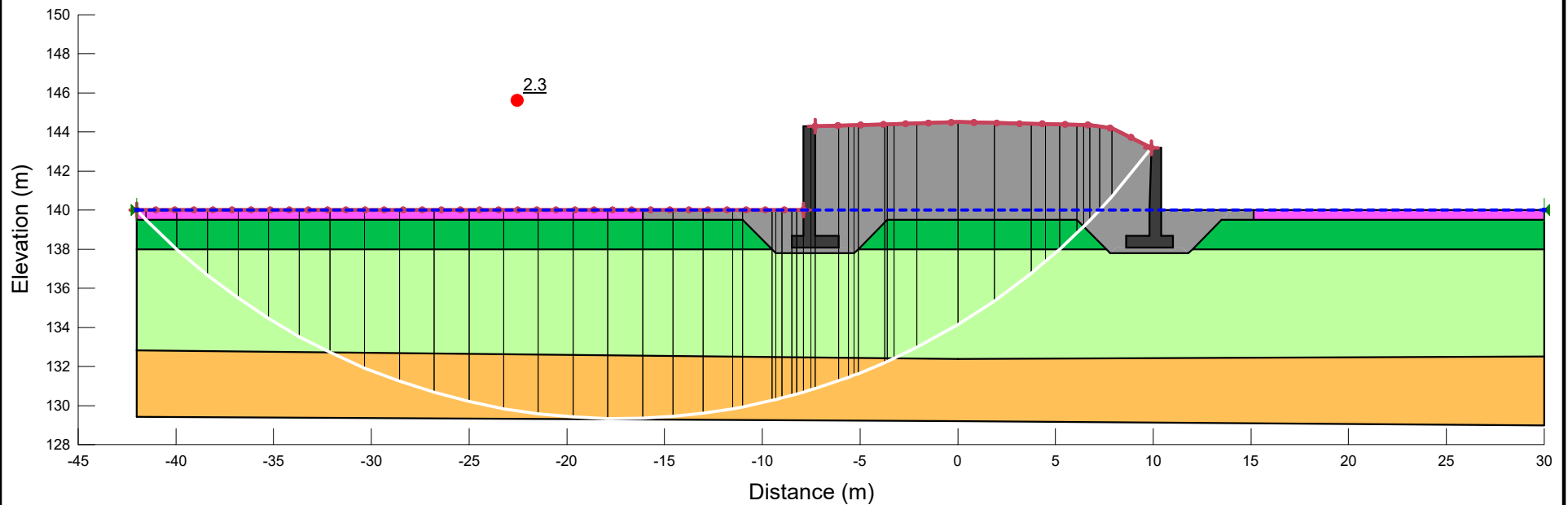
Minimum Slip Surface Depth: 0.1 m

Entry: (-1.4848121, 144.46041) m, Exit: (-16.668571, 140) m

Center: (-9.8519821, 144.86938) m, Radius: 8.3771593 m

Fig. G3-2







Color	Name	Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22				0	40
	4. ORGANIC SILT (TSA)	Mohr-Coulomb	16				25	0
	5. CLAY (TSA)	Mohr-Coulomb	17.5				75	0
	6. CLAYEY SILT (TSA)	S=f(depth)	18	100	-6.4	65		
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18				0	32
	8. CONCRETE	High Strength	24					

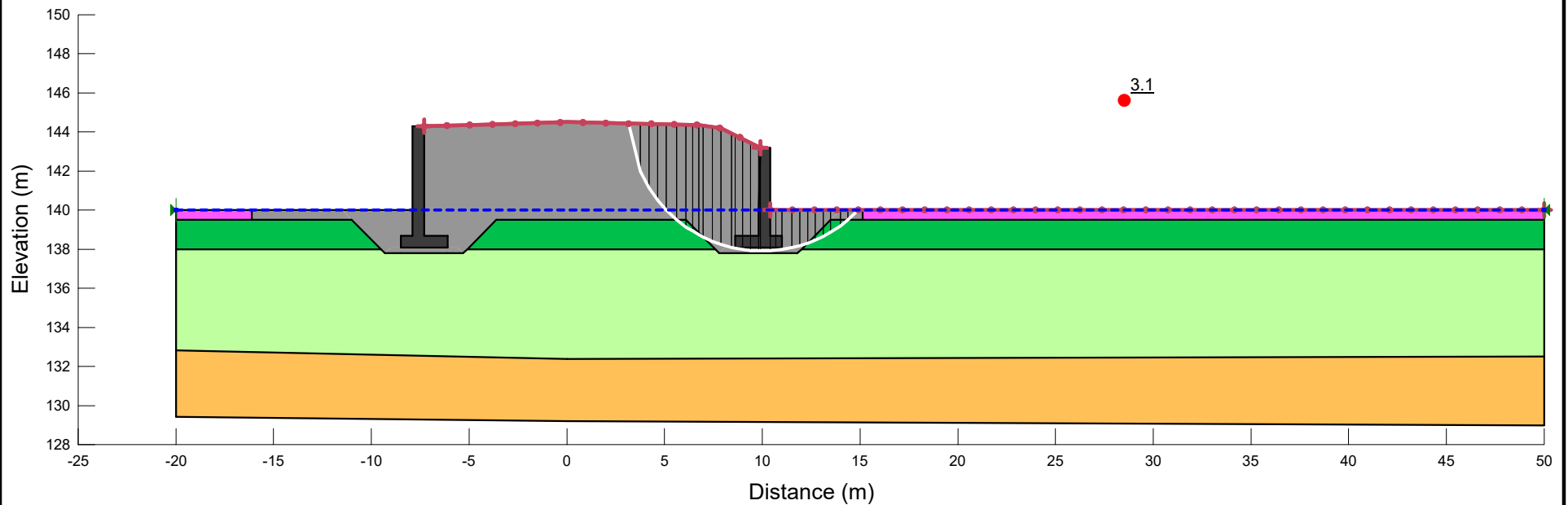


Project Highway 17 Twinning		
Analysis R3. Pseudo-Static Seismic (Unraind)		
Seismic Coefficient H: 0.129g, V: 0g	Last Run 07/19/2021, 03:15:39 PM	Scale 1:315

Additional Details	
Name: Retaining Wall - Horizontal	
Comments: 14+666 Liffey Creek WB	
Method: Morgenstern-Price, Half-Sine	
Minimum Slip Surface Depth: 0.1 m	
Entry: (9.9, 143.2) m, Exit: (-42, 140) m	
Center: (-17.375593, 163.09946) m, Radius: 33.763093 m	

Fig. G3-3

Color	Name	Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22				0	40
	4. ORGANIC SILT (TSA)	Mohr-Coulomb	16				25	0
	5. CLAY (TSA)	Mohr-Coulomb	17.5				75	0
	6. CLAYEY SILT (TSA)	S=f(depth)	18	100	-6.4	65		
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18				0	32
	8. CONCRETE	High Strength	24					



Project

Highway 17 Twinning

Analysis

SR1. Short Term (Undrained)

Seismic Coefficient

H: g, V: g

Last Run

07/20/2021, 06:14:07 AM

Scale

1:315

Additional Details

Name: Retaining Wall - 2H:1V

Comments: 14+666 Liffey Creek WB







Method: Morgenstern-Price, Half-Sine

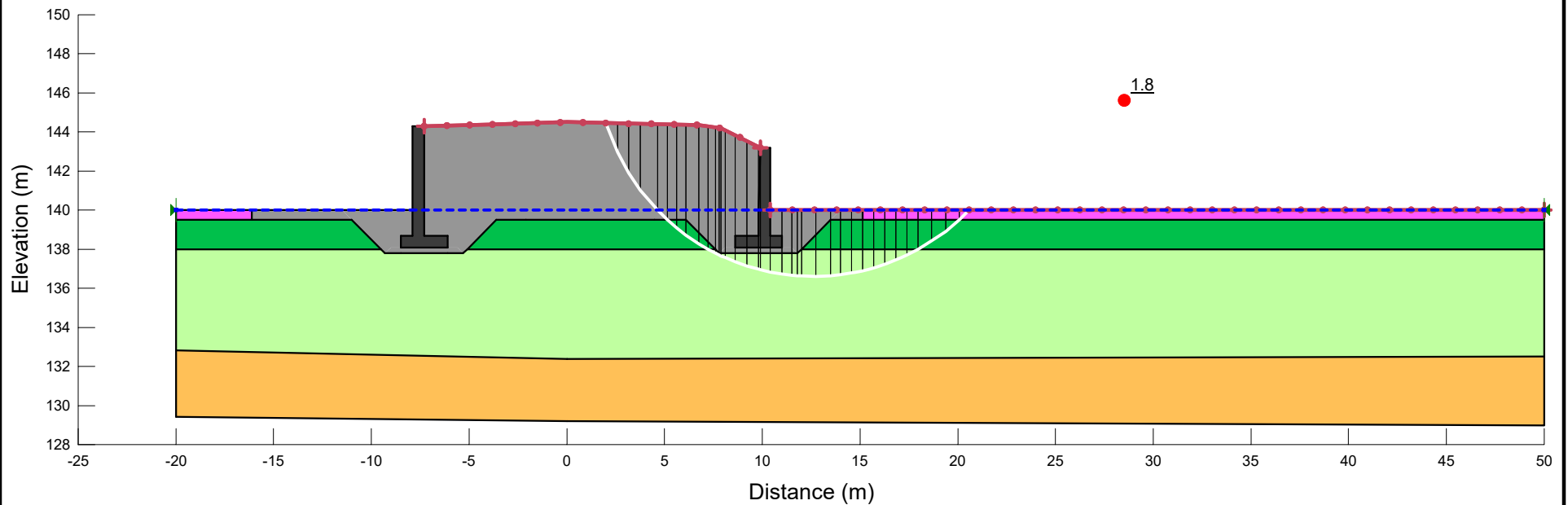
Minimum Slip Surface Depth: 0.1 m

Entry: (3.1678308, 144.43664) m, Exit: (14.925714, 140) m

Center: (10.00659, 144.76201) m, Radius: 6.846495 m

Fig. G4-1

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22	0	40
	4. ORGANIC SILT (ESA)	Mohr-Coulomb	16	2	20
	5. CLAY (ESA)	Mohr-Coulomb	17.5	5	28
	6. CLAYEY SILT (ESA)	Mohr-Coulomb	18	5	28
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18	0	32
	8. CONCRETE	High Strength	24		



Project

Highway 17 Twinning

Analysis

SR2. Long Term (Drained)

Seismic Coefficient

H: g, V: g

Last Run

07/19/2021, 03:15:41 PM

Scale

1:315

Additional Details

Name: Retaining Wall - 2H:1V

Comments: 14+666 Liffey Creek WB







Method: Morgenstern-Price, Half-Sine

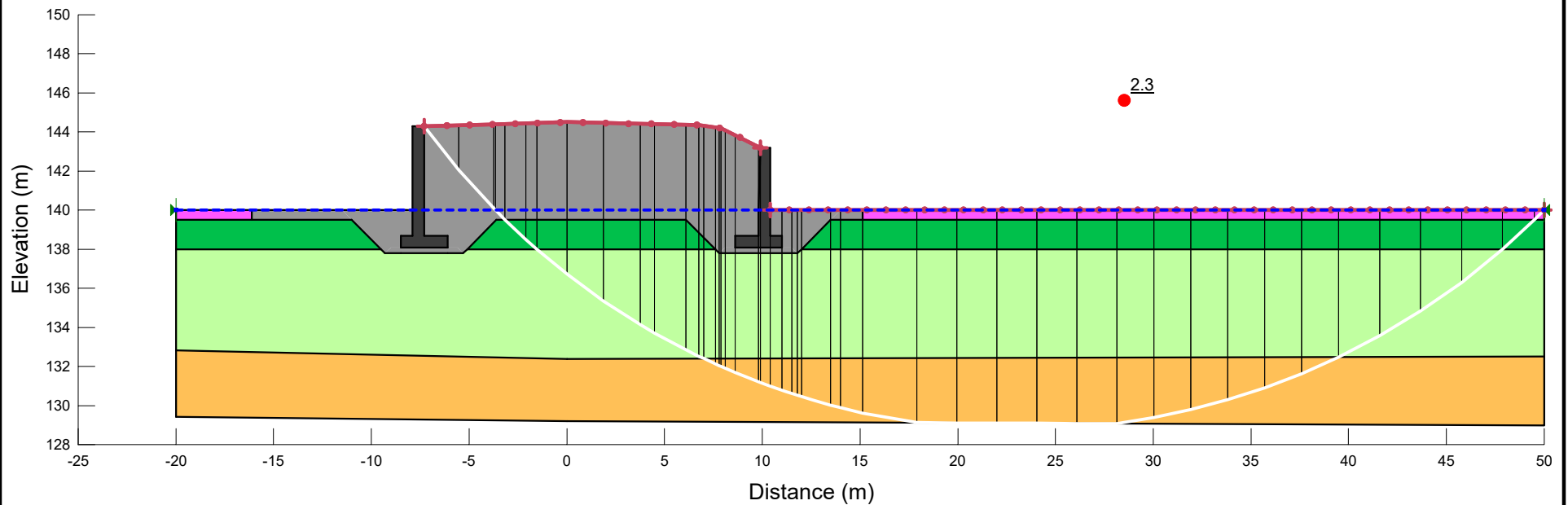
Minimum Slip Surface Depth: 0.1 m

Entry: (2.0046123, 144.45991) m, Exit: (20.582857, 140) m

Center: (12.607247, 147.70154) m, Radius: 11.087111 m

Fig. G4-2

Color	Name	Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	1. GA / GBII FILL	Mohr-Coulomb	22				0	40
	4. ORGANIC SILT (TSA)	Mohr-Coulomb	16				25	0
	5. CLAY (TSA)	Mohr-Coulomb	17.5				75	0
	6. CLAYEY SILT (TSA)	S=f(depth)	18	100	-6.4	65		
	7. CLAYEY SILTY SAND TILL	Mohr-Coulomb	18				0	32
	8. CONCRETE	High Strength	24					



Project		
Highway 17 Twinning		
Analysis		
SR3. Pseudo-Static Seismic (Unraind)		
Seismic Coefficient	Last Run	Scale
H: 0.129g, V: 0g	07/19/2021, 03:15:42 PM	1:315

Additional Details	
Name: Retaining Wall - 2H:1V	
Comments: 14+666 Liffey Creek WB	
Method: Morgenstern-Price, Half-Sine	
Minimum Slip Surface Depth: 0.1 m	
Entry: (-7.3, 144.30533) m, Exit: (50, 140) m	
Center: (23.17123, 166.39156) m, Radius: 37.63373 m	

Fig. G4-3

Appendix H.

List of Referenced Specifications Non-Standard Special Provisions

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

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates Base, Subbase, Select Subgrade, and Backfill Material
OPSS 422	Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less than or Equal to 3.0M
OPSS 1860	Geotextiles
OPSS.PROV 1860	Material Specification for Geotextiles
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSS.PROV 206	Construction Specification for Grading
SP 110S06	Amendment to OPSS 1010, April 2013 - Materials
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

2. Suggested wording for NSSPs

“Protection of Sensitive Foundation Soils”

The Contractor is advised that the native clay to clayey silt that will be exposed at the subgrade is 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 and to minimize construction and personnel traffic on the founding subgrade.

“Structural Backfill”

Structural backfill for the culvert and retaining walls shall consist of OPSS Granular B Type II or Quarry Sourced OPSS Granular A material.

“Notice to Contractor: Obstructions”

The Contractor is hereby notified that the native discontinuous tills at the site 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, installations of deep foundations and temporary protection systems.