



REPORT

Foundation Investigation and Design Report

Amable du Fond Tributary Culvert at Station 12+763 (Site No. 43X-0301/C0)

Highway 630, Lauder Twp., District of Nipissing

GWP 5208-21-00

Agreement No. 5020-E-0014 - Work item No. 8

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PART A

FOUNDATION INVESTIGATION REPORT

Amable du Fond Tributary Culvert at Station 12+763 (Site No. 43X-0301/C0)
Highway 630, Township of Lauder, District of Nipissing
GWP 5208-21-00

1.0 INTRODUCTION

WSP (formerly Golder Associates Ltd., now a member of WSP Canada Inc.) has been retained by D.M. Wills Associates Ltd. (DM Wills) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation investigation and design services for rehabilitation of the Amable du Fond Tributary Culvert along Highway 630 at Station 12+763 in the Township of Lauder, Ontario.

Based on discussion throughout the design process, we understand that the existing unnamed structural culvert (Amable du Fond Tributary Culvert) will be rehabilitated (not replaced) using a trenchless lining option. As the hydraulic capacity of the existing Amable du Fond Tributary Culvert will be reduced by the liner, we understand that additional capacity will be provided by a new overflow/bypass culvert(s) crossing Hwy 630 approximately 150 m to the north of the Amable du Fond Tributary Culvert, as required. The proposed overflow/bypass culvert(s) will replace and/or supplement existing non-structural culverts installed in relatively shallow fill (embankment height less than 2 m) using open cut methods, and the project team has determined that foundation investigation is not required at the overflow/bypass culvert(s) location.

After discussion with the project team, it was agreed that a foundation borehole be advanced near the inlet of the existing Amable du Fond Tributary Culvert to support the design of a potential headwall, if required. This report presents the results of the foundation investigation carried out for the potential headwall near the inlet of the existing Amable du Fond Tributary Culvert.

The foundation investigation services for this project have been delivered under MTO Assignment No. 5020-E-0014 as part of GWP 5208-21-00.

2.0 SITE DESCRIPTION

The orientation (i.e., north, south, east, west) stated in the text of the report is referenced to project north and, therefore, may differ from magnetic north shown on the foundation drawing. For the purposes of this report, Highway 630 is oriented in a north-south direction with the culvert positioned in a skew to the highway in a southeast - northwest direction.

The Amable du Fond Tributary Culvert is located at Station 12+763 along Highway 630 within the Township of Lauder, approximately 15.6 km south of Highway 17. The site location is shown on the key plan in Drawing 1. The existing Amable du Fond Tributary Culvert is a 3 m diameter structural plate corrugated steel pipe (SPCSP) with concrete invert paving, and is about 33.4 m long. The culvert carries the Amable du Fond Tributary watercourse below Highway 630 which flows in an east to west direction. The existing culvert invert is at about Elevation 254.6 m and Elevation 251.9 m at the inlet and outlet location.

Highway 630 has an existing two-lane cross-section with granular shoulders at the culvert location and moderate tree and shrub cover is present within and beyond the MTO right of way. Based on the survey data provided by D.M Wills on August 17, 2023, the highway grade is at approximately Elevation 259.6 m with the ground generally sloping upwards to the south and east. The ground surface at the east and west toe of the Highway 630 embankment are at approximately Elevation 255 m and 253 m, resulting in a 5 m to 6 m high embankment. The existing embankment side-slopes generally range from about 2.7 Horizontal: 1 Vertical (2.7H:1V) on the east (inlet) side to about 2.5H:1V on the west (outlet) side near the culvert location. Locally steeper side-slopes are present directly near the culvert where inferred bedrock was observed at the inlet (south side) and rock fill veneer was observed at the outlet. The ground surface conditions at the culvert location are shown in Photographs 1 to 4 following the text of this report.

Based on our site observations at the time of the field investigation and a review of the available site photographs, the existing embankment in the culvert area appears to be performing satisfactorily. There was no visual evidence of instability (i.e., soil movement) on the embankment side slopes, and no tension cracks near the embankment crest that would be indicative of instability.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation consisted of one borehole (Borehole UC-1), which was drilled on November 9, 2023. The approximate borehole location is shown on Drawing 1.

Borehole UC-1 was located at the east toe of the Highway 630 embankment, to the north of the culvert inlet. The borehole was advanced using a track mounted CME55 drilling rig using a 108 mm inside diameter hollow stem auger, NW casing with wash boring techniques and NQ coring. The drilling rig was supplied and operated by Landcore Drilling of Chelmsford, Ontario.

Soil samples were obtained using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586¹). Soil samples were generally obtained at vertical sampling intervals of about 0.76 m.

The groundwater level in the open borehole was observed during and upon completion of the drilling operations and is described on the Record of Borehole sheet in Appendix A. The borehole was backfilled in general accordance with the intent of Ontario Regulation (O. Reg.) 903, as amended, and the site conditions were restored following completion of the field work.

The field work was supervised on a full-time basis by a member of WSP's technical staff who located the borehole in the field, supervised the drilling, sampling, and logged the borehole. The soil and rock samples were identified in the field, placed in labelled containers / core boxes, and transported to WSP's laboratory in Sudbury for further examination and testing. Laboratory tests such as water content, grain size distribution analyses, Atterberg limit and uniaxial compression strength testing were carried out on selected soil samples and one rock core sample, in general accordance with MTO and/or ASTM Standards, as applicable.

One soil sample was sent to Bureau Veritas located in Sudbury, Ontario, for basic chemical analysis related to potential corrosion of buried steel and concrete elements.

The as-drilled borehole location and elevation was surveyed by a Trimble GPS unit with an accuracy meeting MTO requirements and checked by measuring the distance of the borehole to the existing culvert. The surveyed GPS coordinates were subsequently converted into the NAD 83 CSRS - MTM system. The borehole locations, including geographic coordinates, ground surface elevations referenced to Geodetic datum, and drilled depth is summarized below.

Borehole No.	NAD83 CSRS CBNv6-2010.0 – MTM Zone 10 Coordinates (Geographic Coordinates)		Ground Surface Elevation (m)	Drilled Depth (m)
	Northing (m) (Latitude (°))	Easting (m) (Longitude (°))		
UC-1	5114950.5 (46.172836)	349894.9 (-78.915978)	257.6	7.7 (including 3.6 m bedrock coring)

¹ ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on the Northern Ontario Engineering Geological Terrain Study (NOEGTS)² mapping, the regional soils at the site were deposited as an alluvial plain consisting primarily of sandy soils with silt as a secondary material and is bordered by bedrock knobs.

Based on geological mapping by the Ministry of Northern Development and Mines (MNDM)³, the site is underlain by bedrock consisting of felsic igneous granite.

4.2 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions encountered in the borehole advanced during the investigation, together with the results of laboratory tests carried out on selected soil and bedrock samples are presented on the Record of Borehole and Record of Drillhole sheets in Appendix A. The detailed results of the geotechnical laboratory tests are presented in Appendix B. The results of the in-situ field tests (Standard Penetration Test N-values), as presented in the borehole record and in Section 4, are uncorrected. The results of the analytical testing completed on a select soil sample are provided in Appendix C.

The borehole location and soil strata relative to the existing structural culvert alignment are provided in Drawing 1. The stratigraphic boundaries shown on the Record of Borehole sheet and on the interpreted stratigraphic section in Drawing 1 are inferred from observations of the drilling progress and noncontinuous soil sampling and therefore, represent transitions between soil types rather than exact planes of geological change.

The subsurface conditions will vary beyond the borehole location. A summary description of the soil deposits and groundwater conditions encountered in the borehole is provided below. It should be noted that the interpreted stratigraphy shown on Drawing 1 is a simplification of the subsurface conditions.

4.2.1 Topsoil (Fill)

An approximately 40 mm thick layer of topsoil (fill) was encountered at ground surface at Elevation 257.6 m in Borehole UC-1.

4.2.2 Clayey Silt (CL) (Fill)

A 0.6 m thick cohesive sandy clayey silt fill layer was encountered below the topsoil at Elevation 257.5 m in Borehole UC-1.

The Standard Penetration Test (SPT) 'N'-value measured within the clayey silt fill layer was 9 blows per 0.3 m of penetration suggesting a stiff consistency.

The moisture content measured on one sample of the clayey silt fill was 21%. An Atterberg limits test carried out on one sample of the clayey silt fill measured a liquid limit of about 27%, plastic limit of about 16%, and corresponding plasticity index of about 11%. The Atterberg limits test result is presented on Figure B-1 in Appendix B and indicates that the material is classified as a clayey silt of low plasticity. The results of grain size distribution testing carried out on one sample of the clayey silt fill are presented on Figure B-2 in Appendix B.

² Ministry of Natural Resources. 2005. Digital Northern Ontario Engineering Geology Terrain Study. Ontario Geological Survey, Miscellaneous Release – Data 160.

³ Ministry of Northern Development of Mines, Ontario Geological Survey – MRD126 – Revision 1 1:250 000 Scale Bedrock Geology of Ontario.

4.2.3 Silty Sand (SM) (Fill)

A 1.9 m thick layer of silty sand fill was encountered at Elevation 256.9 m beneath the clayey silt fill in Borehole UC-1.

Two SPT 'N'-values measured within the silty sand deposit were 24 and 49 blows per 0.3 m of penetration indicating a compact to dense state of compactness. A lower 'N'-value of 1 was measured at the transition between the silty sand fill and underlying native gravelly sand where wood pieces were encountered, suggesting a very soft / very loose layer is present at the fill / native interface.

The moisture content measured on one sample from this deposit was 10%. The results of a grain size distribution test carried out on one sample of the silty sand fill are presented on Figure B-3 in Appendix B.

4.2.4 Gravelly Sand (SP-SM)

A 1.5 m thick gravelly sand deposit was encountered at Elevation 255.0 m beneath the silty sand fill in Borehole UC-1.

Two SPT 'N'-values measured within the gravelly sand deposit were 1 and 38 blows per 0.3 m of penetration indicating a very loose to dense state of compactness. The lower 'N' value of 1 was encountered at the fill / native interface as described in the previous section.

The moisture content measured on one sample from this deposit was 16%. The results of a grain size distribution test carried out on one sample of the gravelly sand are presented on Figure B-4 in Appendix B.

4.2.5 Bedrock

Granite bedrock was encountered at 4.1 m depth below the native gravelly sand deposit in Borehole UC-1, corresponding to Elevation 253.5 m. Bedrock was confirmed by coring 3.6 m into the deposit and the retrieved bedrock core is described as grey, medium to coarse grained, strong, granite bedrock. A more detailed description of the condition of the bedrock core is presented in the Record of Drillhole sheet in Appendix A. A photograph of the retrieved bedrock core is shown in Figure B-5 in Appendix B.

A laboratory Uniaxial Compressive Strength (UCS) test was carried out on a selected bedrock core sample and indicates the bedrock is strong. The UCS value is presented on the Record of Drillhole sheet in Appendix A and is summarized below along with the Total Core Recovery and Rock Quality Designation / Classification.

Borehole No.	Total Core Recovery (TCR)	Rock Quality Designation (RQD)	Quality Classification (Table 3.10 of CFEM 2006)	Uniaxial Compressive Strength (MPa)	Strength Classification (Table 3.5 of CFEM 2006)
UC-1	100%	90% – 100%	Excellent	60.8	Strong

4.3 Groundwater Conditions

The groundwater level in Borehole UC-1 was measured inside the hollow stem augers before introducing water as part of NW casing / NQ coring activities. The observed groundwater level should be considered unstabilized and is shown on the borehole record and summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Groundwater Level (m)	Groundwater Elevation (m)	Date	Comments
UC-1	257.6	3.1	254.5	November 9, 2023	Inside Hollow Stem Augers Prior to Coring

The groundwater level at this site will be subject to fluctuations both seasonally and as a result of precipitation events, and will be influenced by the water level in the adjacent watercourse. The water level in the watercourse adjacent to the culvert inlet and outlet was measured to be at about Elevation 254.8 m and Elevation 251.5 m respectively in August 2023.

4.4 Analytical Testing Results

One soil sample was submitted to Bureau Veritas for chemical testing/analysis related to potential corrosion of exposed buried steel and concrete elements. The test results are provided in Appendix C and are summarized below.

Borehole No.	Sample No. / Depth (m)	Soluble Chloride (µg/g)	Soluble Sulphate (µg/g)	Conductivity (µmho/cm)	pH	Resistivity (ohm-cm)	Redox Potential (mV)
UC-1	No. 4 / 2.6	<20	30	54	5.75	19,000	440

5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. Tibor Berecz, P.Eng. under the overall direction of Mr. Matthew Thibeault, P.Eng. This Foundation Investigation Report was prepared by Mr. Tibor Berecz, P.Eng., a Geotechnical Engineer with WSP and reviewed by Mr. Matthew Thibeault, P.Eng., a Senior Geotechnical Engineer with WSP. Mr. Kevin Bentley, M.E.Sc., P.Eng., an MTO Principal Foundations Contact for this project, conducted an independent technical and quality review of the report.

Signature Page

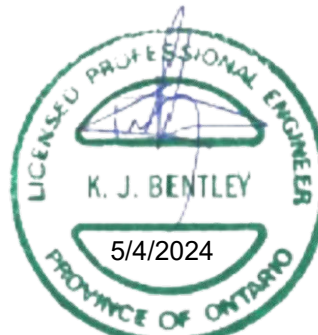
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PART B

FOUNDATION DESIGN REPORT

Amable du Fond Tributary Culvert at Station 12+763 (Site No. 43X-0301/C0)
Highway 630, Township of Lauder, District of Nipissing
GWP 5208-21-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides a discussion on the proposed rehabilitation / lining of the existing Amable du Fond Tributary Culvert (Site No. 43X-0301/C0) and provides foundation recommendations for the detailed design of the associated proposed headwall / wingwall at the inlet. The recommendations provided herein are based on interpretation of the factual data obtained from the borehole advanced during the foundation investigation. The discussion and recommendations presented are intended to provide the designers with information to assess the feasible type of headwall / wingwall and foundation alternatives.

The Foundation Design Report (Part B of this report) including the discussion and recommendations are intended for the use of the MTO and their detail designers and shall not be used or relied upon for any other purpose or by any other parties, including the future construction or design-build contractor. Contractors undertaking this work must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.2 Project Understanding

It is understood that in the Amable du Fond Tributary Culvert (Site No. 43X-0301/C0) along Highway 630 will be rehabilitated / lined using a pipe insert liner. As the hydraulic capacity of the existing Amable du Fond Tributary Culvert will be reduced by the liner, we understand that additional hydraulic capacity for the watercourse system crossing Hwy 630 will be provided by a new overflow/bypass culvert(s) approximately 150 m to the north of the Amable du Fond Tributary Culvert location, as required. The proposed overflow/bypass culvert(s) will replace and/or supplement existing non-structural culverts that cross Hwy 630 installed in relatively shallow fill (embankment height less than 2 m) using open cut methods.

As part of the Amable du Fond Tributary Culvert rehabilitation, it is anticipated that a headwall / wingwalls may be required to accommodate the redirection of water flow at the culvert inlet. For the purpose of this report, it has been assumed that the headwall / wingwalls would be of a similar height as the diameter of the existing structural culvert (i.e., a 3 m high wingwall). It is noted that if wingwalls / headwalls can be limited to less than 2 m in height, they are not designated as retaining walls and not subject to the same level of MTO acceptance criteria. There were no General Arrangement drawings available at the time this report was prepared.

6.3 Culvert Rehabilitation with Pipe Insert Liner

Based on the structural and hydraulic assessment of the Unknown Culvert by others, it is understood that trenchless "lining" of the host SPCSP using a pipe insert liner has been selected.

From a foundations perspective, the pipe insert liner option is considered feasible and design and construction should follow the MTO Trenchless Technology Design Guide titled "Pipe-Insert Liners" (dated September 2017).

Dewatering / flow diversion will need to be implemented such that lining the existing host pipe and grouting of the annulus can be completed in the dry. Recommendations for dewatering / flow diversion are included in Section 6.5.3.

6.4 Culvert Wingwall and Foundation Options

A concrete cantilever wall or retained soil system (RSS) wall could be considered as feasible options, depending on the function and final geometry of the culvert / wall system. The key features of these feasible retaining wall options are summarized below. It should be noted that the selection of the type of walls and foundation alternatives will also depend on factors beyond geotechnical/foundation recommendations (e.g. wall height and proximity to the watercourse, design scour depth, etc.).

Concrete Cantilever Wall on Shallow Foundations: A concrete headwall / wingwall supported on shallow foundations (cast-in-place concrete strip footing) could be considered for the proposed retaining wall. For the retaining wall installation, concrete strip footings should be founded below the existing fill and any soft/loose soils (i.e., founded on the dense gravelly sand or on the granite bedrock), and anticipated scour depth. The footings should also be founded below the frost depth (i.e., 1.9 m as interpreted from Ontario Provincial Standard Drawing (OPSD) 3090.101 - Foundation Frost Penetration Depths for Southern Ontario), however, given the shallow variable bedrock surface, consideration can be given to foundations bearing directly on sound bedrock within the frost depth. Based on the cross section in Drawing 1, granite bedrock is anticipated to be present less than 1 m below the invert of the existing culvert. Shallow footings placed directly on the competent fresh granite bedrock (Elevation 253.5 m in borehole) can be placed above the frost depth and can be field fit to the variable bedrock surface, and as such, are recommended at the site. The excavation footprint for the strip footings are anticipated to remain within the embankment side slope such that temporary protection systems are not required, however, this will need to be confirmed when more details are available.

Retained Soil System (RSS) Wall: An RSS wall with the front facing supported on a concrete levelling pad could be considered for the headwall / wingwalls. RSS walls include a reinforced soil zone behind the wall face with a distance (width) equal to about 70% of the wall height. Typically, RSS walls can be founded about 0.8 m below ground surface and within the frost penetration depth as they are more tolerable to freeze / thaw movement, but must be founded below the design scour depth when located adjacent to a watercourse. Given the presence of wood fragments in the existing fill and very loose zones near the fill / native soil interface, RSS walls (including the reinforced soil zone) should be founded on the dense gravelly sand or bedrock that is anticipated to be near the existing invert or about 1 m below invert, respectively. Temporary excavations to the founding stratum (i.e., dense gravelly sand) would extend to about Elevation 254 m, although it is anticipated that excavations would extent to bedrock (El. 253.5 m in borehole) at some locations due the variable bedrock surface. The excavation footprint for the reinforced soil zone is anticipated to extend about 2 m behind the face of the wall (assuming 3 m high wall). The excavation footprint for the RSS Walls are anticipated to remain within the embankment sideslope such that temporary protection systems are not required, however, this will need to be confirmed when more details are available. It should be noted that RSS wall design (for headwall / wingwalls greater than 2 m in height) near a watercourse is not currently approved in MTOs Designated Sources for Materials (DSM) Index and would likely require the Proprietary Designer / Contractor to submit a site-specific design the MTO RSS Committee for approval. The submission should include working drawings, supporting design documentation and commentary which specially address RSS embedment depth and scour protection, how to address variable bedrock surface, backfill material and control of migration of fines, performance in differential hydrostatic pressures, and structure design requirements for an equivalent service life to the culvert liner. An NSSP should be prepared and included in the Contract document to notify the Contractor of the special requirement if an RSS Wall is selected for design. It is noted that wall heights less than 2 m are not considered to be RSS Walls in MTO's DSM Index.

We understand that the need for, type and height of head wall / wing walls is still being determined by the project team; therefore, general foundation recommendations are provided for consideration at this stage. It is also noted that an inferred bedrock outcrop was observed on the south side of the culvert inlet as shown on Drawing 1. As a result, despite what wingwall / headwall option is selected, the bedrock surface is anticipated to be variable and this will need to be incorporated into the design.

6.5 General Foundation Design Considerations

6.5.1 Consequence and Site Understanding Classification

A “typical consequence level” is considered appropriate for the headwall / wingwall at the culvert inlet for this project, as outlined in Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2019) and its Commentary. Further, given the level of foundation investigation and laboratory testing completed to date as presented in Section 4.0, a “typical degree of site and prediction model understanding” has been utilized. Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC have been used for design.

For seismic design, the consequence factor, ψ , and resistance factor, ϕ_{gu} , should be taken as unity, as per Section 6.14.4 of CHBDC.

6.5.2 Seismic Design

The seismic hazard values associated with the design earthquakes are those established for the National Building Code of Canada (NBCC 2020) by the Geological Survey of Canada (GSC). The current seismic hazard maps (referred to as the 6th generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2020.

6.5.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation. Based on the relatively shallow bedrock stratum below the potential founding level, the site may be classified as Site Class C in accordance with Clause 4.4.3.2 and Table 4.1 of CHBDC (2019), in the absence of site-specific geophysical testing. Geophysics testing such as Multi-Channel Analysis of Surface Waves (MASW) or vertical seismic profiling may be considered to refine the average shear wave velocity and improve the Site Class.

6.5.2.2 Spectral Response Values

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the proposed structure, the Site Class C peak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca) are provided below.

Parameter	2% Probability of Exceedance in 50 Years (2,475-year return period) (g)
PGA	0.235
Sa(0.2)	0.445
Sa(0.5)	0.275
Sa(1.0)	0.147
Sa(2.0)	0.0679
Sa(5.0)	0.0178
Sa(10.0)	0.00593
PGV [m/s]	0.184

6.5.3 Soil Liquefaction

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil which may lead to potentially large surface deformations, and under undrained conditions generate excess pore water pressures that can lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (analogous to slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of slopes often referred to as “flow slides”.

In general, the fill materials and native soils at this culvert site between the potential foundations and the bedrock will generally consist of compact to dense sand to gravelly sand. Based on the thickness, compactness and composition of the soils it is considered to have a low potential for liquefaction during a seismic event.

6.5.4 Frost Protection

The frost penetration depth in this area is approximately 1.9 m as interpreted from Ontario Provincial Standard Drawing (OPSD) 3090.101 (Foundation Frost Penetration Depths for Southern Ontario). As such, the footings for retaining walls should be provided with a minimum of 1.9 m of soil cover or an equivalent combination of insulation and soil cover, unless the retaining wall type can tolerate the anticipated ground movements from freeze / thaw cycles for service life of the structure.

If the proposed foundations are to be founded directly on the bedrock surface for cast-in-place alternatives, it is not necessary for the foundations to be below the depth of frost penetration.

6.6 Concrete Cantilever Wall (Cut-Off Wall)

A concrete cantilever wall founded on shallow foundations could be considered for the potential headwall / wingwalls at the culvert inlet location. The cantilever wall backfill should consist of material meeting OPSS.PROV 1010 Granular ‘A’ or Granular ‘B’ Type I or II, placed and compacted in accordance with OPSS.PROV 501 (*Compacting*).

Further, if the wall and shallow foundations require cutting into the existing slope, the back of the excavation should be keyed into the existing embankment by benching, as per OPSD 208.010 (Benching of Earth Slopes).

6.6.1 Founding Elevations

The shallow foundations could be founded on the properly prepared dense native gravelly sand soils or on properly prepared bedrock. Where footings are placed directly on bedrock, the rock surface should be properly cleaned and all loose material removed prior to pouring foundation concrete. Benching by hoe ram or controlled blasting may be required to achieve a level founding surface.

The subgrade for the shallow foundations should be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS.PROV 902 (*Excavating and Backfilling Structures*), as modified by Amendment 109S61 (Dewatering and Protection Systems), to check that all existing fill and/or other unsuitable material have been removed. Where bedrock is not exposed and sub-excavation of unsuitable soils is required, the sub-excavated area should be backfilled with granular material meeting OPSS.PROV 1010 Granular ‘A’ or Granular ‘B’ Type II, placed and compacted in accordance with OPSS.PROV 501 (*Compacting*). Alternatively, backfill may consist of unshrinkable fill as per OPSS.PROV 578.

6.6.2 Geotechnical Resistance

Assuming that the concrete cantilever wall will be founded on the gravelly sand below the frost depth (i.e., at least 1.9 m embedment depth) or on the fresh bedrock, the factored ultimate and serviceability geotechnical resistances given below may be used for design.

Headwall / Wingwall Location	Estimated Footing Dimensions (width × length) (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement (kPa)
East Side (Inlet)	2 x 5	500	500

1) For cast in place concrete footings found on fresh bedrock, geotechnical resistance values would exceed 500 kPa.

The geotechnical resistances provided above are given for loads applied perpendicular to the subgrade surface. Where the load is not applied perpendicular to this surface, inclination of the load should be considered in accordance with Section 6.10.2 of the *CHBDC* (2019).

6.6.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the subgrade and concrete footing should be calculated in accordance with Section 6.10.4 of the *CHBDC* (2019), applying a consequence factor of 1.0 and a geotechnical resistance factor of 0.8. The unfactored coefficients of friction, $\tan \phi'$, for the concrete footing on the properly prepared subgrade may be taken as summarized below.

Retaining Wall Area	Subgrade Material	Coefficient of Friction, $\tan \phi'$
East Side (Culvert Inlet)	Precast concrete on dense gravelly sand	0.4
	Cast-in-place concrete on dense gravelly sand	0.58
	Cast-in-place concrete on bedrock	0.7

Where additional lateral restraint is needed or if the bedrock is found to be sloping at greater than 10 degrees, the base of the concrete footing should be doweled into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout, and steel. Where the rock mass is stronger than the concrete (as is the case at this site), the design of the dowels into the rock may be handled in the same way as a dowel embedment into concrete for a uniaxial compressive strength of grout similar to that of concrete. Dowels should have a minimum 1 m embedment into fair quality (i.e., RQD >50%) bedrock and the structural strength of the grout should not be exceeded. If dowels are incorporated into the design, a Non-Standard Special Provision for Dowels into Rock should be included in the Contract Documents; an example NSSP is provided in Appendix D.

6.7 Retained Soil System (RSS) Wall

Mechanically Stabilized Earth (MSE) retaining systems such as Retained Soil System (RSS) walls could be considered for the potential headwall / wingwalls near the culvert inlet location. RSS walls include a reinforced soil zone behind the wall face with a distance (width) equal to about 70% of the wall height. The reinforced soil zone (i.e., the reinforced soil mass) is comprised of reinforcing strips and RSS backfill. The RSS backfill should consist of material meeting OPSS.PROV 1010 Granular 'A' or Granular 'B' Type I or II, placed and compacted in accordance with OPSS.PROV 501 (*Compacting*).

The design of the RSS walls will be the responsibility of the proprietary wall designer and should be designed for a low performance rating in accordance with the MTO RSS Design Guidelines (2008). The proprietary design should also be in general accordance with the 2019 CHBDC Section 6.19, MTO SP 599 S22 (General RSS Specification) and MTO SP 599 S23 (Concrete Elements), as applicable.

Further, if the wall and soil mass require cutting into the existing slope, the back of the excavation / reinforced soil mass should be keyed into the existing embankment by benching, as per OPSS 208.010 (Benching of Earth Slopes).

6.7.1 Founding Elevations

A typical RSS wall has a front facing panel system that is supported on a concrete levelling pad placed at a shallow depth below the ground surface at the front of the wall. In general accordance with the MTO RSS Design Guidelines (2008), a concrete levelling pad for the face material is typically founded on a minimum 0.3 m thick compacted granular pad, the base of which is a minimum of 0.8 m below the ground surface. The compacted granular pad should consist of granular material meeting OPSS.PROV 1010 Granular 'A', Granular 'B' Type I or II, placed and compacted in accordance with OPSS.PROV 501 (*Compacting*).

The compacted granular pad and reinforced soil mass are recommended to be founded below the fill and very loose fill / native interface, and on the dense gravelly sand or bedrock.

The subgrade for the granular pad and reinforced soil mass should be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*), as modified by Amendment 109S61 (Dewatering and Protection Systems), to check that all fill and any unsuitable materials have been removed. Where sub-excavation of fill or unsuitable materials is required, the sub-excavated area should be backfilled with granular material meeting OPSS.PROV 1010 Granular 'A', Granular 'B' Type I or II, or SSM placed and compacted in accordance with OPSS.PROV 501 (*Compacting*), or the thickness of the granular pad increased to the full sub-excavation depth. Consideration could also be given to backfilling with unshrinkable fill as per OPSS.PROV 578.

6.7.2 Geotechnical Resistance

Assuming that the RSS wall will act as a reinforced unit (reinforced soil mass width taken as equal to 0.7 times the wall height) with the base of the leveling pad founded a minimum 0.8 m below the culvert invert on the dense gravelly sand or bedrock, the factored ultimate and serviceability geotechnical resistances given below may be used for design.

Retaining Wall Area	Estimated RSS Wall Dimensions (width × length) ¹ (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement (kPa)
East Side (Culvert Inlet)	2 x 5	500	500

- 1) The wall width is equal to the estimated minimum reinforcing strip length of about 70% of the estimated wall height (3m) which achieves the required minimum factor of safety for global stability. Longer strip lengths may be required by the proprietary designer to address internal stability of the wall, or if the geometry is modified such that there is sloping ground above the wall.
- 2) For RSS walls founded on bedrock, geotechnical resistance values would exceed 500 kPa.

The geotechnical resistances provided above are given for loads applied perpendicular to the subgrade surface. Where the load is not applied perpendicular to this surface, inclination of the load should be considered in accordance with Section 6.10.2 of the *CHBDC* (2019).

6.7.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the compacted granular pad and the subgrade should be calculated in accordance with Section 6.10.4 of the *CHBDC* (2019), applying a consequence factor of 1.0 and a geotechnical resistance factor of 0.8.

The coefficient of friction, $\tan \phi'$, for the compacted granular pad on the properly prepared subgrade may be taken as summarized below. The coefficient of friction value should be reviewed and revised, if necessary, by the proprietary RSS wall designer.

Retaining Wall Area	Subgrade Material	Coefficient of Friction, $\tan \phi'$
East Side (Culvert Inlet)	Compacted Granular Pad (Granular 'A', or Granular 'B' Type I or II) on dense gravelly sand	0.58
	Compacted Granular Pad (Granular 'A', or Granular 'B' Type I or II) on bedrock	0.65

6.8 Lateral Earth Pressures

The lateral earth pressures acting on the headwall / wingwalls will depend on the type and method of placement of backfill materials, the nature of the soils behind the backfill, the magnitude of the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the wall.

The following recommendations are made concerning the design of the walls. These design recommendations and parameters assume level backfill and ground surface behind the wall. If there is sloping ground behind the wall, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and OPSD 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall, in accordance with *CHBDC* (2019) Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.
- For a retaining wall constructed on shallow foundations (i.e., an unrestrained wall), fill should be placed within the wedge-shaped zone defined by a line drawn flatter than 1 horizontal to 1 vertical (1H:<1V) extending up and back from the rear face of the footing in accordance with Figure C6.31(b) of the *Commentary to the CHBDC* (2019).

The static lateral earth pressure parameters below assume level backfill and ground surface behind the retaining wall. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope in accordance with the equations provided in CHBDC Section C6.12.1, Figures C6.28 (for active earth pressure), and Section C6.12.2.2 (for at-rest earth pressure). For an unrestrained retaining wall, the pressures are based on the properties of the granular backfill, and the following parameters (unfactored) may be used:

Soil Type	Bulk Unit Weight, γ (kN/m ³)	Internal Angle of Friction ϕ (degrees)	Lateral Earth Pressure Coefficients ⁽¹⁾		
			At Rest, K_o	Active, K_a	Passive, K_p ⁽²⁾
New Granular 'A' or 'B' (Type I or II) Fill	22	35	0.43	0.27	3.69
Existing Embankment Sandy Clayey Silt Fill (Stiff)	19	30	0.5	0.33	3.00
Existing Embankment Silty Sand Fill (Compact to dense)	20	32	0.47	0.31	3.26
Gravelly Sand (Dense)	19	34	0.44	0.28	3.54

- 1) The lateral earth pressure coefficients presented above are based on a horizontal surface behind the excavation. If sloped surfaces are present, the coefficients should be corrected accordingly.
- 2) The total passive resistance below the base of the wall may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.27 of the CHBDC (2019) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

If the wall support allows for lateral yielding, active earth pressures may be used in the geotechnical design of the retaining wall. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.12 of the *Commentary of the CHBDC* (2019).

If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.9 Global Stability

Assuming the height of the proposed headwall / wingwalls is less than 3 m and the foundations are supported on a thin layer of dense gravelly sand over bedrock, or directly on bedrock, global stability is not considered to be a concern. The internal stability of any proprietary retaining wall system must be checked by the wall designer.

6.10 Construction Considerations

6.10.1 Open-Cut Excavations

At the location of the head wall / wingwalls, excavations for the granular pad / reinforced soil mass or concrete footing are anticipated to extend through the fill and native gravelly sand, and likely expose the granite bedrock. Any organics, fill and any other deleterious materials encountered within the footprint of the proposed granular pad / reinforced soil mass or concrete footing should be sub-excavated and replaced with OPSS.PROV 1010 (*Aggregates*) Granular 'A', Granular 'B' Type I, Granular 'B' Type II or SSM.

All excavations must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended.

The existing fills and native gravelly sand soils to be excavated can be classified according to OHSA as Type 3 soils above the water level. The localized very loose zone at the fill / native interface and the existing fills and native gravelly sand below the water table can be classified as Type 4 soils. Temporary excavations (i.e., those open for a relatively short time period) should be made with side slopes of 1H:1V or flatter for Type 3 soils and 3H:1V for Type 4 soils.

Temporary excavations should be observed during construction by qualified geotechnical personnel to confirm that the soil and groundwater conditions are as anticipated. If unexpected conditions are encountered, a qualified geotechnical engineer should review the excavation plan based on the changed conditions.

6.10.2 Temporary Protection Systems

Temporary protection systems are likely not required for the staged construction for retaining wall construction. Depending on the final location and geometry of the retaining walls, if required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System), as amended by SP 105S09. The lateral movement should meet Performance Level 2 provided that any existing adjacent utilities, if present, can tolerate this magnitude of deformation.

The selection and design of the protection system will be the responsibility of the contractor. Driven, interlocking sheet pile systems or a soldier pile and lagging system could be considered at this site, although the feasibility of these systems may be limited due to the shallow sloping bedrock.

The water level / groundwater level was measured to be at about Elevation 254.8 m near the culvert inlet; therefore, if the excavation depth extends below this elevation, it would be necessary to control surface water / groundwater seepage or include measures to mitigate loss of soil particles through the lagging boards. The sheet piles or soldier piles would have to be driven to sufficient depth to provide the necessary passive resistance for the retained soil height, including any surcharge loads behind the protection system within at least a 1H:1V zone relative to the base of the excavation. Lateral support to the sheet piles or soldier piles could be provided in the form of rakers or temporary anchors. Given the shallow bedrock depth, consideration will need to be given to socketing the protection system into the bedrock.

Consideration could be given to either partial or full removal of the protection system upon completion of construction or each stage of construction (as required). Where possible, full removal of the protection system should be considered to mitigate potential impediments to future rehabilitation/reconstruction work at the retaining wall locations. If the temporary protection system is left in place, it should be cut off at or below frost depth, not less than 1.9 m below the ground surface.

6.10.3 Groundwater and Surface Water Control

The groundwater level in the vicinity of the culvert inlet was measured to be about Elevation 254.5 m and the water level near the culvert inlet was measured to be at Elevation 254.8 m in August 2023. Given the diameter of the culvert and visual evidence of staining near the springline, water / groundwater levels are anticipated to be significantly higher throughout the year. Excavations for the headwall / wingwall foundation construction are anticipated to extend to the bedrock level that was measured to be at Elevation 253.5 m, about 1.3 m below the measured water level.

The foundation excavations for wingwalls / headwalls and interior of the existing host pipe must be maintained in a dry condition to facilitate foundation and lining construction (e.g. placement and compaction of granular pad, placement of unshrinkable fill and/or concrete, and placement of grout within annulus of pipe insert liner operation).

Based on the conditions encountered in the borehole near the culvert inlet and the anticipated depth of the excavations, seepage and groundwater flow rates can likely be maintained below the threshold for registration in MECP's Environmental Activity and Sector Registry (EASR), although the effectiveness of the flow diversion system and multiple concurrent water-takings within the contract may result in volumes between 50,000 L/day and 400,000 L/day, requiring such registration.

Surface water should be directed away from the excavations at all times and will need to be diverted away from the host pipe during lining activities (especially during grouting of the pipe insert liner). Temporary flow passage provisions will be required to maintain flow of the watercourse during construction of the headwalls / wingwalls and lining operations. This is expected to consist of pumping or diverting flow from behind cofferdams across the existing watercourse upstream of the inlet. The cofferdams may be comprised of inflatable bladders, rows of sandbags containing an impermeable layer, or driven steel sheet piles; as discussed in Section 6.5.2, the presence of shallow sloping bedrock may impede the installation of driven sheet piles.

In accordance with OPSS.PROV 902 (Excavating and Backfilling - Structures), as modified by Special Provision 109S61 (Dewatering and Protection Systems), dewatering and temporary flow passage systems shall be according to OPSS517 (as amended by Special Provision 517F01). Given the cohesionless subgrade and relatively shallow bedrock conditions encountered at this site, as well as the absence of any settlement-sensitive infrastructure in the vicinity of the culvert, the risk of settlement impacts due to dewatering is considered low from a foundation perspective. As such, the Foundation fill-in in Table 1 of Special Provision 517F01 should indicate that a preconstruction survey is not applicable and the minimum lowered groundwater depth below base of excavation or work area should be 1.0 m or to competent bedrock.

6.10.4 Analytical Testing of Construction Materials

The summary results of analytical tests carried out on one sample of the gravelly sand deposit are presented in Section 4.4 and on the Certificate of Analysis in Appendix C.

The analytical test results for sulphate were compared to CSA A23.1 Table 3 (*Additional requirements for concrete subjected to sulphate attack*) to assess the potential severity of sulphate attack on concrete during its service life. The sulphate concentration measured on the soil sample is less than 0.2%, which is below the Moderate degree of exposure (i.e., below the Class S3 exposure limits), and the degree of sulphate attack is considered "Negligible" according to Table 7.2 in MTO's Gravity Pipe Design Guidelines (2014). Therefore, based on the soil sample tested, when the designer is selecting the exposure class for the concrete structure, the effects of sulphates from within the site soils in contact with any portion of the proposed structure constructed below the ground surface may not need to be considered.

The analytical test results of the soil sample for resistivity were also compared to Table 3.2 of MTO's Gravity Pipe Design Guidelines (2014), to assess the relative level of corrosion potential on buried steel in contact with soil. The measured resistivity value of 19,000 ohm-cm indicates the soil corrosiveness is less than "Very low" (10,000 ohm-cm > R > 6,000 ohm-cm).

It is also noted that the measured pH level was about 5.75. Referring to the Gravity Pipe Design Guidelines (2014), a pH level between 5.5 and 8.5 is not considered detrimental to culvert durability.

Given that the proposed structure will be exposed to de-icing salt/chemicals, consideration should be given by the designer to designing the concrete structure for a "C" type exposure class as defined by CSA A23.1 Table 1.

Ultimately, it is the structural designer's decision to determine the appropriate construction materials, including the exposure class, and ensuring that all aspects of CSA A23.1-14 (2014) Section 4.1.1 "Durability Requirements" are followed when designing concrete elements, as applicable.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Tibor Berecz, P.Eng. a Geotechnical Engineer and reviewed by Mr. Matthew Thibeault, P.Eng., a Senior Geotechnical Engineer. Mr. Kevin Bentley, M.E.Sc., P.Eng., an MTO Principal Foundations Contact for this project, conducted an independent technical and quality review of the report.

Signature Page

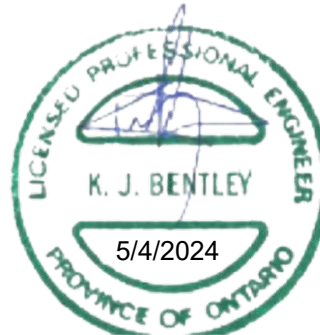
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Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

Canadian Standards Association (CSA), 2019. Canadian Highway Bridge Design Code and Commentary on CSA S6:19.

Canadian Standards Association (CSA), 2014. CSA A23.1-09 Concrete Materials and Methods of Construction (R2014).

Ministry of Northern Development of Mines. Bedrock Geology of Ontario – Southern Sheet, Ontario Geological Survey – Map 2544.

Ministry of Transportation, MTO Gravity Pipe Design Guidelines, MTO Drainage and Hydrology Design and Contract Standards Office, May 2014.

Occupational Health and Safety Act and Regulation for Construction Projects (as amended).

ASTM International

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering and Temporary Flow Passage Systems
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 578	Unshrinkable Fill
OPSS.PROV 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS Standard Special Provisions

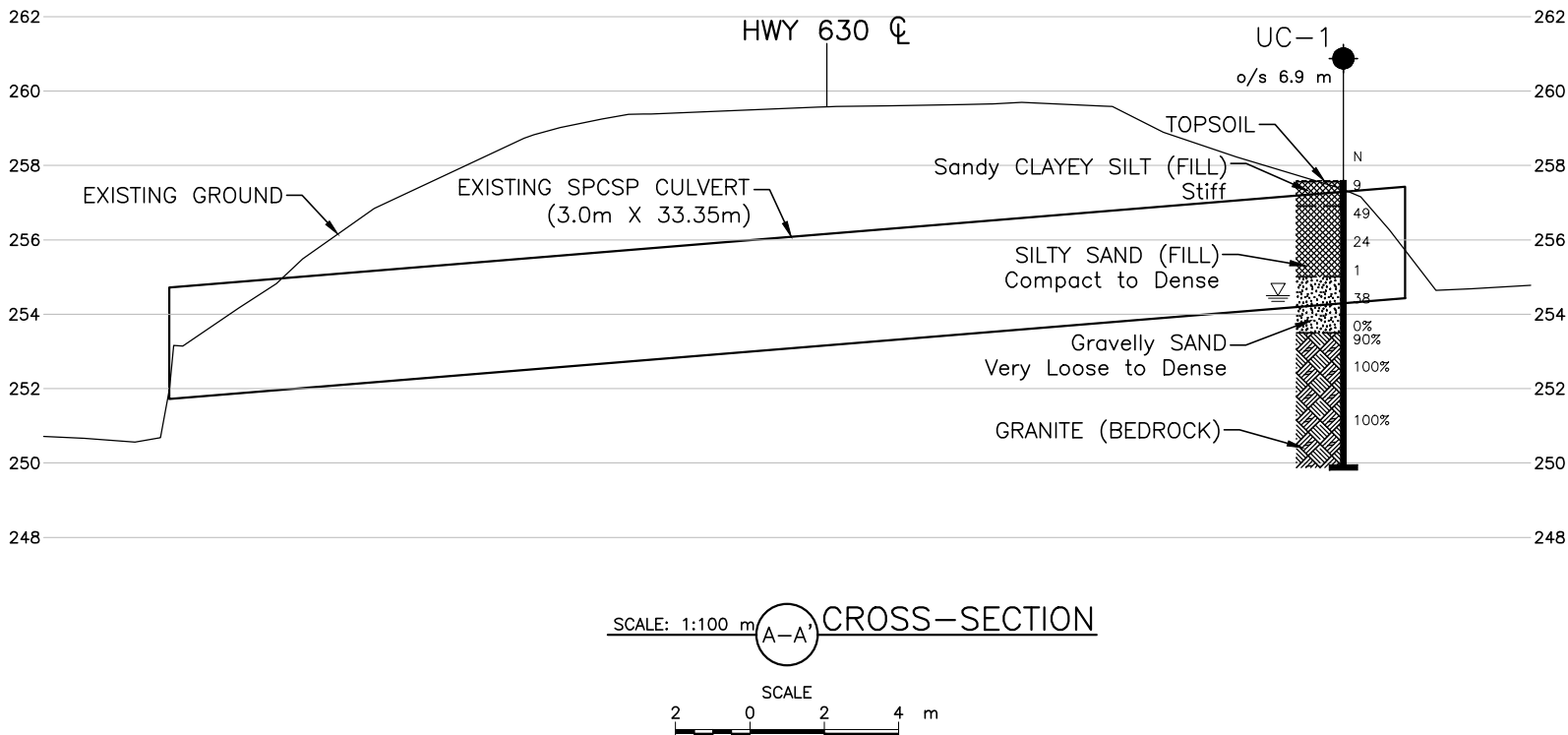
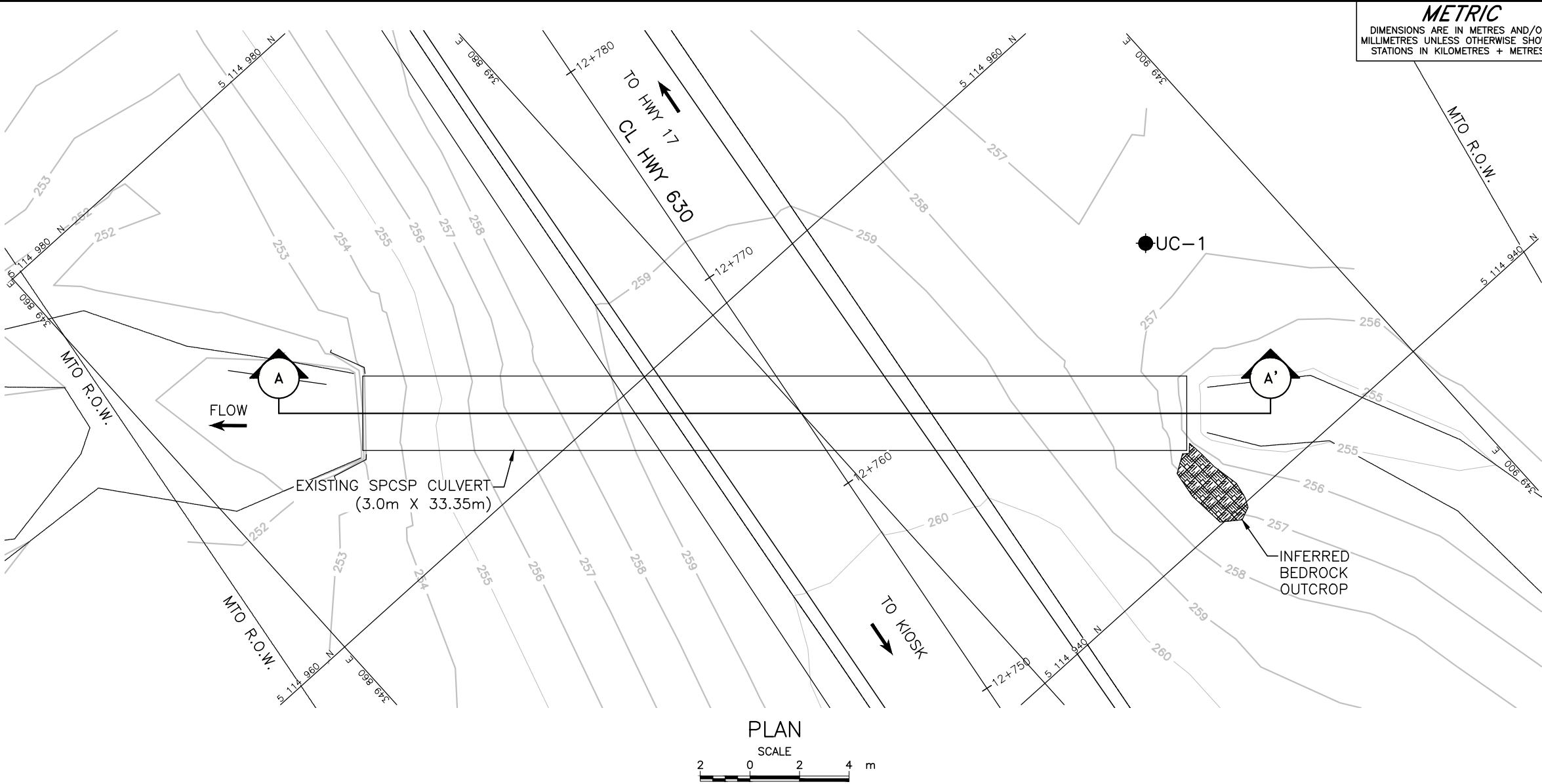
SSP 109S61	Amendment to OPSS 902
SSP 517F01	Amendment to OPSS 517

Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
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Ontario Water Resource Act

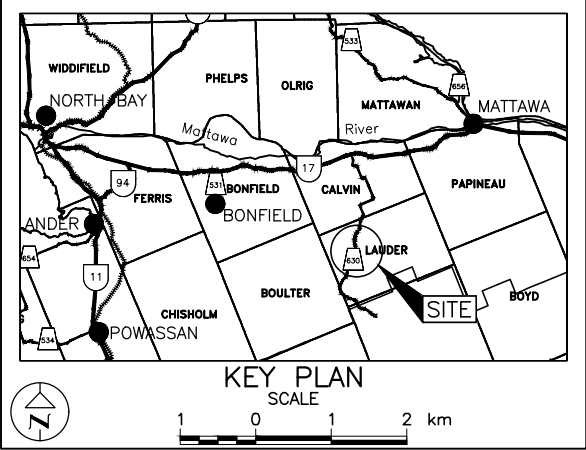
Regulation 903	Wells (as amended)
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METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5208-21-00

HIGHWAY 630
AMABLE DU FOND TRIBUTARY CULVERT
SITE NO. 43X-0301/CO
LOCATION AND SOIL STRATA



- LEGEND**
- Borehole - Current Investigation
 - Standard Penetration Test Value
 - Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
 - Rock Quality Designation (RQD)
 - WL upon completion of drilling

* BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 10)			
No.	ELEVATION	NORTHING	EASTING
UC-1	257.6	5114950.5	349894.9



NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plan and Topography provided in digital format by DM Wills, drawing file no. 23-25574 Hwy 630 Kiosk.dwg, Received August 17, 2023.

NO.	DATE	BY	REVISION
Geocres No. 31L04-001			
HWY. 630	PROJECT NO. CA0008394.9800		DIST. .
SUBM'D.	CHKD. TB	DATE: 3/25/2024	SITE: 43X-0301/CO
DRAWN: TR	CHKD. MT	APPD. KJB	DWG. 1



Photograph 1: Highway 630 – Amable du Fond Tributary Culvert Inlet – Facing West (November 2023)



Photograph 2: Highway 630 – Amable du Fond Tributary Culvert Inlet – Facing West (May 2023)



Photograph 3: Highway 630 – Amable du Fond Tributary Culvert East Embankment Slope – Facing West (May 2023)



Photograph 4: Highway 630 – Amable du Fond Tributary East Embankment Slope – Facing North (May 2023)

APPENDIX A

Record of Borehole

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
		2.00 to 4.75	(10) to (4)
SAND	Coarse	0.425 to 2.00	(40) to (10)
	Medium	0.075 to 0.425	(200) to (40)
	Fine		
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (<i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (<i>i.e.</i> , some sand)
≤ 10	trace (<i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q_t*), porewater pressure (*u*) and sleeve friction (*f_s*) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, w _L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

1. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

2. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ . Unit weight symbol is γ .
where $\gamma = \rho \cdot g$ (i.e., mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index = $(w_L - w_P)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_P) / I_P$
I_C	consistency index = $(w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{a(e)}$	secondary compression index
C_a	rate of secondary compression
$C_{a(e)}$	modified secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING CLASSIFICATION

Fresh (W1): no visible sign of rock material weathering.

Slightly Weathered (W2): discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

Moderately Weathered (W3): less than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Highly Weathered (W4): more than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Completely Weathered (W5): 100% of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

Residual Soil (W6): all rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

AXJ Axial Joint	KV Karstic Void
BD Bedding	K Slickensided
BC Broken Core	LC Lost Core
CC Continuous Core	MB Mechanical Break
CL Closed	PL Planar
CO Contact	PO Polished
CU Curved	RO Rough
CT Coated	SA Slightly Altered
FLT Fault	SH Shear
FOL Foliation	SM Smooth
FR Fracture	SR Slightly Rough
GO Gouge	SY Stylolite
IN Infilled	UN Undulating
IR Irregular	VN Vein
JN Joint	VR Very Rough

ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250



PROJECT		CA0008394.9800		RECORD OF BOREHOLE No. UC-1				1 OF 1		METRIC	
G.W.P.		5208-21-00		LOCATION		N 5114950.5; E 349894.9 NAD83 MTM ZONE 10 (LAT. 46.172836; LONG. -78.915978)				ORIGINATED BY TB	
DIST		HWY 630		BOREHOLE TYPE		108 mm I.D. Hollow Stem Augers, NW Casing and NQ Coring				COMPILED BY TR	
DATUM		GEODETIC		DATE		November 9, 2023				CHECKED BY MT	

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			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	20	40	60		
257.6	GROUND SURFACE																
0.8	TOPSOIL (40 mm) (FILL)		1	SS	9											2 33 42 23	
256.9	Sandy CLAYEY SILT (CL), trace gravel (FILL)		2	SS	49											7 79 (14)	
0.7	Stiff Brown Moist		3	SS	24												
	SILTY SAND (SM), trace gravel (FILL)		4A	SS	1												
	Compact to dense Brown Moist to wet		4B														
255.0	- Wood pieces encountered at 2.6 m depth.		5	SS	38											22 71 (7)	
2.6	Gravelly SAND (SP-SM), trace silt Very loose to dense Grey Wet		1	RC	REC 78%											RQD = 0%	
253.5	- 75 mm diameter cobble and gravel recovered in rock core run No. 1.		2	RC	REC 100%											RQD = 90%	
4.1	GRANITE (BEDROCK)		3	RC	REC 100%											RQD = 100%	
	For coring details refer to Record of Drillhole UC-1.		4	RC	REC 100%											RQD = 100%	
249.9	END OF BOREHOLE																
7.7	NOTE: 1. Water level measured at a depth of 3.1 m below ground surface (Elev. 254.5 m) in augers prior to coring operations.																

SUD-MTO 001 R:\VANCULVER\CAD-GIS\CLIENT\MINISTRY_OF_TRANSPORTATION_ONTARIO-MTO\HWY630\12_GINT\CA0008394.9800\GPJ_GAL-MISS.GDT 1/2/24 TR

INCLINATION: -90° AZIMUTH: --

DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

1 : 60

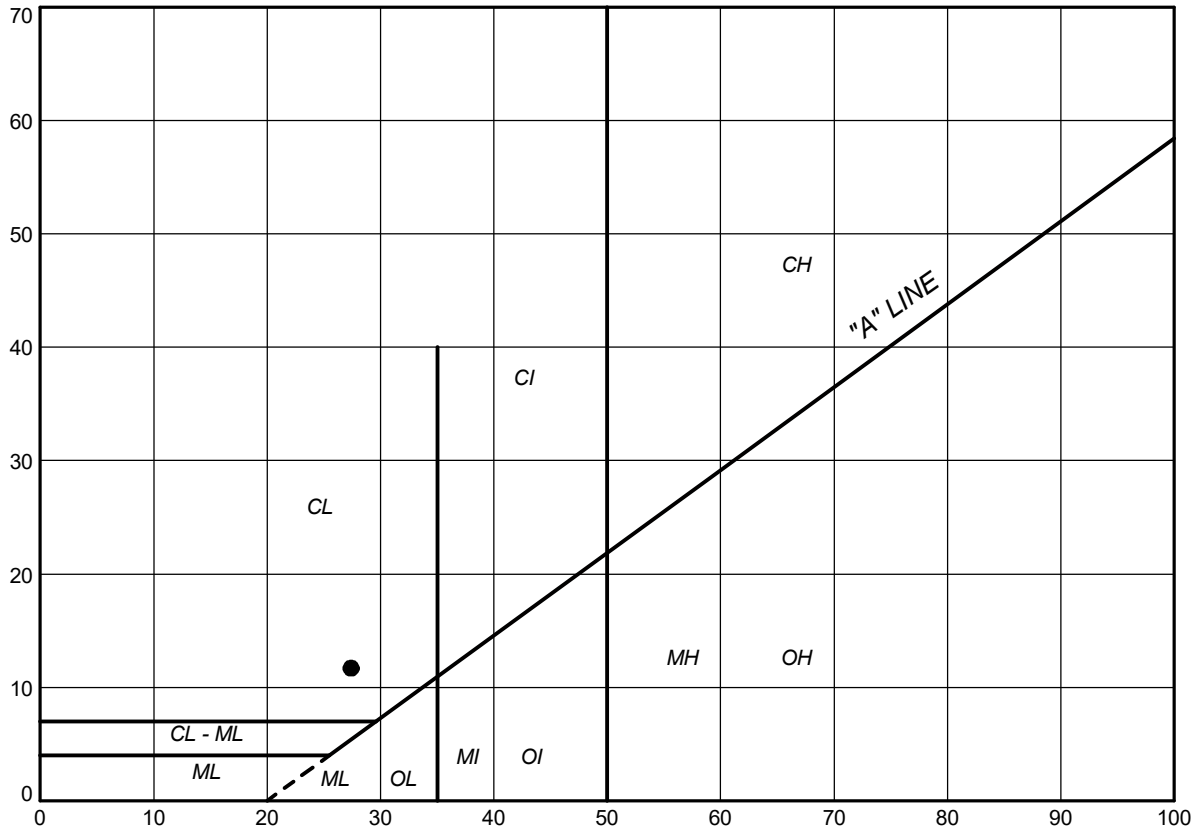


CHECKED: MT

APPENDIX B

Geotechnical Laboratory Test Results

PLASTICITY INDEX (Percent)




LIQUID LIMIT (Percent)

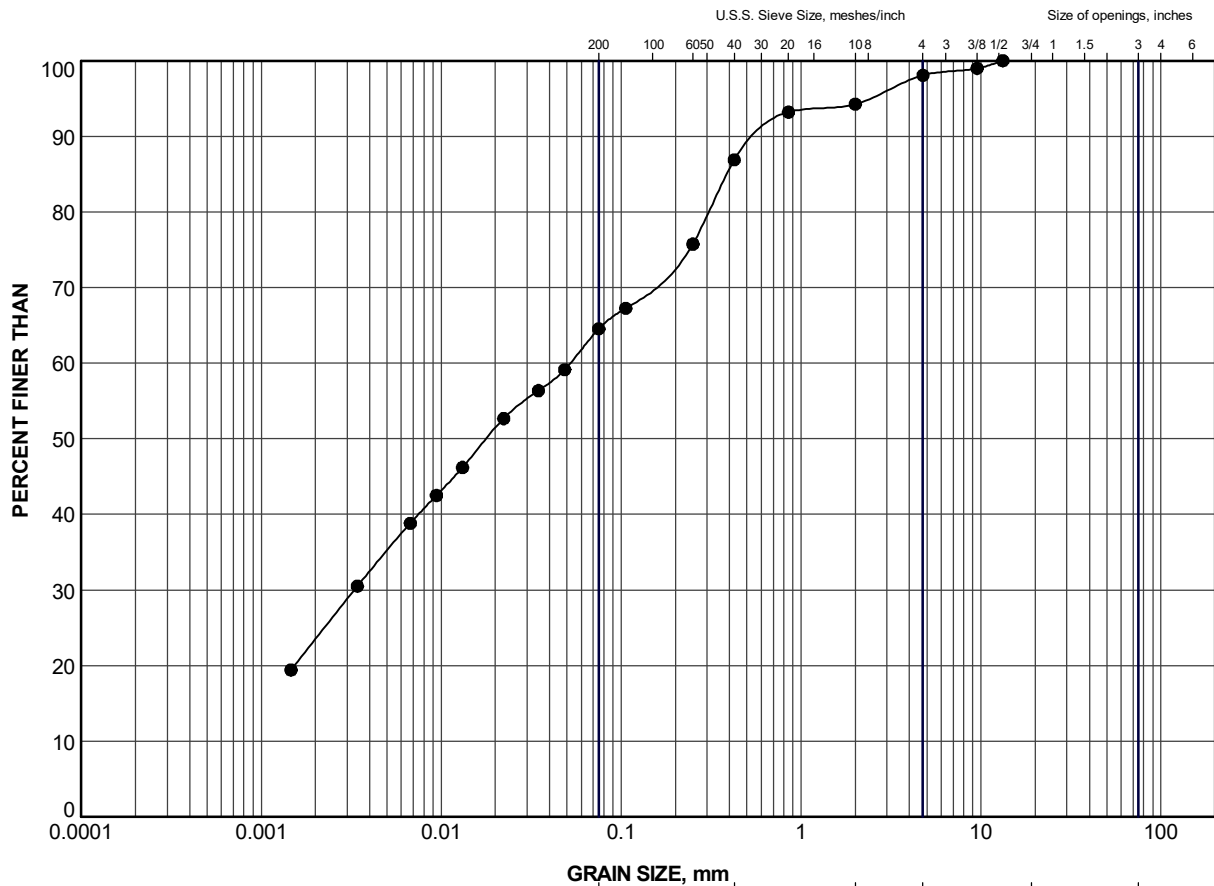
SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	UC-1	1	27.4	15.7	11.7


PROJECT						
HIGHWAY 630 AMABLE DU FOND TRIBUTARY CULVERT						
TITLE						
PLASTICITY CHART SANDY CLAYEY SILT (CL) (FILL)						
 SUDBURY, ONTARIO		PROJECT No. CA0008394.9800		FILE No. CA0008394.9800.GPJ		
		DRAWN	TR	Jan 2024	SCALE	N/A
		CHECK	MT	Jan 2024	REV.	
		APPR	KJB	Jan 2024		
FIGURE B-1						

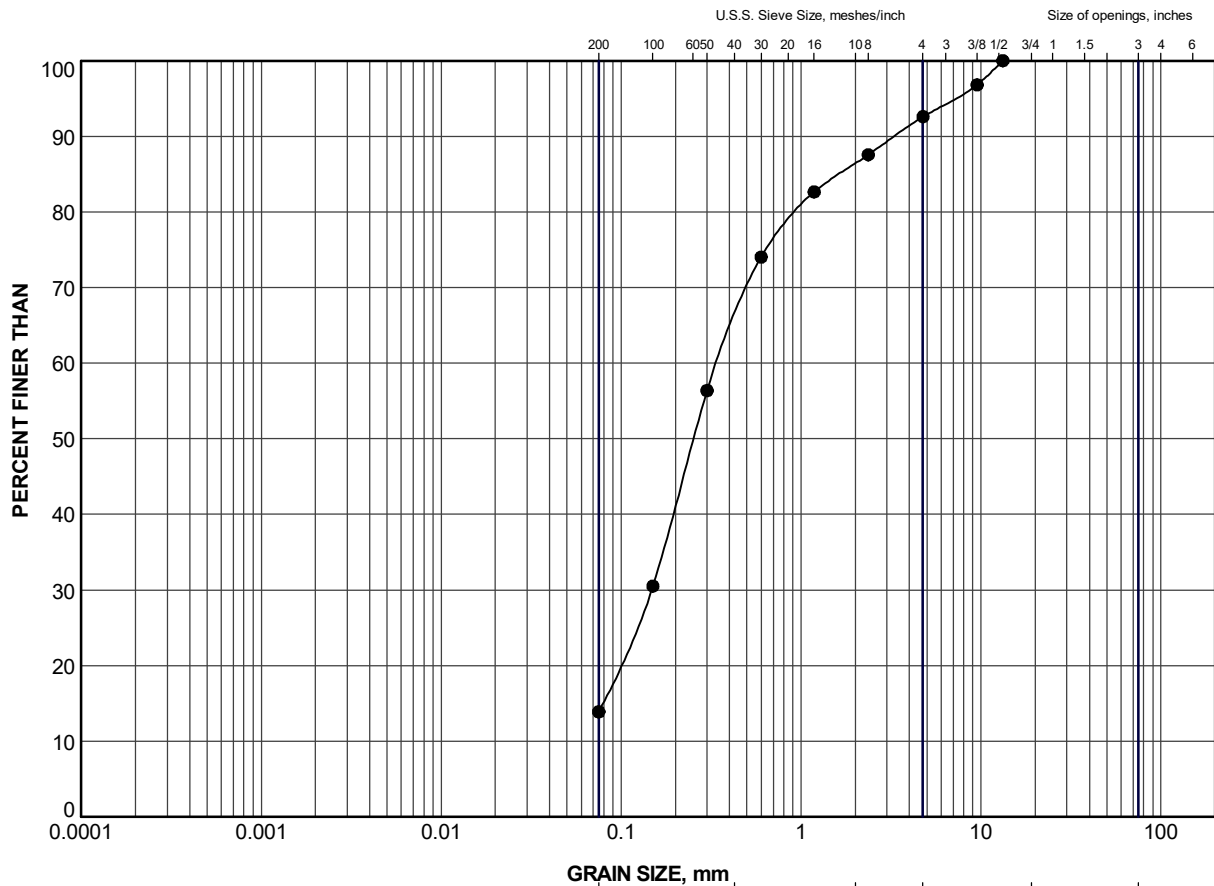


CLAY AND SILT	GRAIN SIZE, mm					Cobble Size
	fine		medium	coarse		
	SAND SIZE		GRAVEL SIZE			

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	UC-1	1	257.3

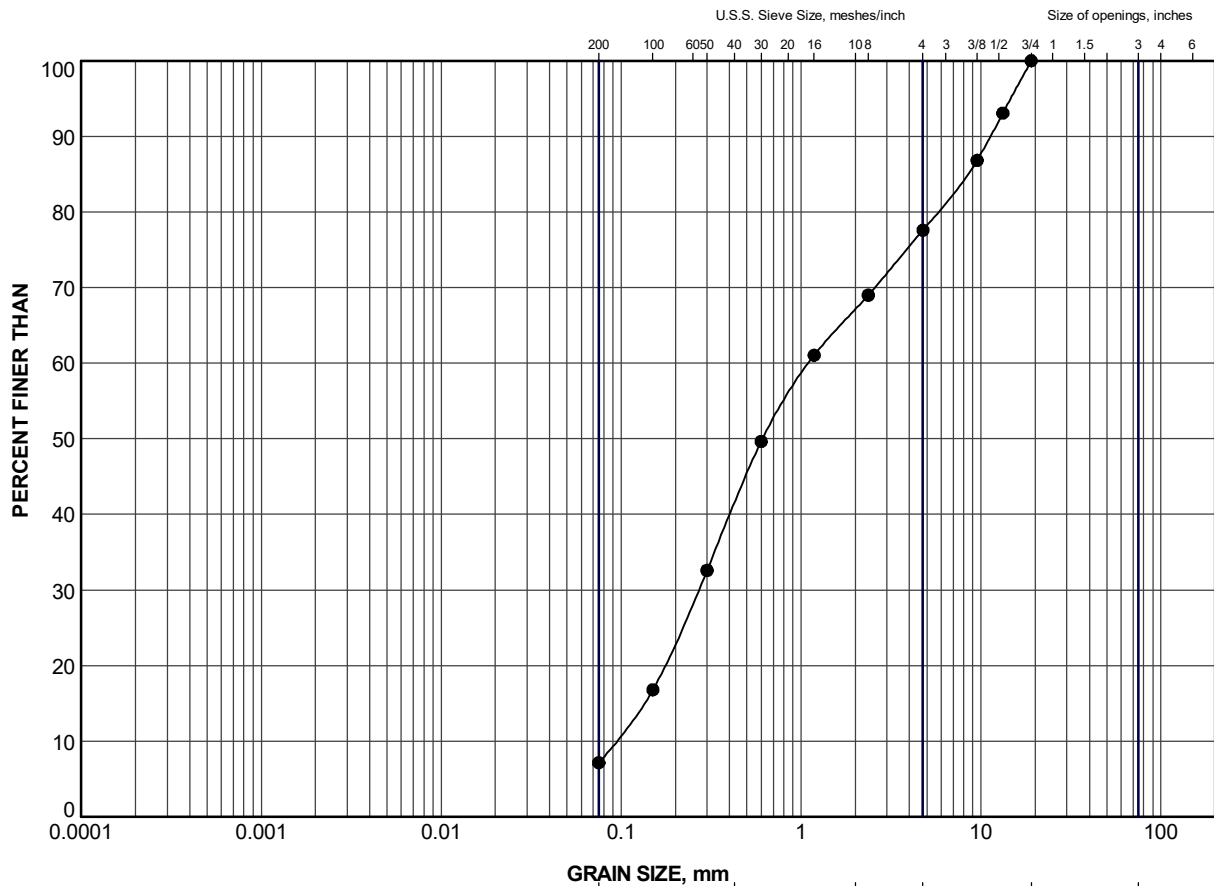
PROJECT		HIGHWAY 630 AMABLE DU FOND TRIBUTARY CULVERT			
TITLE		GRAIN SIZE DISTRIBUTION SANDY CLAYEY SILT (CL) (FILL)			
 SUDBURY, ONTARIO		PROJECT No. CA0008394.9800		FILE No. CA0008394.9800.GPJ	
		DRAWN	TR	Jan 2024	SCALE N/A
		CHECK	MT	Jan 2024	REV.
		APPR	KJB	Jan 2024	
		FIGURE B-2			



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	UC-1	2	256.5


PROJECT				
HIGHWAY 630 AMABLE DU FOND TRIBUTARY CULVERT				
TITLE				
GRAIN SIZE DISTRIBUTION SILTY SAND (SM) FILL				
 SUDBURY, ONTARIO		PROJECT No. CA0008394.9800		
		FILE No. CA0008394.9800.GPJ		
		DRAWN	TR	Jan 2024
		CHECK	MT	Jan 2024
		APPR	KJB	Jan 2024
		FIGURE B-3		



CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	UC-1	5	254.4

PROJECT					HIGHWAY 630 AMABLE DU FOND TRIBUTARY CULVERT					
TITLE					GRAIN SIZE DISTRIBUTION GRAVELLY SAND (SP-SM)					
 SUDBURY, ONTARIO					PROJECT No. CA0008394.9800			FILE No. CA0008394.9800.GPJ		
					DRAWN	TR	Jan 2024	SCALE	N/A	REV.
					CHECK	MT	Jan 2024	FIGURE B-4		
					APPR	KJB	Jan 2024			



Borehole UC-1: Bedrock cored between depths of about 4.1 m to 7.7 m

PROJECT		HIGHWAY 630	
		AMABLE DU FOND TRIBUTARY CULVERT (SITE NO. 43X-0301/C0)	
		LAUDER TWP., DISTRICT OF NIPISSING	
TITLE		BEDROCK CORE PHOTOGRAPH	
		DRILLHOLE UC-1	
 SUDBURY, ONTARIO	PROJECT No. CA0008394.9800		FILE No. ----
	DESIGN	TB	SCALE NTS
	CADD	--	FIGURE B-5
	CHECK	MT	
	REVIEW	KJB	

APPENDIX C

Analytical Laboratory Test Results



Your Project #: CA0008394.9800/1000FOUNDATIONS
Site Location: HWY 630-UNKNOWN CREEK CULVERT
Your C.O.C. #: n/a

Attention: Tibor Berecz

WSP Canada Inc.
33 Mackenzie Street
Suite 100
Sudbury, ON
Canada P3C 4Y1

Report Date: 2023/12/04
Report #: R7940886
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C3AG662

Received: 2023/11/22, 09:07

Sample Matrix: Soil
Samples Received: 1

Analyses	Quantity	Date	Date	Laboratory Method	Analytical Method
		Extracted	Analyzed		
Chloride (20:1 extract)	1	2023/11/27	2023/11/28	CAM SOP-00463	MOE E3013 m
Conductivity	1	2023/11/27	2023/11/27	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	1	N/A	2023/12/04	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	1	N/A	2023/12/04	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	1	2023/11/28	2023/11/28	CAM SOP-00413	EPA 9045 D m
Redox Potential (3)	1	2023/11/27	2023/11/28	CAM SOP-00421	SM 24 2580 B
Resistivity of Soil	1	2023/11/23	2023/11/27	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	1	2023/11/27	2023/11/28	CAM SOP-00464	MOE E3013 m

Remarks:

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCCFP, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Bureau Veritas' profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Bureau Veritas in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Bureau Veritas liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Bureau Veritas has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Bureau Veritas, unless otherwise agreed in writing. Bureau Veritas is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.

This Certificate shall not be reproduced except in full, without the written approval of the laboratory.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

(1) This test was performed by Bureau Veritas Calgary (19th), 4000 19th Street NE, Calgary, AB, T2E 6P8

(2) Offsite analysis requires that subcontracted moisture be reported.



Your Project #: CA0008394.9800/1000FOUNDATIONS
Site Location: HWY 630-UNKNOWN CREEK CULVERT
Your C.O.C. #: n/a

Attention: Tibor Berecz

WSP Canada Inc.
33 Mackenzie Street
Suite 100
Sudbury, ON
Canada P3C 4Y1

Report Date: 2023/12/04
Report #: R7940886
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C3AG662

Received: 2023/11/22, 09:07

(3) Oxidation-Reduction Potential (ORP) values are determined using a Ag/AgCl reference electrode. The test is therefore, not SCC accredited for this matrix.

Encryption Key

Please direct all questions regarding this Certificate of Analysis to:

Julie Clement, Technical Account Manager

Email: Julie.CLEMENT@bureauveritas.com

Phone# (613)868-6079

=====

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RESULTS OF ANALYSES OF SOIL

Bureau Veritas ID		XRF476		
Sampling Date		2023/11/09 13:55		
COC Number		n/a		
	UNITS	UC-1,SA#4 (7.5' 9.5')	RDL	QC Batch
Calculated Parameters				
Resistivity	ohm-cm	19000		9068220
CONVENTIONALS				
Redox Potential	mV	440	N/A	9073665
Inorganics				
Soluble (20:1) Chloride (Cl-)	ug/g	<20	20	9074044
Conductivity	umho/cm	54	2	9073607
Available (CaCl2) pH	pH	5.75		9076699
Soluble (20:1) Sulphate (SO4)	ug/g	30	20	9074144
Sulphide	mg/kg	0.6 (1)	0.5	9090129
Physical Testing				
Moisture-Subcontracted	%	22	0.30	9089916
RDL = Reportable Detection Limit QC Batch = Quality Control Batch N/A = Not Applicable (1) Extracted past method specified hold time Sample contained greater than 10% headspace at time of extraction.				



BUREAU
VERITAS

Bureau Veritas Job #: C3AG662

Report Date: 2023/12/04

WSP Canada Inc.

Client Project #: CA0008394.9800/1000FOUNDATIONS

Site Location: HWY 630-UNKNOWN CREEK CULVERT

Sampler Initials: TB

TEST SUMMARY

Bureau Veritas ID: XRF476
Sample ID: UC-1,SA#4 (7.5' 9.5')
Matrix: Soil

Collected: 2023/11/09
Shipped:
Received: 2023/11/22

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	SKAL/EC	9074044	2023/11/27	2023/11/28	Alina Dobreanu
Conductivity	AT	9073607	2023/11/27	2023/11/27	Kien Tran
Moisture (Subcontracted)	BAL	9089916	N/A	2023/12/04	Simranjeet Batth
Sulphide in Soil	SPEC	9090129	N/A	2023/12/04	Bailey Morrison
pH CaCl2 EXTRACT	AT	9076699	2023/11/28	2023/11/28	Taslima Aktar
Redox Potential	COND	9073665	2023/11/27	2023/11/28	Kien Tran
Resistivity of Soil		9068220	2023/11/27	2023/11/27	Automated Statchk
Sulphate (20:1 Extract)	SKAL/EC	9074144	2023/11/27	2023/11/28	Alina Dobreanu



GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	12.3°C
-----------	--------

Results relate only to the items tested.



BUREAU
VERITAS

Bureau Veritas Job #: C3AG662

Report Date: 2023/12/04

QUALITY ASSURANCE REPORT

WSP Canada Inc.

Client Project #: CA0008394.9800/1000FOUNDATIONS

Site Location: HWY 630-UNKNOWN CREEK CULVERT

Sampler Initials: TB

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
9073607	Conductivity	2023/11/27			102	90 - 110	<2	umho/cm	3.5	10
9073665	Redox Potential	2023/11/28			100	95 - 105			7.7	35
9074044	Soluble (20:1) Chloride (Cl-)	2023/11/28	NC	70 - 130	102	70 - 130	<20	ug/g	3.8	35
9074144	Soluble (20:1) Sulphate (SO4)	2023/11/28	NC	70 - 130	104	70 - 130	<20	ug/g	3.4	35
9076699	Available (CaCl2) pH	2023/11/28			100	97 - 103			2.0	N/A
9089916	Moisture-Subcontracted	2023/12/04					<0.30	%		
9090129	Sulphide	2023/12/04	84	75 - 125	93	75 - 125	<0.5	mg/kg	NC	30

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).



BUREAU
VERITAS

Bureau Veritas Job #: C3AG662

Report Date: 2023/12/04

WSP Canada Inc.

Client Project #: CA0008394.9800/1000FOUNDATIONS

Site Location: HWY 630-UNKNOWN CREEK CULVERT

Sampler Initials: TB

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by:

Anastassia Hamanov, Scientific Specialist

Veronica Falk, B.Sc., P.Chem., QP, Scientific Specialist, Organics

Sandy Yuan, M.Sc., QP, Scientific Specialist

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CHAIN OF CUSTODY RECORD

ENV COC - 00014v3

Page _____ of _____

Invoice Information		Invoice to (requires report) <input type="checkbox"/>		Report Information (if differs from invoice)		Project Information	
Company: WSP Canada Inc.		Company:		Quotation #:			
Contact Name: Tibor Berecz		Contact Name:		P.O. #/ AFE#:			
Street Address: 33 MacKenzie St. Suite 100		Street Address:		Project #:		CA0008394.9800/1000Foundations	
City: Sudbury	Prov: ON	Postal Code: P3C 4Y1	City:	Prov:	Postal Code:	Site #:	
Phone: 647 616 8397		Phone:		Site Location:		Hwy 630 - Unknown Creek culvert	
Email: tibor.berecz@wsp.com		Email:		Site Location Province:		ON	
Copies:		Copies:		Sampled By:		Tibor Berecz	

22-Nov-23 09:07

Julie Clement

C3AG662

SBS ENV-1454

Regulatory Criteria REG 153 Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/Fine <input type="checkbox"/> CME <input type="checkbox"/> Reg 406, Table: <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Course <input type="checkbox"/> Reg 558* <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/other <input type="checkbox"/> For RSC <input type="checkbox"/> *min 3 day TAT <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> Table <input type="checkbox"/> MISA <input type="checkbox"/> Municipality <input type="checkbox"/> TWQO <input type="checkbox"/> Other: <input type="checkbox"/>										Regular Turnaround Time (TAT) <input checked="" type="checkbox"/> 5 to 7 Day <input type="checkbox"/> 10 Day Rush Turnaround Time (TAT) Surcharges apply <input type="checkbox"/> Same Day <input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Day <input type="checkbox"/> 3 Day <input type="checkbox"/> 4 Day									
Include Criteria on Certificate of Analysis (check if yes): <input type="checkbox"/> SAMPLES MUST BE KEPT COOL (<10°C) FROM TIME OF SAMPLING UNTIL DELIVERY TO BUREAU VERITAS										# OF CONTAINERS SUBMITTED HOLD - DO NOT ANALYZE									

Sample Identification	Date Sampled			Time (24hr)		Matrix	LAB FILTRATION REQUIRED										# OF CONTAINERS SUBMITTED	HOLD - DO NOT ANALYZE	Comments										
	YY	MM	DD	HH	MM		1	2	3	4	5	6	7	8	9	10				11	12	13	14	15	16	17	18	19	20
1 UC-1, Sa# 4 (7.5'-9.5')	23	11	09	13	55	Soil																						1	
2 UC-1, Sa# 4 (7.5'-9.5') dup	23	11	09	13	55	Soil																						Sa#4 Duplicate if required	
3																													
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*UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BUREAU VERITAS STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS AND CONDITIONS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVNA.COM/TERMS-AND-CONDITIONS OR BY CALLING THE LABORATORY LISTED ABOVE TO OBTAIN A COPY

LAB USE ONLY		Yes	No	°C	12	12	13	LAB USE ONLY		Yes	No	°C	1	2	3	LAB USE ONLY		Yes	No	°C	1	2	3	Temperature reading by:
Seal present								Seal present			Seal present													
Seal intact								Seal intact			Seal intact													
Cooling media present				Cooling media present			Cooling media present																	

Relinquished by: (Signature/ Print)		Date		Time		Received by: (Signature/ Print)		Date		Time		Special Instructions	
1 Tibor Berecz / Tibor B.		YY	MM	DD	HH	MM	1 [Signature]		YY	MM	DD	HH	MM
		23	11	20			2 [Signature]		22	11	22	09	07

APPENDIX D

Special Provisions

DEWATERING SYSTEM - Item No.
TEMPORARY FLOW PASSAGE SYSTEM - Item No.

Special Provision No. 517F01

February 2024

Amendment to OPSS 517, November 2023

Return Period Flow and Preconstruction Survey Distance

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

Clause 517.04.01.01 of OPSS 517 is amended by deleting the second last paragraph in its entirety and replacing it with the following:

The temporary flow passage system shall allow the work to be conducted as specified in the Contract Documents. Design flow shall include groundwater discharge and flow resulting from a minimum 2 year return period design storm, except for the work specified in Table 1. For the work specified in Table 1, design flow shall include groundwater discharge and flow resulting from a design storm of the minimum return period specified in Table 1. A longer return period shall be used when determined appropriate for the work.

The flow estimates as specified in Table 1 do not include flow volumes from groundwater discharge.

The Owner specifically excludes flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

TABLE 1
Site Location and Reference Information

TEMPORARY FLOW PASSAGE SYSTEMS							
Source of Return Period Flow Estimates:							
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m ³ /s) (Note 1)				Design Engineer Requirements (Note 2)	Fish Passage Required (Note 3)
		2 Year	5 Year	10 Year	25 Year		
DEWATERING SYSTEMS							
Site Name / Station Reference	Preconstruction Survey Distance (m) (Note 4)	Minimum Lowered Groundwater Depth Below Base of Excavation or Work Area (m) (Note 5)			Design Engineer Requirements (Note 2)		
Station 12+763	N/A	1.0 or to competent bedrock			No		
<p>Notes:</p> <ol style="list-style-type: none"> 1. a) The Design Engineer is to satisfy themselves to the accuracy and applicability of the provided flows. <li style="padding-left: 20px;">b) The intensity-duration-frequency (IDF) information can be accessed through MTO's IDF Curve Lookup web-based application tool at https://idfcurlves.mto.gov.on.ca/ <li style="padding-left: 20px;">c) The design, operation and maintenance of the temporary flow passage system is the sole responsibility of the Contractor. 2. "Yes" means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. "No" means a minimum experience level is not required for the design Engineer and design-checking Engineer. 3. "Yes" means that the design Engineer must design the temporary flow passage system to meet the fish passage requirements. "No" means fish passage is not required. 4. "N/A" means a preconstruction survey is not required. 5. Groundwater shall be lowered within the excavation or work area to below this minimum depth. 							

[* Designer Fill-Ins for Table 1, See Notes to Designer]

NOTES TO DESIGNER:

Designer Fill-Ins for Table 1:

1. Fill-in the source of the return period flow estimates.
2. Fill-in the site name, work, and station reference as appropriate for the dewatering system and/or temporary flow passage system item locations. Add additional rows as necessary.
3. For temporary flow passage system item locations, fill-in the minimum return period flow for each site based on MTO Drainage Design Standard TW-1. The return period flow shall not be less than 2 years.
4. For temporary flow passage system item locations, fill-in the design flow rate estimates for the various return periods.
5. Fill-in "Yes" under Design Engineer Requirements when recommended by the Foundation Engineer. Fill-in "No" otherwise.
6. For temporary flow passage system item locations, fill-in "Yes" under Fish Passage Required, when maintaining fish passage is a condition of a permit/ authorization or as recommended by the MTO Fisheries Assessment Specialist, in consultation with the MTO Environmental Planner. Fill-in "No" otherwise.
7. Fill-in the required distance under Preconstruction Survey Distance, when recommended by the Foundation Engineer. Fill-in "N/A" if not recommended.
8. Fill-in the Minimum Lowered Groundwater Depth Below Base of Excavation or Work Area provided by the Foundation Engineer.
9. When applicable, add a point d) to Note 1 of the table notes to indicate when Return Period Flow Estimates do not include base flows, for example:
 - d) The Return Period Flow Estimates do not include base flows.
 - d) The Return Period Flow Estimates at [enter Site Name/Description] do not include base flows.

WARRANT: Always with these tender items.

DOWELS INTO ROCK - Item No. 31

Special Provision

CONSTRUCTION SPECIFICATION FOR THE SUPPLY, INSTALLATION AND TESTING OF DOWELS INTO ROCK

1.0 SCOPE

The work for the above noted tender item shall be in accordance with OPSS 904, including all Special Provisions, except as extended herein. This document specifies additional requirements for the supply, installation, and testing of Dowels into Rock.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications, or publications:

ASTM International D1143M Standard Test Methods for Deep Foundations Under Static Axial Compressive Load

ASTM International D3689M Standard Test Methods for Deep Foundation Element Under Static Axial Tensile Load

3.0 DEFINITIONS

For the purpose of this Special Provision, the following definitions apply:

Dowels into Rock means reinforcing steel dowels installed within rock bores and secured with non-shrink grout within the annular space surrounding the dowel.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Working Drawings

The Contractor shall submit Working Drawings two weeks prior to construction to the Contract Administrator as follows:

- a) All Working Drawings shall be sealed and signed by the Design Engineer and Design Check Engineer.

- b) A plan illustrating the layout of the dowels.
- c) Detail drawing of the dowel into bedrock (typical).
- d) The method for constructing of the holes, maintaining the holes, and placing reinforcing steel dowels, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
- e) The procedures to verify hole length.
- f) Test procedures for Dowels into Rock.
- g) Drawings and design calculations for a suitable reaction system for the applied test loads.
- h) Drawings and details for reference system arrangement.
- i) Calibration curves for all gauges.

5.0 MATERIALS

5.01 Non-Shrink Grout

The non-shrink grout shall be an approved product from the MTO's Pre-Qualified Products List.

5.02 Anti-Washout Agent

The anti-washout agent shall be used with the non-shrink grout for the Dowels into Rock.

5.03 Manufacturer Information

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

- a) Data sheets for the non-shrink grout and anti-washout agent,
- b) Technical information that proves that the non-shrink grout and anti-washout agent are compatible, and
- c) Installation procedures.

5.04 Steel Dowels

Steel dowels shall conform to the requirements of OPSS 905 and OPSS 1440.

Dowels shall be new, clean, and free of deleterious material.

6.0 EQUIPMENT

All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

The equipment shall not cause damage to the reinforcing steel dowels.

7.0 CONSTRUCTION

7.01 General

The Contractor shall supply equipment, materials, and skilled personnel to install production Dowels into Rock. The Contractor shall conduct the specified acceptance tests.

The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

7.02 Subsurface Conditions

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

The Owner warrants the data in the Foundation Investigation Report, except that interpretations of the data and opinions expressed in the Foundation Investigation Report are not warranted.

7.03 Construction of Holes

The sides and end of the hole shall not be disturbed. The Contractor shall construct the holes, maintain the holes, and place reinforcing steel dowels, grout, and other necessary materials in the holes.

The hole diameter should be at least 2 times the nominal dowel diameter. The hole diameters and hole length for this project are as specified on the Contract Drawings.

At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, installation times, and drilled hole lengths. The Contractor shall submit these records to the Contract Administrator upon completion of the work.

7.04 Installation of Reinforcing Steel Dowels

Reinforcing steel dowels shall be installed in strict accordance with the Contract Drawings.

Centering devices shall be provided to ensure that the reinforcing steel dowels are located centrally in the hole.

Dowels shall extend into sound bedrock at least to the embedment depth noted on the Contract Drawings.

The Contractor shall submit records of the installed length of each dowel to the Contract Administrator.

7.05 Grout and Anti-Washout Agent

The non-shrink grout shall entirely fill the annular space between the reinforcing steel dowels and the sides of the dowel hole. The grout shall be of the same strength as the footing concrete or at least 30 MPa at 28 days.

Anti-washout agent shall be used in accordance with the specifications of the manufacturer.

The Contractor shall submit test results verifying the 28-day strength of non-shrink grout to the Contract Administrator. Testing shall be completed for each lot of dowels installed, where a lot shall consist of dowels of the same dowel type installed on a given day, in a single stage. Where a given day's production is less than 50 dowels, the day's work may be combined with the next day's production to form a single lot.

7.06 Testing for Dowels

Upon completion of installation and testing of each group of dowels, the Contractor shall submit to the Contract Administrator a Request to Proceed.

The next operation after the completion of testing shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

8.0 QUALITY ASSURANCE

In each group, 10% of the dowels rounded up to the next whole number, but no fewer than two dowels, shall be tested.

8.01 General Inspection and Testing Requirements

The Contractor shall supply materials and equipment to conduct the tests for the dowels into rock. The equipment and materials shall be capable of stressing the dowels into rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements. The testing device shall not apply compression to the bedrock surrounding the dowels into rock within the area delimited by a circle concentric with the dowel hole and a diameter equal to 5.0 m.

The inspection of dowels shall be carried out by the Contract Administrator in advance of the installation of Dowels into Rock.

Testing for Dowels into Rock shall be conducted by the Contractor as scheduled by the Contract Administrator. The Contract Administrator will be present to inspect the testing of the Dowels into Rock. The Contractor shall notify the Contract Administrator of the testing schedule at least 10 working days prior to commencement of the testing program.

8.01.01 Testing Equipment

The Dowels into Rock will be carried out generally in accordance with the prevailing requirements of ASTM International D1143M and D3689M superseded where applicable by the procedures specified in the Contract Drawings.

Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel dowels or stressing system.

The Contractor shall construct suitable enclosures to provide complete protection for all equipment from variations in the weather conditions and disturbances.

8.01.02 Documentation of Testing for Dowels into Rock

At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel dowels, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit the above noted records to the Contract Administrator.

Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit details for current calibration and curves for all gauges to the Contract Administrator.

8.01.03 Testing Loading

Jacks used for reinforcing steel bars shall have a minimum stroke extension of 152.4 mm (6.0 in.).

Rock dowels shall be loaded and unloaded in 2 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4
% Design Load	50	75	25	50	75	100	25

The design load shall be taken as 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced point.

8.02 Acceptance Criteria

The following acceptance criteria apply:

- a) Tests for Dowels into Rock shall have a capacity of at least 108 kN. The dowels are acceptable if the total elastic movement is greater than 80 percent of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50 percent of the bond length.

9.0 MEASUREMENT FOR PAYMENT

For measurement purposes, a count shall be made of the number of dowels installed in accordance with the Contract Drawings.

10.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work.

Schedule Of Materials To Be Supplied By The Owner

The Ministry shall supply NO materials for this contract.

