Foundation Investigation and Design Report

Culvert Replacement, Knife Creek
Station 15+640, Township of Dahl, Algoma District
Highway 17

GWP 5119-06-00

Geocres No: 42C-041
Site No: 38C-154/C

Prepared for
MTO Northeastern Region
447 McKeown Avenue
North Bay, ON
P1B 9S9

Prepared By:
TBT Engineering Limited
1918 Yonge Street
Thunder Bay, Ontario, P7E 6T9

November 7, 2017 Ref. No. 16-138 Rev 2

Table of Contents

APPENDICIES

Part A - FOUNDATION INVESTIGATION REPORT

1 Introduction

TBT Engineering Limited (TBTE) has been retained by the Ministry of Transportation Northeastern Region (MTO) to provide a foundation investigation and design services for the proposed culvert replacement at station 15+640, Township of Dahl, district of Algoma. The site is located within the boundaries of Obatanga Provincial Park, approximately 37.6 km east of the junction of Highway 17 and Highway 631 (White River). The site coordinates are as follows:

- Latitude: 48.331565°
- Longitude: -85.027971°

This project has been assigned Geocres No. 42C-041 and structural site number 38C-154/C.

The foundation investigation was conducted to provide subsurface data to for stability analysis of finished grade and safe excavation slopes, provide commentary on conceptual cofferdam design and roadway protection measures, and for replacement recommendations including but not limited to lateral earth pressures, foundation types (deep and shallow) and associated ULS resistances and SLS reactions.

A total of eight boreholes were advanced for this investigation. Two boreholes were advanced through the embankment at the culvert location, three at the inlet and three at the outlet (two of the boreholes at each end of the culvert were at potential cofferdam locations). All borehole locations were determined through consultation with the MTO. This report (Part A) describes the subsurface conditions encountered during the investigation.

2 Site Description

The foundation investigation was conducted to investigate subsurface conditions at the culvert located at station 15+640 Township of Dahl, Algoma District. Knife creek flows through triple structural corrugated steel culverts beneath an embankment that crosses low lying terrain. As provided in the terms of reference, the three culverts are 2.7 m wide x 25 m long, with approximate 1.5m of cover. The maximum height of the embankment is approximately 2.0 m. The culvert inverts on the right are typically 416.1, and 416.3 on the left. Based on survey data provide the creek level was measured at elevation 417.6 m.

Photo 2.1 – Near Station 15+640, Facing South

Photo 2.2 – Near Station 15+640, Facing North

2.1 Surficial Geology

Available surficial geology mapping (OGS NOEGTS Map 5096 – Pukaskwa River) indicates the site is located along the boundary between organic and rock knob terrain. The organic terrain is over a sand outwash plain with mainly low local relief with wet drainage. TBTE's investigation encountered silt beneath organic material in the "greenfield" borehole locations. The rock knob terrain has areas of till ground moraine and peat organic terrain, with moderate dry relief. Bedrock outcrops were observed roughly 400 m east of the site, and organic terrain was observed west and north of the site.

3 Investigation Procedures

A geotechnical site investigation was undertaken between September 20 and October 3, 2016. A total of 8 boreholes were advanced during the field investigation. The borehole locations and depths were determined through conversations with the MTO and are illustrated on the Borehole Location Plan found in Appendix C.

The borehole locations were identified in the field by TBTE personnel and service clearances were completed prior to mobilizing the drill rig to site. The boreholes were advanced using a track mounted drill rig, equipped with hollow stem augers and a cat head used conduct Standard Penetration Testing (SPT). Soil samples were obtained at the boreholes from the auger flights and using a split spoon sampler as a part of the SPT. Rock cores were obtained using NQ diamond coring techniques.

All aspects of implementation of geotechnical test holes (were completed in accordance with the Ministry of Environment Regulation 903, as amended by Regulation 128/03. Boreholes on the road surface were capped with cold mix asphalt upon decommissioning.

Borehole locations were surveyed by TBTE and were referenced to elevations provided on MTO Plate No. 692-17/13-0 WP 267-90-00 Station 15+500 to 16+200 Surveyed September 1994 TWP of Dalh. The provided MTO Plate drawing is based on NAD 83 CSRS MTM Zone 13, and Canadian Geodetic Vertical Datum CGVD28.

4 Laboratory Testing

Samples which were obtained during the field investigation were subjected to routine laboratory testing. The routine testing included moisture content, liquid and plastic limit tests, and grain size analysis. The results of this testing are shown on the Borehole Logs (Appendix A and on the laboratory data reports Appendix B). In order to classify the bedrock with respect to strength, point load tests were carried out on select rock cores.

In addition to routine testing, a single sample (BH2, SS4) was selected for analytical laboratory testing. Analytical tests performed included conductivity, moisture content, pH, Redox Potential, resistivity, chloride, sulphide and sulphate testing. Test results are included with Appendix B, laboratory test data and summarized on Table 5.1.

5 Subsurface Conditions

Details of the subsurface conditions are provided on the test hole logs (Appendix A), and on the Soil Strata Drawings (Appendix C).

The subsurface soils at this site typically consist of organics or fill over silt overlying bedrock. Bedrock was sampled when encountered.

5.1 Asphalt

Asphalt was encountered at the surface of Boreholes 1 and 2. Boreholes 1 and 2 were advanced through the roadway embankment. The asphalt was 75 mm thick.

5.2 Organic Material

Organic material was encountered at the surface of Boreholes 6 and 8, and extended to a depth of 1.6 and 1.4 m, (elev. 416.5 m) respectively. The organic material is in a very loose condition as indicated by “N” values of 2 and 3 blows/0.3 m. The natural moisture content of this material ranges from 50 % to 183%.

5.3 Granular Fill

Granular embankment fill, comprised of gravels and sands with occasional cobbles, was encountered beneath the asphalt at Boreholes 1 and 2. The fill extended to depths of 2.2 and 2.3 m (elev. 417.4 and 417.5 m respectively). Two samples were selected for grain size distribution testing. The test results indicated a grain size distribution of 7 to 58% gravel, 38 to 70% sand, and 4 to 23% silt/clay sized particles. Numerous cobbles were noted in Borehole 1. The material is in a dense to very dense condition as indicated by “N” values ranging from 36 to 100+ blows/0.3 m.

5.4 Sand

A sand layer was encountered beneath the fill at Borehole 1 and beneath the organic material at Borehole 8. The sand had a thickness of 0.7 and 1.5 m, extending to elevations of 416.7 and 415 m at Boreholes 1 and 8, respectively. The material is in a very loose to compact condition as indicated by “N” values ranging from 3 to 19 blows/0.3 m. A single sample was selected for grain size distribution testing. The test results indicated a grain size distribution of 15 % gravel, 74 % sand, and 11 % silt/clay sized particles.

5.5 Silt

Silt was encountered beneath the embankment fill at Borehole 2, the organics at Borehole 6, the sand at Boreholes 1 and 8 and at the surface of Boreholes 3, 4, 5, and 7. The silt extended to auger refusal at depths ranging from 7.5 to 9.5 m (elev. 409.0 to 411.1 m) on the north side of the highway (at Boreholes 1, 3, 4 and 5). The thickness of the silt was unknown at Boreholes 2, 6, 7 and 8 as these boreholes terminated within the silt stratum at depths ranging from 10.6 to 12.7 m (elev. 406.7 to 407.5 m). Sixteen samples were selected for grain size distribution testing. The test results indicated a grain size distribution of 0 to 21% gravel, 0 to 34% sand, and 62 to 100% silt/clay sized particles. The material is in a very loose to very dense condition as indicated by “N” values ranging from 1 to 90 blows/0.3 m. Six samples were selected for Atterberg limit testing, and all samples were found to be non-plastic (silt).

A sample of this material from Borehole 2 was submitted for corrosivity and conductivity testing, detailed results are provided in Appendix B. The results are summarized as follows:

Table 5.1: Analytical Testing Results

Test	Unit	Result
Conductivity	mS/cm	0.362
Moisture	%	12.1
Acidity/Basicity	pH	7.78
Redox Potential	mV	123
Resistivity	ohm*cm	2760
Chloride	ppm	91.8
Sulphide (as S)	mg/kg	<0.2
Sulphate	ppm	42

5.6 Bedrock

Bedrock was confirmed below the silt at Boreholes 1, 3, 4 and 5 (elevations 410.2m, 409.9m, 411.1m, and 409.0m, respectively). Generally, the bedrock encountered was un-weathered, grey and white gneiss. Detailed bedrock core logs and photos are provided as Appendix D.

In order to classify the bedrock with respect to strength, 12-point load tests were completed on selected core samples. The test results are tabulated below:

Table 5-2: Estimated Uniaxial Compressive Strength of Bedrock

Borehole	Test depth from ground surface (m)	Test Elevation (m)	*Estimated Uniaxial Compressive Strength (MPa)
1	9.7	410.0	393
1	10.7	409.0	519
1	11.8	407.9	448
3	9.0	409.0	404
3	10.5	407.5	527
3	11.1	406.9	432

4	8.7	409.9	493
4	9.2	409.4	450
4	10.5	408.1	156
5	9.8	408.6	407
5	10.6	407.8	246
5	12.1	406.3	503

** Estimated based on published correlations with point load testing*

Based on the estimated uniaxial compressive strength of the intact rock, the bedrock is generally extremely strong (uniaxial compressive strengths greater than 250 MPa). Two occurrences of very strong (100 to 250 MPa) rock were at Borehole 4 at 10.5m, and Borehole 5 at 10.6m.

The rock quality designation (RQD) is an indirect measure of the number of fractures and the amount of jointing in the rock mass. The RQD is expressed as a percentage of the ratio of the summed core lengths (greater than 100 mm) to the total length cored. The RQD index is used to provide a classification for the rock quality according to the following limits.

Table 5-3: RQD / Rock Quality Designation

RQD (%)	Rock Quality
0 – 25	Very Poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

The RQD measured over the core lengths ranged from 93 to 100% indicating the rock quality is excellent.

6 Ground Water

Groundwater levels were measured upon completion of drilling operations and are summarized in the table below. Groundwater levels will vary from season to season and from the effects of heavy precipitation events.

Table 6.1: Groundwater Levels

Borehole	Groundwater Depth (m)	Groundwater Elevation (m)	Date Measured
4	1.1	417.5	Sept 28, 2016
5	1.6	416.8	Sept 29, 2016
6	1.0	417.1	Sept 30, 2016
7	1.0	417.1	Oct 2, 2016
8	1.2	416.7	Oct 3, 2016

7 Miscellaneous

Laboratory testing was completed at the TBT Engineering laboratory in Thunder Bay. The drill equipment for this investigation was operated by TBT Engineering Limited. The field operations were supervised by Walter Mainville. Laboratory testing was supervised by T. Fummerton C.E.T. This report was prepared by Craig Johnson, P.Eng and Steven Seller, P.Eng, and reviewed by W. Hurley, P.Eng (TBTE designated principal contact identified for MTO Foundation Engineering projects).

Part B - FOUNDATION DESIGN RECOMMENDATIONS

8 Introduction

TBT Engineering Limited (TBTE) has been retained by the Ministry of Transportation Northwestern Region (MTO) to provide foundation investigation and design services for the proposed culvert replacement at station 15+640, Township of Dahl, district of Algoma. The site is located within the boundaries of Obatanga Provincial Park, approximately 37.6 km east of the junction of Highway 17 and Highway 631 (White River). The site coordinates are:

- Latitude: 48.331565°
- Longitude: -85.027971°

The foundation investigation was conducted to provide subsurface data to for stability analysis of finished grade and safe excavation slopes, provide commentary on conceptual cofferdam design and roadway protection measures, and for replacement recommendations including but not limited to lateral earth pressures, foundation types (deep and shallow) and associated ULS resistances and SLS reactions.

A total of eight boreholes were advanced for this investigation. Two boreholes were advanced through the embankment within 20 m from the existing culverts, three near the inlet and three at the outlet (two of the boreholes at each end of the culvert were at potential cofferdam locations). All borehole locations were determined through consultation with the MTO. This report (Part A) describes the subsurface conditions encountered during the investigation. The foundation soils at this site consists of a sand and gravel embankment overlying sand and silt, which is underlain by bedrock, outside of the highway embankment the silt is overlain by organic material.

The purpose of this section of the report (Part B) is to provide embankment design recommendations for staging and culvert replacement. These are based on the conditions encountered at the borehole locations, TBTE's interpretation of the subsurface conditions at the site and analyses of embankment stability.

9 Structure Foundations

Multiple foundation systems have been considered for the proposed culvert replacement. The foundation systems considered were:

- Closed Bottom Culverts
- Spread Footings on Native Soil
- Spread Footings on Rock Fill
- Driven Piles or Sheet Piles

Design recommendations for viable foundation systems are presented below in Table 9.1 based on the subsurface conditions encountered on site and the existing embankment profile.

Unless noted otherwise, foundation design parameters are given for static, vertically and concentrically loaded foundations in compression.

9.1 Initial Review of Foundation Options

Several options for the proposed culvert replacement were reviewed from a foundations perspective and are presented in below. Options reviewed address closed bottom culverts, open footing culverts and a sheet pile structure.

Table 9.1: Foundation Options

Option		Advantages	Disadvantages	Comment
Closed Bottom Culvert(s)	Typical, steel or concrete culvert with appropriate bedding. Similar to existing culvert.	- least costly option - less excavation required than open footing culvert options - least construction time required	- requires construction within the creek	Preferred Geotechnical Option
Open Footing Culvert	Footings on Native Silt	- longer spans may be considered to minimize construction within the existing channel - least excavation required of footing options - least costly footing option - no rock fill required below the footing	- excavation below ground and surface water is required, complete dewatering will be required - low geotechnical resistance and reactions - the native silt material is sensitive to disturbance from construction activities and construction traffic - highest risk footing option for frost effects unless extensive fill cover is provided -the low embankment height may require a low-profile culvert. -longer spans may require multiple foundations	-
	Footing on Rock Fill	- longer spans may be considered to minimize construction within the existing channel - highest geotechnical capacities for footings - precast footings may be considered - rock fill cover and pad below footing can be considered to reduce / limit frost effects	- excavation below ground and surface water is required, less dewatering required - additional cost of rock fill - rock fill cannot be compacted below water, as such verification of a level of compaction measurement is challenging - potential disturbance of subgrade during excavation - the low embankment height may require a low-profile culvert. -longer spans may require multiple foundations	-
	Driven Piles	- typically high capacities can be achieved - excavation below water level may be reduced or eliminated - longer spans may be considered to minimize construction within the existing channel	- inadequate pile lengths to achieve lateral capacity may occur. -additional investigation to determine rock line for a more accurate pile length - the low embankment height may require a low-profile culvert. -longer spans may require multiple foundations	-
Sheet Pile Structure	Sheet piles with structural slab.	- less excavation required than open footing culvert options - can be constructed outside of channel footprint - construction within the existing channel can be minimized	-speciality contractor, and equipment -cobbles and boulders may hinder installation there by producing a lower capacity -potentially high variability in pile lengths may be encountered due to shallow bedrock and/or cobbles and boulders -additional investigation to determine rock line for a more accurate pile length	-

9.2 Closed Bottom Culverts

Closed bottom culvert(s) can be placed on and in compacted granular material in an earth excavation or embankment. The culvert shall be placed on bedding fill material and backfilled in accordance with the appropriate OPSD 802 series drawings. Possible applicable OPSD drawings include; 802.020, 802.024, 802.031, 802.034, 802.051, and 802.054. The designer should choose which is the most appropriate drawing for the actual culvert chosen.

A resistance factor of 0.5 has been applied for the estimation of the factored geotechnical resistance at ULS. Settlements for SLS have been estimated assuming a uniform pressure distribution over the entire base of the foundation, with an allowance for potential of some disturbance of the founding surface during construction. A resistance factor of 0.8 has been applied.

Geotechnical resistances at ULS and geotechnical reactions at SLS for closed bottom culverts founded on native silt are provided below, and are subject to the following conditions:

- A minimum depth of cover 0.3 m (depth of soil to the underside of the foundation) must be provided.
- Foundations shall be placed on a minimum of loose native sand or silt.
- Vertically and concentrically loaded foundations in compression
- Assumed elevation of 415.8 m at underside of footing.

**Table 9.2: Geotechnical Resistances and Reactions
Closed Bottom Culvert**

Effective Footing Width (m)	Depth of Cover to Underside of Footing (m)	Factored Geotechnical Resistance, ULS (kPa)	Factored Geotechnical Reaction, SLS (kPa) for 25 mm settlement	Factored Geotechnical Reaction, SLS (kPa) for 50 mm settlement
5	0.3	150	52	72

9.3 Spread Footings

Spread footings are considered to be appropriate for open footing culverts. A resistance factor of 0.5 has been applied for the estimation of the factored geotechnical resistance at ULS. Settlements for SLS have been estimated assuming a uniform pressure distribution over the entire base of the foundation, with an allowance for potential of some disturbance of the founding surface during construction. A resistance factor of 0.8 has been applied.

Any divergence from the conditions described herein could result in the reduction of ULS values presented. For example if the foundation is placed shallower (less depth of cover to the underside of footing) and/or the ground is sloping away from the foundation, a reduction in the ULS values may be realized.

To eliminate the effects of frost, footings must be placed below the depth of frost penetration or placed over/within non-frost susceptible fills (such as rock fill) which extend from the top of creek low water level or backfill (which ever will govern) to the depth of frost penetration. Volumetric expansion of the water (when frozen) within the pore space of the rock needs to be considered by the designer. This expansion (heave) can be estimated at 1 % of the height of the rock fill pad which is subjected to frozen condition.

9.3.1 Spread Footings on Rock Fill

Geotechnical resistances at ULS and geotechnical reactions at SLS for typical footings founded on rock fill are provided below, and are subject to the following conditions:

- A minimum depth of cover 1.3 m (depth of soil to the underside of the foundation) must be provided over a distance of at least 5 times the footing width from the edge of footing. The depth of cover has been determined from the slope stability modelling. Cover material should consist of Granular B Type II or rock fill.
- Foundations shall be placed on a minimum 1.0 m thick compacted graded rock fill pad founded on loose native sand or silt.
- Vertically and concentrically loaded foundations in compression.
- Assumed elevation of 414.5 m at underside of footing.

Table 9.3: Geotechnical Resistances and Reactions
Strip Footings on Rock Fill

Effective Footing Width (m)	Thickness of Rock Fill Below Footing (m)	Depth of Cover to Underside of Footing (m)	Factored Geotechnical Resistance, ULS (kPa)	Factored Geotechnical Reaction, SLS (kPa) for 25 mm settlement	Factored Geotechnical Reaction, SLS (kPa) for 50 mm settlement
1.2	1.0	1.3	315	100	180
1.5	1.0	1.3	335	85	150
1.8	1.0	1.3	355	70	130

2.0	1.0	1.3	355	65	120

For the values presented above resistance factors of 0.5 and 0.8 have been applied for the calculation of factored geotechnical resistance at ULS and the geotechnical reactions at SLS, respectively. The SLS reactions have been computed for settlements of up to 25 mm and 50 mm under foundation loading. The resistance factors are as provided in the 2014 Canadian Highway Bridge Code (CHBC).

The rock fill pad should consist of graded rock fill. The upper 150 mm of the rock fill pad should be constructed with 19 mm clear stone. The base of the pad should extend horizontally beyond the edge of the footings by a distance at least equal to the thickness of the rock fill pad provided.

The excavations required for construction of the rock fill pad should be considered when planning for the locations of the footings especially if construction within the existing channel is not permitted.

9.3.2 Spread Footings on Native Silt

Geotechnical resistances at ULS and geotechnical reactions at SLS for typical footings founded on native silt are provided below, and are subject to the following conditions:

- A minimum depth of cover 2.3 m (depth of soil to the underside of the foundation) must be provided over a distance of at least 5 times the footing width from the edge of footing. The depth cover is taken as the depth of frost penetration to protect the foundation from frost action. Cover material should consist of Granular B Type II or rock fill.
- Foundations may be placed on loose to compact native silt.
- Foundations will be completed in the dry.
- Vertically and concentrically loaded foundations in compression.
- Assumed elevation of 413.5 m at underside of footing.

Table 9.4: Geotechnical Resistances and Reactions Strip Footings on Native Silt

Effective Footing Width (m)	Depth of Cover to Underside of Footing (m)	Factored Geotechnical Resistance, ULS (kPa)	Factored Geotechnical Reaction, SLS (kPa) for 25 mm settlement	Factored Geotechnical Reaction, SLS (kPa) for 50 mm settlement
1.2	2.3	270	75	135
1.5	2.3	275	65	115
1.8	2.3	280	55	100

2.0	2.3	285	50	95

For the values presented above resistance factors of 0.5 and 0.8 have been applied for the calculation of factored geotechnical resistance at ULS and the geotechnical reactions at SLS, respectively. The SLS reactions have been computed for settlements of up to 25 mm and 50 mm under foundation loading. The resistance factors are as provided in the 2014 Canadian Highway Bridge Code (CHBC).

Subgrade soils found at the site are sensitive to disturbance. Equipment and/or worker traffic on this material should be kept to a minimum during excavation to prevent excessive disturbance and loss of strength of the subgrade.

The excavations required for construction should be considered when planning for the locations of the footings especially if construction within the existing channel is not permitted.

9.4 Resistance to Lateral Loads

Resistance to lateral forces (sliding) shall be calculated in accordance with Section 6.10.5 of the CHBDC using the following unfactored parameters and appropriate resistance factor from Section 6.9.1 of the CHBDC be applied:

- Between granular pads and pre -cast concrete
    - Co-efficient of friction of 0.5
- Between cast in place concrete and silt subgrade
    - Co-efficient of friction of 0.4

9.5 Sheet Pile Foundations

A sheet pile culvert configuration could be considered for this location.

The factored geotechnical resistance for sheet piling bearing on bedrock has been calculated based on static analyses and through WEAP analysis methods. The methodology provided by Tomlinson and Woodward (2008) was used for the static analysis and provided a geotechnical resistance (with a resistance factor of 0.4) that exceeds the structural capacity of 350 grade steel. WEAP analyses was completed to assess a driving criteria for the sheet piles and to determine the available geotechnical resistance of the piles. It is assumed that the sheet piles will be driven through the existing fills starting at a depth below that of the top of roadway surface.

Potential for difficult driving conditions may be experienced at some locations due to the presence of cobbles and boulders. Numerous cobbles and boulders were noted at a depth of 7.6 m in Borehole 5.

End Bearing Sheet Piles

For this analysis the following assumptions have been made:

- Sheet piles are PZ22 made from 350 grade steel,
- Resistance factor of 0.4 for dynamic analysis as per the 2014 CHBC
- Driving stress will not exceed 80% of steel yield stress.
- Minimum sheet pile length below the dredge line of 5 m.
- Sheet pile bearing on bedrock
- Dredge line has been estimated at elevation 415.5 m.

Based on the above; piles with a minimum length greater than or equal to 5 m below the dredge line driven to bedrock yield the following

- Factored geotechincal resistance ULS of 900 kN per meter length of pile
- Factored geotechnical reaction SLS does not govern.

Piles driven to bedrock should be driven in accordance with OPSS 903. Care must be taken to ensure piles are not overstressed during driving.

Friction Sheet Piles

For sheet piles not bearing on bedrock, the geotechnical resistance of the sheet piling is developed through the skin friction between the steel sheet pile wall and the native silt soils. The geotechnical resistance of sheet piles not on bedrock will vary depending on the depth of installation and the steel section selected. Based on guidelines as provided in the 2014 Canadian Highway Bridge Code (CHBC).commentary, the factored geotechnical resistance of sheet piles that do not reach bedrock (in silt) can be calculated using the following equation:

$$Qf = 0.4 \times \beta \times \sigma'v \times Ap$$

Where:

- Q_f = Factored geotechnical resistance (kN per sheet)
- β = Empirical factor that is based on the effective angle of internal friction of the soil mass. It has been assumed that the sheet pile will terminate within the native silt material. The native silt has an effective angle of internal friction of 29° based on published correlations and borehole data. As such beta can be taken as 0,48.for driven piles.
- A_s = Area of one side of the sheet pile in contact with soil below the dredge line. (m^2)
- Sigma Prime = Effective overburden stress at depth "h" below the dredge line. It has been assumed that the sheet pile will terminate within the native silt material.
- $\sigma'v = h \times (\gamma_s - \gamma_w)$
    - Unit weight of native silt $\gamma_s = 18$ kN/ m^3
    - Unit weight of water $\gamma_w = 9.8$ kN/m^3
    - h = Depth of embedment below dredge line (m)

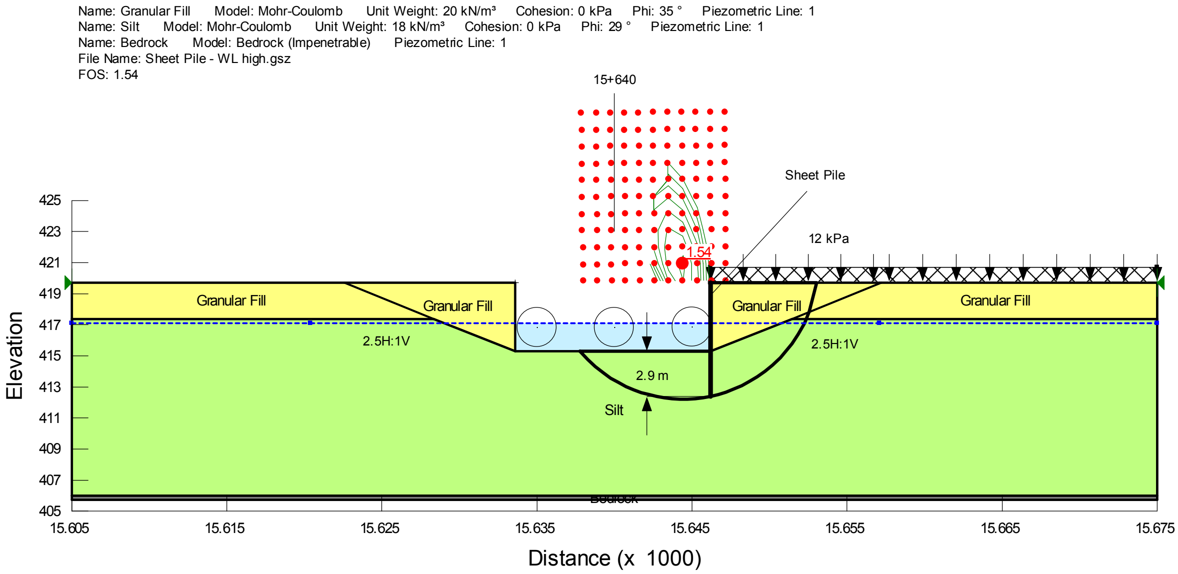
Factored geotechnical reactions (SLS) does not govern for friction piles. Settlement will be less than 25 mm at ULS.

It is recommended that all piles be advanced to bedrock. If both end bearing and skin friction piles are used, differential settlements of piles must be accounted for. The differential settlement between friction and end bearing piles can be taken to equal 100 % of the estimated settlement of the friction pile.. Mixed piles should not be used under the same foundation element.

9.5.1 Sheet Pile Global Stability

In order to protect the system against rotational failure the sheet piles must be driven to a minimum depth of 2.9 m below the dredge line to provide a minimum factor of safety of (FoS) of 1.54 (resistance factor of 0.65) for final configuration. The resistance factor is as presented in the 2014 Canadian Highway Bridge Code. Material properties are provided in Section 11. Sheet pile structural design requirements may require deeper embedment.

Figure 9.1 – Minimum Sheet Pile Wall Depth, Profile Section



9.6 Driven Piles

Driven piles for an open footing culvert solution can be considered. The use of piled foundations must consider the quality and variation of the bedrock surface. The following pile recommendations are subject to the following conditions:

- Pile type is a HP 310 x 110
- Steel graded to 350 MPa
- Assumed pile cap elevation at approximately 415.5 m.
- Piles for the east abutment are end bearing on bedrock. The provided resistances are based on static analysis.

End Bearing Piles

For design purposes the pile capacities indicated below are appropriate.

Table 9-5 - Pile Design Capacities for Piles Driven to Bedrock

Abutment	Pile Designation	ULS Factored Geotechnical Axial Resistance	SLS Geotechnical Resistance for 25 and 50 mm of Settlement
East	HP 310x110	2000 kN	Does not govern

For estimation quantity purposes only, the pile tip elevations of 410 m for the east abutment and 407 m for the west abutment may be used. Pile lengths can be estimated based on the pile cap elevation as determined by the designer.

Piles driven to bedrock should be driven in accordance with OPSS 903. Care must be taken to ensure piles are not overstressed during driving. The installed depth of the piles may vary. The contractor must be prepared to drive piles of varying length.

The piles should be equipped with a driving shoe to limit pile “walking” and prevent damage to the pile tip. The driving shoe can be a Titus Rock Injector, or equivalent, if it is not available Oslo points as indicated in OPSD 3000.201 can be used. The behaviour of the piles should be monitored during driving for any signs indicative of pile damage, walking or skipping. It should be noted that cobbles and boulders were encountered within Borehole 5.

Friction Piles

For piles not bearing on bedrock, the geotechnical resistance of the piling is developed through the skin friction between the steel pile perimeter and the native silt soils. The geotechnical resistance of piles not on bedrock will vary depending on the depth of installation and the steel section selected. Based guidelines as provided in the 2014 Canadian Highway Bridge Code (CHBC)., the factored geotechnical resistance of driven piles that do not reach bedrock (in silt) can be calculated using the following equation:

$$Q_f = 0.4 \times \beta \times \sigma'v \times Ap$$

Where:

- Q_f = Factored geotechnical resistance (kN)
- β = Empirical factor that is based on the effective angle of internal friction of the soil mass. It has been assumed that the pile will terminate within the native silt material. The native silt has an effective angle of internal friction of 29° based on published correlations and borehole data. As such beta can be taken as 0,48.for driven piles.
- A_s = Area of the pile in contact with soil below the dredge line. (m^2)
- Sigma Prime = Effective overburden stress at depth “h” below the pile cap. It has been assumed that the pile will terminate within the native silt material.

- $\sigma'v = h \times (\gamma_s - \gamma_w)$
    - Unit weight of native silt γ_s = 18 kN/ m^3
    - Unit weight of water γ_w = 9.8 kN/m^3
    - h = Depth of embedment below pile cap (m)

Factored geotechnical reactions (SLS) do not govern for friction piles. Settlement will be less than 25 mm at ULS.

If both end bearing and skin friction piles are used, differential settlements of piles must be accounted for. Mixed piles should not be used under the same foundation element.

Piles should be spaced at least 2.5 pile widths apart (centre to centre). No load reduction is required for this pile spacing.

Downdrag loads are not anticipated since there is no proposed change in vertical alignment.

Pile caps should be protected from frost action, or placed below the estimated frost penetration depth.

10 Culvert Camber

It is understood that the existing embankment will not be raised and no appreciable settlements are expected. Culverts will not require camber.

11 Culvert Replacement – Staging

11.1 Staging – General

The replacement of the culvert can be completed utilizing a staged construction methodology. To provide a single trafficable lane (during construction) and expose sufficient length of existing culverts, and attempt to avoid utility poles (both sides of the highway) the vertical profile of the roadway will need to be temporarily lowered (0.6 m) and a temporary widening will also be required. The temporary widening can be expected to experience approximately 30 mm of total settlement. However due to the temporary nature of this widening it is anticipated that only a portion of this total settlement will occur

11.2 Staging - Geotechnical Model

Stability modeling was completed using Slope/W software and limit equilibrium analysis using the Morgenstern-Price method.

The soil properties established for the embankment are presented below.

Stability analyses have been completed to investigate potential configurations for the proposed embankment during construction for the proposed culvert replacement. The design was based on providing a minimum calculated factor of safety (FoS) of 1.33 (resistance factor of 0.75) during construction (staging embankments) and a (FoS) of 1.54 (resistance factor of 0.65) for final configuration. The resistance factors are as provided in the 2014 Canadian Highway Bridge Code. A uniformly distributed traffic load of 12 kPa over the traversable lane(s) was applied in all cases.

Table 11.1: Stability Analyses Soil Properties

Soil	Effective Shear Strength Properties		Unit Weight, γ (kN/m³)
	Effective Angle of Internal Friction, φ' (degrees)	Effective Cohesion Intercept, C' (kPa)	
Granular B Type II	35	0	20
Rock Fill	40	0	18
Existing Granular Fill	35	0	20
Native Silt	29	0	18

11.3 Stability Analysis Results and Recommendations

The culvert can be replaced in two stages, with traffic maintained over alternate sections. This may require a significant longitudinal section of temporary road construction to achieve a temporary vertical alignment. The final roadway embankment will then be restored at its current location (Stage 3).

Various slope configurations were analyzed to determine sections which would meet the design stability requirements.

The following assumptions were made for the analysis:

- Assumed water level to be maintained at natural levels for profile excavation and removal of culverts (Figure 11.1), and cross section culvert removal and staging (Figure 11.2)
- Assumed water level to be maintained at the base of excavation for the excavation and construction of shallow footings on rock fill. (Figure 11.3)
- Temporary reduction in grade raise of 0.6 m.

The following recommendations have been derived from the analysis:

- Cut slopes through existing embankment fills shall be constructed at 1.5(H):1(V) or flatter (Figure 11.2).
- Cut slopes through the native silts for culvert removal should be constructed at 2.5(H):1(V), or flatter (Figure 11.1)
- Cut slopes through the native silts for shallow foundation excavations should be constructed at 2.6(H):1(V), or flatter (Figure 11.3)
- Where dewatering is required for shallow foundation construction, granular sheeting (1.2 m thick) is to be placed on cut slopes through native silts prior to dewatering to prevent failure due to rapid drawdown (Figure 11.3).
- For permanent construction the minimum depth of cover for shallow footings on a 1 m thick rock fill pad shall be maintained at 1.3 m (Figure 11.4)
- The minimum depth of cover of 2.3 m for foundations placed on native silt shall be provided for frost protection (Figure 11.5)

Figure 11.1 – Culvert Excavation, Profile Section

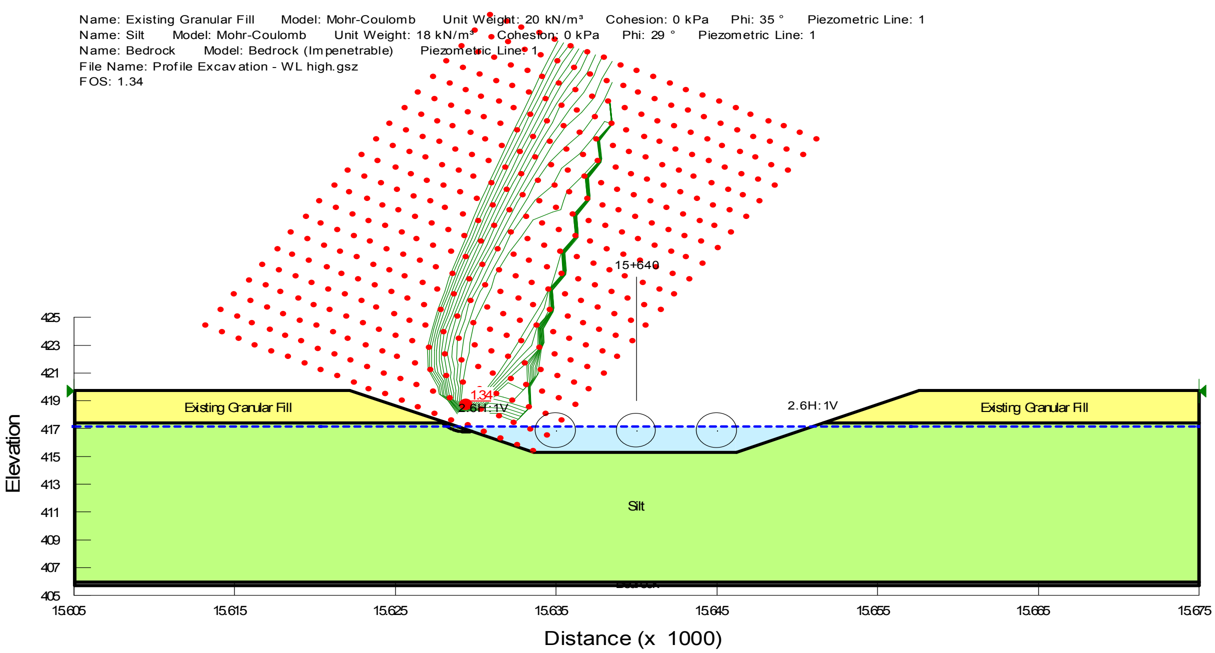


Figure 11.2 – Culvert Excavation and Staging, Cross Section

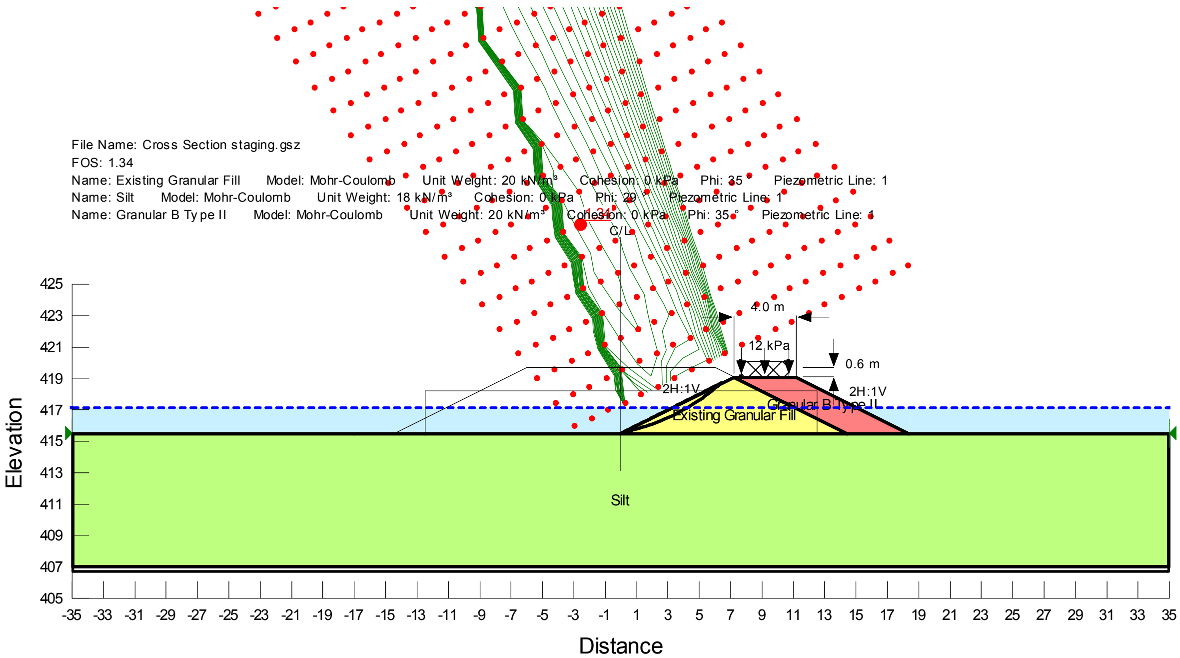


Figure 11.3 – Shallow Foundation Excavation, Profile Section

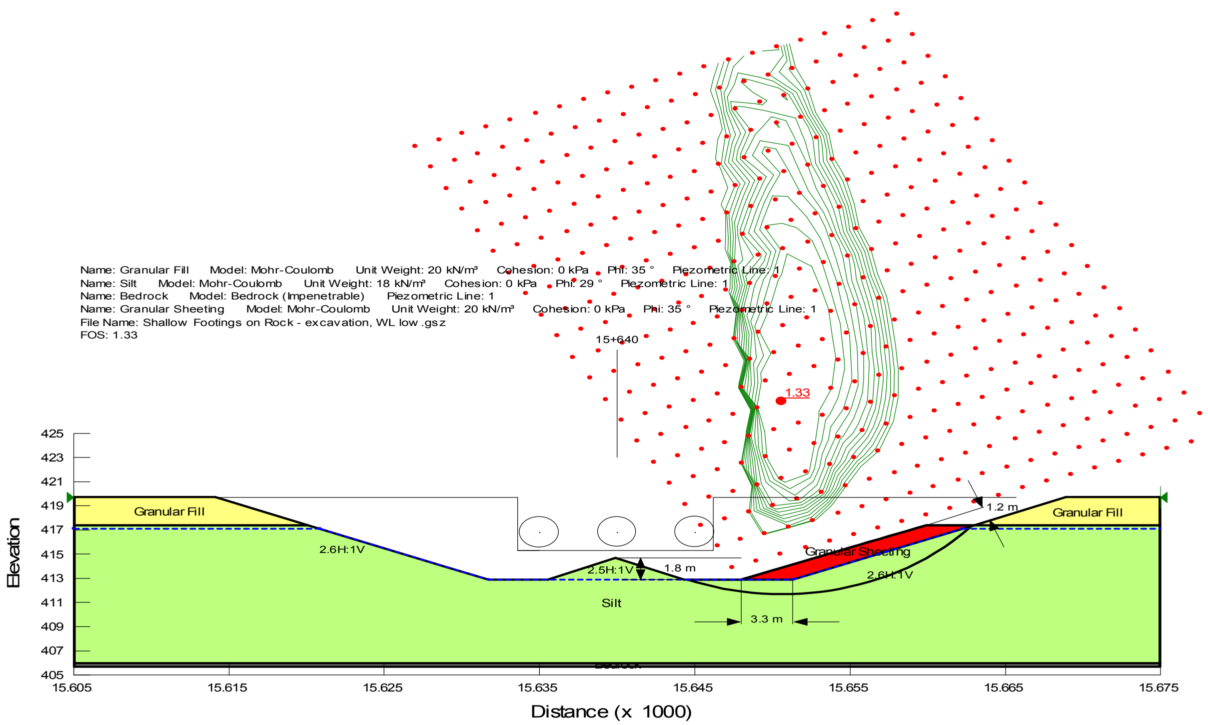


Figure 11.4 – Shallow Foundation on Rock Fill Pad, Profile Section

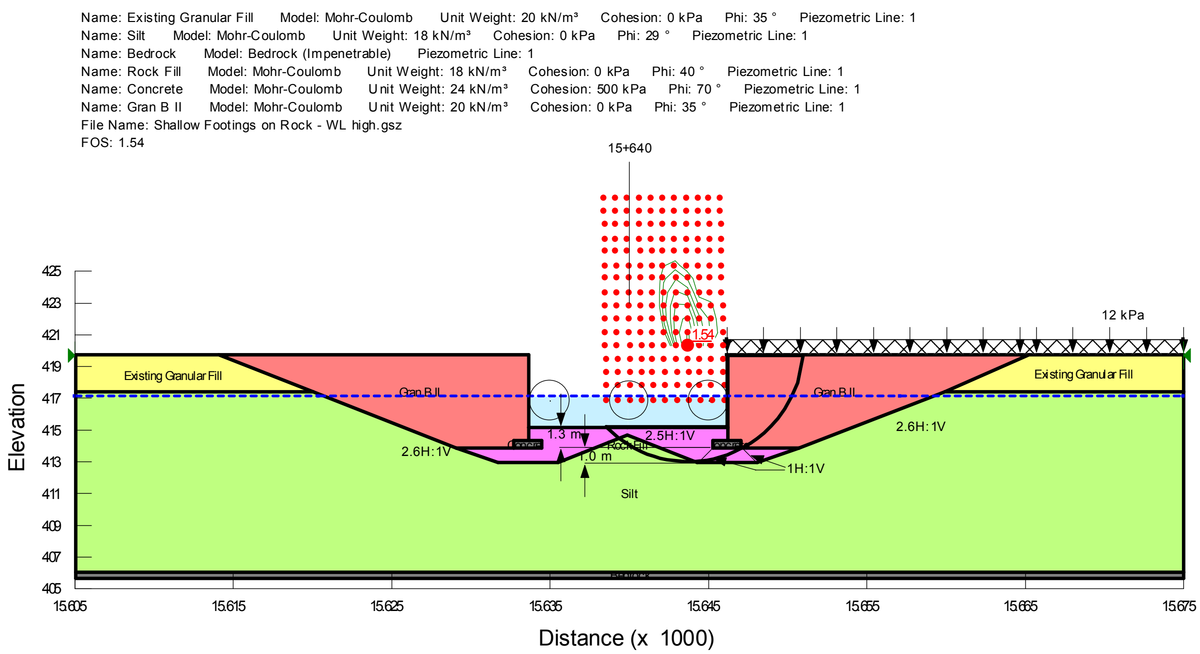
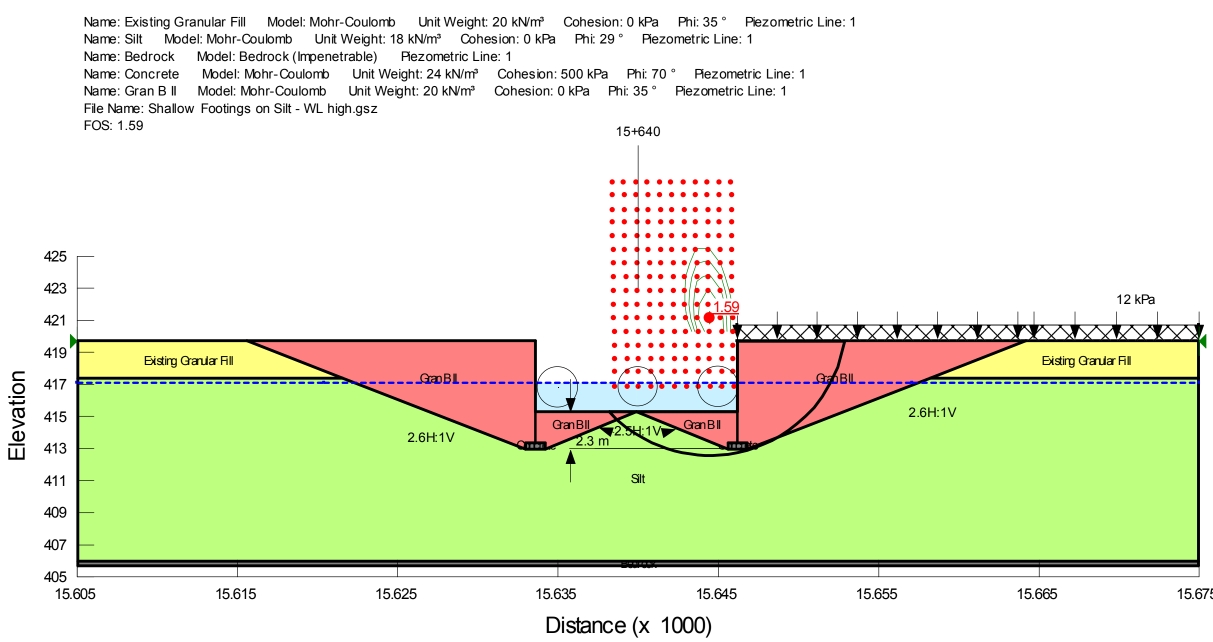


Figure 11.5 – Shallow Foundation on Native Silt, Profile Section



12 Permanent Embankment

Stability analyses have been completed to determine the final embankment configuration for the proposed culvert replacement. The design was based on providing a minimum calculated factor of safety (FoS) of 1.54 (resistance factor of 0.65 as provided in the CHBDC) and FoS of 1.3 for final configuration. A uniformly distributed traffic load of 12 kPa over the traversable lane(s) was applied.

Permanent slopes constructed of compacted Granular B, Type II shall be constructed at:

- 2(H):1(V) or flatter with a 1 m high 3 m long flanking berm for a FoS of 1.54 or greater as per Figure 12.1
- 2(H):1(V) or flatter for a FoS of 1.3 or greater as per Figure 12.2.

Figure 12.1 – Re-Built Cross Section Minimum FoS 1.54

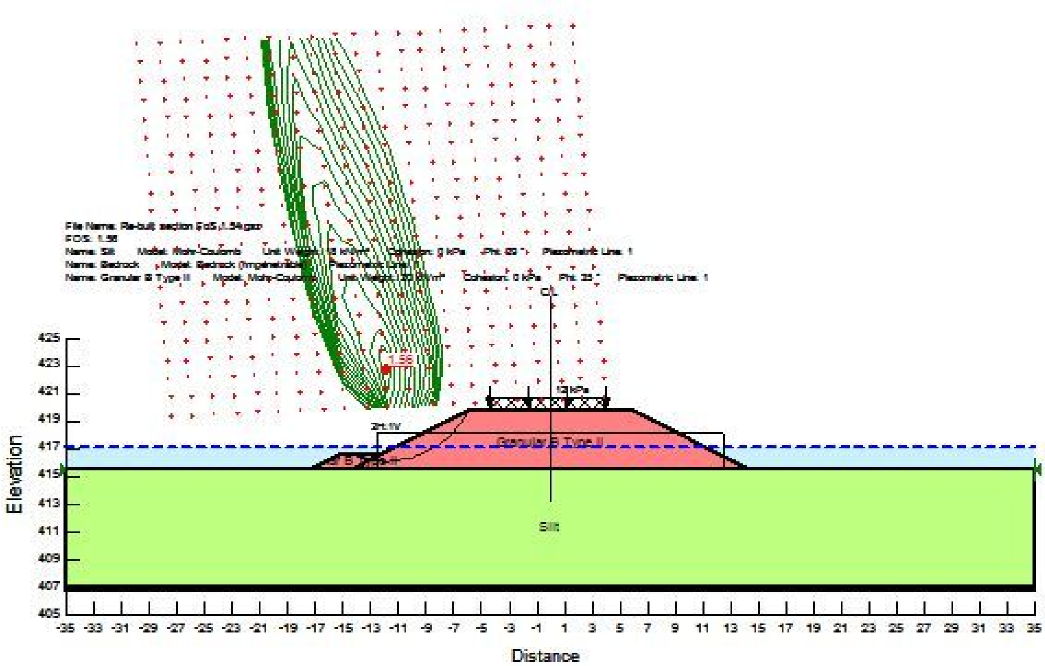
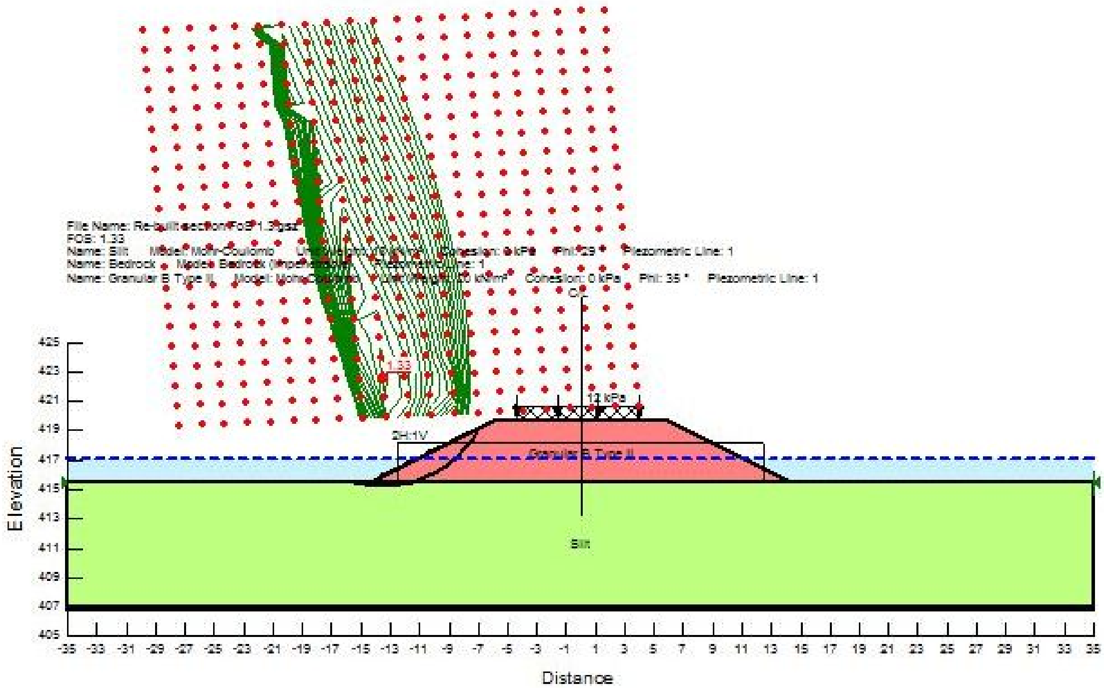


Figure 12.2 – Re-Built Cross Section Minimum FoS 1.3



It is understood that there will be no grade raise and no appreciable settlements are anticipated. Any settlement experienced from a temporary widening to facility staging will be minimal and will not affect overall performance of the roadway. Settlement from the flanking berm will minimal and will be less than a 25 mm, assuming all organics are removed from beneath the flanking berm.

13 Temporary Roadway Protection

The implementation of temporary roadway protection is not anticipated for this culvert replacement. Traversable lanes can be provided through stable embankment construction and the temporary lowering of the vertical alignment. Temporary roadway protection will likely prove cost prohibitive.

Where a sheet pile culvert system is utilized or where a sheet pile cofferdam system is utilized, roadway protection using sheet piles may be cost effective. Traffic protection systems, if considered, can consist of interlocking sheet piles, or soldier piles and lagging. Due to shallow bedrock at some locations, relative to the current culvert inverts, simple cantilevered systems may not prove feasible. Temporary shoring of the traffic protection system may be required to provide lateral resistance. Shoring measures could consist of grouted anchors, rakers, tiebacks and/or deadmans. Design of any shoring should consider the geometry of the embankment and its ability to provide adequate embedment, and/or cover, and lateral spacing to ensure no overlap of the active wedge of any anchors and the passive wedge of the traffic protection. All traffic protection systems should be designed in accordance to OPSS 539 to Performance Level 2, by engineers with a minimum of five years of experience designing similar systems. Design should also include the global stability of the chosen traffic protection system. Design of roadway protection systems is the responsibility of the contractor. Material properties as used in stability analyses may be utilized.

14 Backfill and Lateral Earth Pressures

The existing site materials are not suitable for use as structural backfill. Structural backfill should consist of Granular "B" Type I, or II. Granular "A" may be specified as structural backfill in specific zones.

Lateral earth pressure coefficients for potential granular backfill and level ground conditions have been provided below.

Table 13.1: Lateral Earth Pressure Coefficients

Lateral Earth Pressure Coefficients (K)					
Compacted Granular Backfill	ϕ' (°)	Bulk Unit Weight of Soil, γ (kN/m^3)	Active Ka	At Rest Ko	Passive Kp
OPSS Granular A, or Granular B Type II	35	20	0.27	0.43	3.7
OPSS Granular B Type I	32	20	0.47	0.31	3.3
Native Silt	29	18	0.35	0.52	2.9

No factor of safety or resistance factor has been included in the above coefficients. A compaction surcharge should be added in accordance with the CHDBC s6-14 Section 6.12.3. The culvert must also be designed to resist hydrostatic pressures where applicable.

15 Dewatering, Excavations and Channel Diversion

Excavations should be excavated and sloped in accordance with the requirements of the Occupational Health and Safety act. Dewatering systems should be designed in accordance to OPSS 517 and SP 517F01 (July 2017), and it is recommended that any dewatering system be designed and checked by engineers with a minimum of five years of experience designing similar systems.

The current creek level is approximately 1.0 m above the existing inlet invert. The soils below the ground water level consist of relatively low permeable silt. Excavations for culvert construction and/or placement of fill are expected to extend below the ground water level.

To facilitate construction in the dry, control of surface and ground water will be required. Dewatering of the site will likely require the use of coffer dams constructed across the water course.

Dewatering of the excavation may include simple sump and pump techniques ranging to well point systems. Simple sump and pump techniques may be adequate when surface water flow through the upper sand zone is limited. Where high water flow through the upper sand zone is anticipated well points can be used to drop the water level within this zone. The complexity of the dewatering system will be governed by the depth of the excavation and any requirements for working in the dry. Protection from rapid draw down effects will be required to prevent the potential instability of the excavation slopes. Granular sheeting covering the cut slopes in native silt will be required to project the embankment slopes from rapid draw down effects. Silt embankments should be sloped at 2.6(H) to 1 (V) and covered with granular sheeting 1.2 m thick sloped to match the cut slope. Additionally, well points could be used to mitigate rapid drawdown effects within the silt.

The soil through the embankment and the native silt can be preliminarily classified as Type 3 soils, as defined by the Occupational Health and Safety Act and Regulations for Construction Projects. Cut slopes for unsupported excavations shall be no steeper than those provided in Section 11.3. The soil types must be reassessed as excavations proceed and adjustments to construction methodologies should be taken as required.

Channel diversion options are limited without the construction of a diversion and subsequent temporary culvert. The use of temporary cofferdams utilizing either controlled flow or pumping should be considered the best option for channel diversion.

15.1 Preliminary Considerations for Cofferdams

The potential use of cofferdams to control inlet and outlet water conditions can be considered at this location. A cofferdam system can range from earthen structures to sheet piles. Subsurface investigations were completed near potential cofferdam locations (Boreholes 3, 5, 6 and 8). Bedrock was encountered at Boreholes 3 and 5 at depths of 8.1 and 9.4 m (elev. 409.9 and 409 m), and no refusal was encountered at Boreholes 6 and 8 which were advanced to depths of 11.2 m, (elev. 406.9 and 406.7 m). It should be noted that cobles and boulders were noted within Borehole 5. Boreholes 3 and 5 are on the right side of the highway and Boreholes 6 and 8 are on the left side of the highway.

Cofferdam design should be completed by the contractor’s designer and consider, but not limited to, the following potential issues:

- Requirement for bracing and/or tie backs;
- Global and internal stability;
- Sufficient seepage cut off measures be employed to avoid piping of the soil. The native silt is of low permeability and may pipe if sufficient seepage exit gradients develop on the inboard side of the cofferdam.
- Potential loss of soil adjacent to the cofferdam.
- Potential sheet pile refusal on cobbles and boulders (as indicated in Borehole 5), or shallow bedrock.

16 Estimated Frost Depth and Frost Protection

Based on OPSD 3090.100 Foundation Frost Penetration Depths for Northern Ontario; the estimated frost depth penetration within the expected embankment fill is 2.3 m. The embankment soils anticipated within the frost depth are considered to be of low frost susceptibility (MTO Pavement Design and Rehabilitation Manual).

17 Seismic Considerations

Seismic analysis for the culvert will not be required based on the following rationale as per the 2014 Canadian Highway Design Bridge Code (CHBDC). In accordance with Section 4.4.3.1 spectral ground acceleration data for the sites was obtained from www.earthquakescanada.nrcan.gc.ca. In accordance with Section 4.4.4, Table 4.10 and assuming the culverts have a Seismic Importance Category of "Major-route and other bridges", the site is classified as Seismic Performance Category 1. As per Section 4.4.5.1, no seismic analyses are required for structures located in Seismic Performance Category 1.

18 Corrosion and Sulphate Attack Potential

Corrosivity and sulphate content testing was conducted on a sample of the native soil, and the results are provided in Appendix B. The results of the test indicate the following conditions at the test location:

- Sulphate was measured at 42 ppm (0.0042%) and does not require sulphate resistant concrete since it is less than 0.1 %.
- The pH of the soil was measured at 7.8, with resistivity of 2760 ohm-cm, and sulphide content less than 0.2 mg/kg. Considering these factors, the native soils are not considered to be aggressively corrosive.

19 Scour and Erosion Protection

Erosion/scour protection should be provided at culvert inlet and outlet, and for any foundation element. The ultimate design of erosion protection measures should be provided by designers with sufficient experience. Where appropriate, foundation elements should be provided with sufficient scour protection in the event of elevated creek levels. Scour protection should be designed in accordance with Section 1.9.5 of the 2014 Canadian Highway Bridge Design Code, where clay seals are considered

OPSS 1205 should be reviewed and OPSD 810.010 for rip rap placement should be reviewed.

20 Potential Construction Issues

No major construction difficulties are foreseen at this site. Issues which may require consideration include:

- Control of surface and groundwater during excavation below the creek/groundwater level.
- Potential for construction ‘in the wet’.
- Staging Requirements.
- Potential additional drilling to determine bedrock depth near Boreholes 7 and 8.
- Native subgrade is highly susceptible to disturbance and construction traffic on it should be kept to a minimum.
- Cobbles and boulders were noted within Borehole 5.
- Dewatering of the site to facilitate construction in the dry may be subject to rapid drawdown issues in the silt and a higher flow from sand seams.

21 Limitations

Conclusions and recommendations presented in this report are based on the information determined at a limited number of test hole locations. Subsurface and groundwater conditions between and beyond these locations may differ from those encountered. Conditions may become apparent during construction that were not detected and could not be anticipated at the time of the site investigation.

The comments given in this report on potential construction problems and possible methods of construction are intended only for the guidance of the designer.

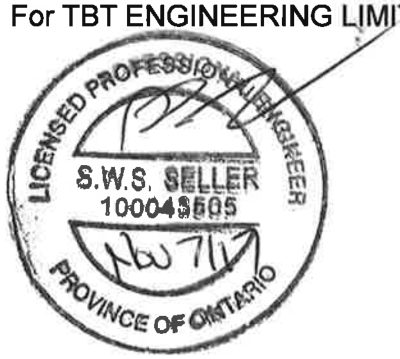
Groundwater levels indicated are based on the information described within the report. The presence of all conditions that could affect the type and scope of dewatering procedures which may be considered cannot readily be determined from boreholes. These include local and seasonal fluctuations of the groundwater level, changes in soil conditions between test locations, thin and/or discontinuous layers of highly permeable soils, etc.

The information contained within this report in no way reflects any environmental aspect of the site or soil.

22 Closure

We trust the above addresses your project requirements at this time. Should you have any questions or comments, please do not hesitate the contact us at your convenience.

Yours truly,
For TBT ENGINEERING LIMITED



Steven Seller, P.Eng
Senior Project Engineer

Wayne Hurley, P.Eng.
Principal Contact for MTO Foundations

APPENDIX A
Borehole Logs

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

RECORD OF BOREHOLE No 1

METRIC

W.P. 5119-06-00	LOCATION Knife Creek N:5355264.472; E:228612.697 MTM Zone:13		ORIGINATED BY WM
DIST NER HWY 17	BOREHOLE TYPE Hollow Stem Auger		COMPILED BY SW
DATUM Geodetic	DATE 2016.09.20 - 2016.09.20	LATITUDE 48.331512 LONGITUDE -85.02764	CHECKED BY CJ

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
419.7					
419.6 0.1	ASPHALT - 75 mm				
	FILL - GRAVEL & SAND - trace silt, brown, dense to very dense	1	AS		58 38 (4)
	----- - white & brown	2	SS	100+	
	----- brown, numerous cobbles	2	SS	49	
417.4 2.3	SAND - Silty, grey, compact	4	SS	19	
416.7 3.0	SILT - some sand, grey, trace gravel, loose to compact	5	SS	7	3 19 (78)
		6	SS	14	
		7	SS	6	
		8	SS	19	
		9	SS	100+	
410.2 9.5	BEDROCK - grey and white gneiss	1	RC		RC #1 REC 100% RQD 100%
407.1 12.6	End of Borehole @ 12.6 m.				

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (419, 418, 417, 416, 415, 414, 413, 412, 411, 410, 409, 408); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

+³, ×³: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 2

METRIC

W.P. 5119-06-00 LOCATION Knife Creek N:5355287.823; E:228579.711 MTM Zone:13 ORIGINATED BY WM

DIST NER HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SW

DATUM Geodetic DATE 2016.09.20 - 2016.09.20 LATITUDE 48.331718 LONGITUDE -85.028089 CHECKED BY CJ

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
419.7	Ground surface
419.6 / 0.1	ASPHALT - 75 mm
	FILL - SAND & GRAVEL to trace gravel - Silty, brown, dense to very dense
417.5 / 2.2	SILT - trace sand, grey, very dense to compact
407.0 / 12.7	End of Borehole @ 12.7 m.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AS		
2	SS	72	7 71 (22)
3	SS	36	
4	SS	45	
5	SS	70	
6	SS	92	
7	SS	10	0 1 (99)
8	SS	15	
9	SS	16	
10	SS	12	
11	SS	14	

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (419 to 407); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+³, ×³: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

METRIC

W.P. 5119-06-00 LOCATION Knife Creek N:5355273.629; E:228567.735 MTM Zone:13 ORIGINATED BY TP

DIST NER HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SW

DATUM Geodetic DATE 2016.09.21 - 2016.09.21 LATITUDE 48.331589 LONGITUDE -85.028248 CHECKED BY CJ

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100 | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
418.0 0.0	SILT - some to trace sand, brown, loose to dense		1	AS		0 19 (81)
			2	SS	5	0 5 (95)
			3	SS	11	
			4	SS	28	0 6 (94) Non-Plastic.
			5	SS	9	0 5 (95) Non-Plastic.
			6	SS	18	
			7	SS	18	
			8	SS	36	
409.9 8.1	BEDROCK - grey and white gneiss		1	RC		RC #1 REC 100% RQD 100%
406.8 11.2	End of Borehole @ 11.2 m.					

Elevation scale: 417, 416, 415, 414, 413, 412, 411, 410, 409, 408, 407

+³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 4

METRIC

W.P. 5119-06-00 LOCATION Knife Creek N:5355258.094; E:228598.445 MTM Zone:13 ORIGINATED BY WM

DIST NER HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SW

DATUM Geodetic DATE 2016.09.27 - 2016.09.28 LATITUDE 48.331453 LONGITUDE -85.027831 CHECKED BY CJ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
418.6						
0.0	SILT - Sandy to trace sand, trace gravel, loose to very dense	1	AS			Non-plastic 4 34 (62)
		2	SS	8	▽	
		3	SS	11		0 5 (95) Water level @ 1.1m upon completion.
		4	SS	19		
		5	SS	18		
		6	SS	90		0 2 (98)
		7	SS	27		
411.1 7.5	BEDROCK - grey and white gneiss	1	RC			RC #1 REC 100% RQD 93%
408.0 10.6	End of Borehole @ 10.6 m.					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; ELEVATION SCALE (418 to 408); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3.

+^{3}, ×^{3}: Numbers refer to Sensitivity O$^{3\%}$ STRAIN AT FAILURE

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 5

METRIC

W.P. 5119-06-00 LOCATION Knife Creek N:5355252.716; E:228593.41 MTM Zone:13 ORIGINATED BY WM

DIST NER HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SW

DATUM Geodetic DATE 2016.09.29 - 2016.09.29 LATITUDE 48.331404 LONGITUDE -85.0278979 CHECKED BY CJ

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
418.4 / 0.0	SILT - Sandy to trace sand, trace to some organics, brown, loose to very dense	1	AS			0 31 (69)
		2	SS	5		
	----- - grey	3	SS	5	∇	Water @ 1.6m upon completion.
		4	SS	23		Non-plastic
		5	SS	14		0 0 (100)
		6	SS	66		
		7	SS	29		0 2 (98)
	----- - numerous cobbles & boulders					Auger refusal @ 7.6m. Advanced with casing.
409.0 / 9.4	BEDROCK - grey and white gneiss	1	RC			RC #1 REC 100% RQD 98%
406.0 / 12.4	End of Borehole @ 12.4 m.					

ELEVATION SCALE: 418, 417, 416, 415, 414, 413, 412, 411, 410, 409, 408, 407, 406

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Sample	Natural moisture content (approx. %)
1	53
2	46
3	37
4	23
5	25
6	20
7	19

+3, ×3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 6

METRIC

W.P. 5119-06-00	LOCATION Knife Creek N:5355272.248; E:228621.178 MTM Zone:13		ORIGINATED BY WM
DIST NER HWY 17	BOREHOLE TYPE Hollow Stem Auger		COMPILED BY SW
DATUM Geodetic	DATE 2016.09.30 - 2016.09.30	LATITUDE 48.331583 LONGITUDE -85.027527	CHECKED BY CJ

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
418.1 / 0.0	ORGANICS - black, very loose	1	AS			
		2	SS	2	▽	Water level @ 1.0m upon completion.
416.5 / 1.6	SILT - grey, very loose to compact	3	SS	1		
		4	SS	1		Non-plastic
		5	SS	3		0 0 (100)
		6	SS	18		
		7	SS	26		
		8	SS	21		
		9	SS	18		
	- - - - - - some gravel, trace sand	10	AS			21 7 (72) Could not perform SPT.
406.9 / 11.2	End of Borehole 11.2 m.					Drill rods jammed in hollow stem auger.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (418 to 407); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3.

Water content values off scale: 146, 182.9

$+^3, \times^3$: Numbers refer to Sensitivity   O $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 7

METRIC

W.P.	5119-06-00
LOCATION	Knife Creek N:5355289.06; E:228595.153 MTM Zone:13
ORIGINATED BY	WM
DIST	NER
HWY	17
BOREHOLE TYPE	Hollow Stem Auger
COMPILED BY	SW
DATUM	Geodetic
DATE	2016.02.10 - 2016.02.10
LATITUDE	48.331731
LONGITUDE	-85.027881
CHECKED BY	CJ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
418.1 / 0.0	FILL - SAND & GRAVEL, trace silt, brown	1	AS			41 51 (8)
417.5 / 0.6	SILT - some organics, brown, very loose to compact	2	SS	3	▽	Water level @ 1.0m upon completion.
	- - - - - - grey	3	SS	25		
		4	SS	15		
		5	SS	23		
		6	SS	16		
		7	SS	14		Non-plastic
		8	SS	25		0 1 (99)
		9	SS	30		
407.5 / 10.6	End of Borehole 10.6 m.					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (418 to 408); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

+3, ×3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 8

METRIC

W.P. 5119-06-00 | LOCATION Knife Creek N:5355296.975; E:228601.858 MTM Zone:13 | ORIGINATED BY WM

DIST NER HWY 17 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY SW

DATUM Geodetic | DATE 2016.03.10 - 2016.03.10 | LATITUDE 48.331803 | LONGITUDE -85.027792 | CHECKED BY CJ

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
417.9 / 0.0	TOPSOIL/ORGANICS - brown/black, very loose	1	AS				Water level @ 1.2m upon completion.
		2	SS	3	∇	417	
416.5 / 1.4	SAND - some gravel, some silt, brown, very loose to compact	3	SS	3		416	Non-plastic
		4	SS	13			15 74 (11)
415.0 / 2.9	SILT - grey, compact to dense	5	SS	10		415	
						414	
		6	SS	34		413	
						412	
		7	SS	19		411	
		8	SS	30		410	
						409	
		9	SS	23		408	
	----- - trace sand & gravel	10	SS	20		407	9 5 (86)
406.7 / 11.2	End of Borehole @ 11.2 m.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; 143.8

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

TBT ENGINEERING
CONSULTING GROUP

Strength (MPa)
VH = Very High = >200
H = High = 50-200
M = Medium = 15-50
L = Low = 4-15
VL = Very Low = 1-4

Discontinuity type
B = Bedding joint
J = Cross Joint
F = Fault
S = Shear Plane

Roughness
RU = Rough undulating
RP = Rough planar
SU = Smooth undulating
SP = Smooth planar
LU = Slicken sided undulating
LP = Slicken sided planar

Orientation
F = Flat (0-20°)
D = Dipping (20-50°)
V = Near Vertical (>50°)

Aperture
O = Open
C = Closed
F = Filled

Weathering
U = Unweathered (No signs)
S = Slightly (Oxidized)
M = Moderately (Discoloured)
H = Highly (Friable)
C = Completely (Soil-like)

Spacing
VW = Very wide = >3m
W = Wide = 1-3m
M = Moderate = 0.3-1m
C = Close = 5-30cm
VC = Very close = <5cm

Filling
T = Tight, hard
O = Oxidized
SA = Slightly altered, clay free
S = Sandy, Clay free
Si = Sandy, silty, minor clay
NC = Non-softening clay
SC = Swelling, softening clay
N= No filling

ROCK CORE LOG	Page 1 of 1	
Project #: 16-138	Borehole #1	Lab# 16-16509
Client: MTO NER	Logger: Larry Wells	
Site: Knife Creek Culvert	Date: October 13, 2016	

DEPTH FROM SURFACE (m)	DEPTH (m)	BOX/RUN	% REC (m)	% RQD (m)	GENERAL DESCRIPTION (Rock type(s), %, colour, texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES # OF SETS	TYPE(S)	Orientation	SPACING	Roughness	APERTURE	FILLING	OCCASIONAL FEATURES
From	From 9.60	1	100%	100%	Grey and White Gneiss		u	3	J	F	W	RP	O	N	3 Dipping Unfractured Epidote Planes @ 9.9, 11.0, and 12.0 metres.
To	To 12.60														
From	From														
To	To 0.00														
From	From 0.00														
To	To 0.00														
From	From 0.00														
To	To 0.00														

Full Rock Core Dry

Full Rock Core Wet

-3.0 metre core of Archean Gneiss

TBT ENGINEERING
CONSULTING GROUP

ROCK CORE LOG	Page 1 of 1	
Project #: 16-138	Borehole #3	Lab# 16-16510
Client: MTO NER	Logger: Larry Wells	
Site: Knife Creek Culvert	Date: October 13, 2016	

Strength (MPa)
VH = Very High = >200
H = High = 50-200
M = Medium = 15-50
L = Low = 4-15
VL = Very Low = 1-4

Weathering
U = Unweathered (No signs)
S = Slightly (Oxidized)
M = Moderately (Discoloured)
H = Highly (Friable)
C = Completely (Soil-like)

Discontinuity type
B = Bedding joint
J = Cross Joint
F = Fault
S = Shear Plane

Orientation
F = Flat (0-20°)
D = Dipping (20-50°)
V = Near Vertical (>50°)

Spacing
VW = Very wide = >3m
W = Wide = 1-3m
M = Moderate = 0.3-1m
C = Close = 5-30cm
VC = Very close = <5cm

Roughness
RU = Rough undulating
RP = Rough planar
SU = Smooth undulating
SP = Smooth planar
LU = Slicken sided undulating
LP = Slicken sided planar

Aperture
O = Open
C = Closed
F = Filled

Filling
T = Tight, hard
O = Oxidized
SA = Slightly altered, clay free
S = Sandy, Clay free
Si = Sandy, silty, minor clay
NC = Non-softening clay
SC = Swelling, softening clay
N= No filling

DEPTH FROM SURFACE (m)	DEPTH (m)	BOX/RUN	% REC (m)	% RQD (m)	GENERAL DESCRIPTION (Rock type(s), %, colour, texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES # OF SETS	TYPE(S)	Orientation	SPACING	Roughness	APERTURE	FILLING	OCCASIONAL FEATURES
From	From 8.10	1	100%	100%	Grey and White Gneiss		U	2	J	F	W	RU	O	N	-Unfilled Joint @ 9.4 metres
To	To 11.20														
From	From														
To	To 0.00														
From	From 0.00														
To	To 0.00														
From	From 0.00														
To	To 0.00														

Full Rock Core Dry

Full Rock Core Wet

-3.1 metre core of Archean Gneiss

TBT ENGINEERING CONSULTING GROUP

ROCK CORE LOG

Project # 16-138	Borehole #4	Lab# 16-16511	Page 1 of 1
Client: MTO NER	Logger: Larry Wells		
Site: Knife Creek Culvert	Date: October 12, 2016		

Strength (MPa)
VH = Very High = >200
H = High = 50-200
M = Medium = 15-50
L = Low = 4-15
VL = Very Low = 1-4

Weathering
U = Unweathered (No signs)
S = Slightly (Oxidized)
M = Moderately (Discoloured)
H = Highly (Friable)
C = Completely (Soil-like)

Discontinuity type
B = Bedding joint
J = Cross Joint
F = Fault
S = Shear Plane

Orientation
F = Flat (0-20°)
D = Dipping (20-50°)
V = Near Vertical (>50°)

Spacing
VW = Very wide = >3m
W = Wide = 1-3m
M = Moderate = 0.3-1m
C = Close = 5-30cm
VC = Very close = <5cm

Roughness
RU = Rough undulating
RP = Rough planar
SU = Smooth undulating
SP = Smooth planar
LU = Slicken sided undulating
LP = Slicken sided planar

Aperture
O = Open
C = Closed
F = Filled

Filling
T = Tight, hard
O = Oxidized
SA = Slightly altered, clay free
S = Sandy, Clay free
Si = Sandy, silty, minor clay
NC = Non-softening clay
SC = Swelling, softening clay
N= No filling

DEPTH FROM SURFACE (m)	DEPTH (m)	BOX/RUN	% REC (m)	% RQD (m)	GENERAL DESCRIPTION (Rock type(s), %, colour, texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES # OF SETS	TYPE(S)	Orientation	SPACING	Roughness	APERTURE	FILLING	OCCASIONAL FEATURES
From	From 7.50	1	100%	93%	Grey and White Gneiss		U	4	J	D	W	SP	F	T	Slight Epidote Joint Filling @ 8.4 metres
To	To 10.60														
From	From														
To	To 0.00														
From	From 0.00														
To	To 0.00														
From	From 0.00														
To	To 0.00														

Full Rock Core Dry

Full Rock Core Wet

-3.1 metre core of Archean Gneiss

TBT ENGINEERING CONSULTING GROUP

Strength (MPa)
VH = Very High = >200
H = High = 50-200
M = Medium = 15-50
L = Low = 4-15
VL = Very Low = 1-4

Discontinuity type
B = Bedding joint
J = Cross Joint
F = Fault
S = Shear Plane

Roughness
RU = Rough undulating
RP = Rough planar
SU = Smooth undulating
SP = Smooth planar
LU = Slicken sided undulating
LP = Slicken sided planar

Orientation
F = Flat (0-20°)
D = Dipping (20-50°)
V = Near Vertical (>50°)

Aperture
O = Open
C = Closed
F = Filled

Weathering
U = Unweathered (No signs)
S = Slightly (Oxidized)
M = Moderately (Discoloured)
H = Highly (Friable)
C = Completely (Soil-like)

Spacing
VW = Very wide = >3m
W = Wide = 1-3m
M = Moderate = 0.3-1m
C = Close = 5-30cm
VC = Very close = <5cm

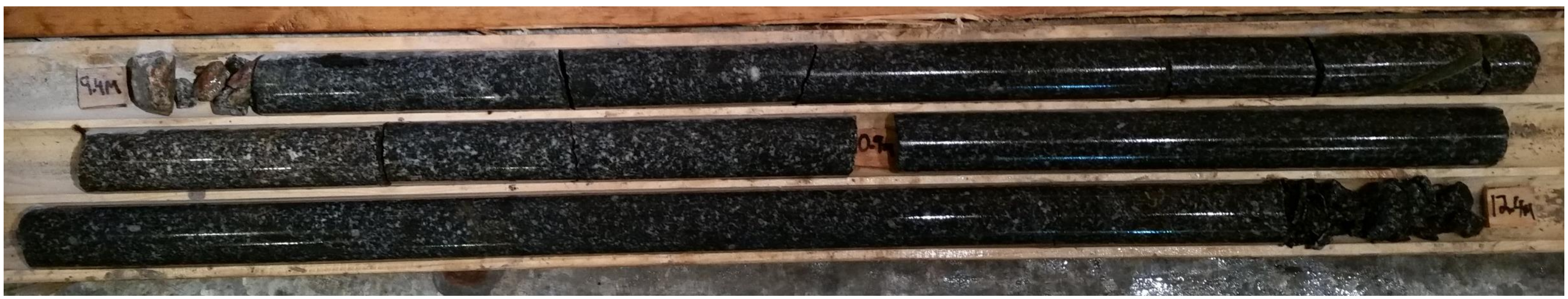
Filling
T = Tight, hard
O = Oxidized
SA = Slightly altered, clay free
S = Sandy, Clay free
Si = Sandy, silty, minor clay
NC = Non-softening clay
SC = Swelling, softening clay
N= No filling

ROCK CORE LOG		
Project #: 16-138	Borehole # 5	Lab# 16-16512
Client: MTO NER	Logger: Larry Wells	
Site: Knife Creek Culvert	Date: October 12, 2016	

DEPTH FROM SURFACE (m)	DEPTH (m)	BOX/RUN	% REC (m)	% RQD (m)	GENERAL DESCRIPTION (Rock type(s), %, colour, texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES # OF SETS	TYPE(S)	Orientation	SPACING	Roughness	APERTURE	FILLING	OCCASIONAL FEATURES
From	From 9.40	1	100%	98%	Grey and White Gneiss		U	3	J	D	W	SP	F	T	Epidote Joint Filling @ 10.3 m
To	To 12.40														
From															
To	To 0.00														
From	From 0.00														
To	To 0.00														
From	From 0.00														
To	To 0.00														

Full Rock Core Dry

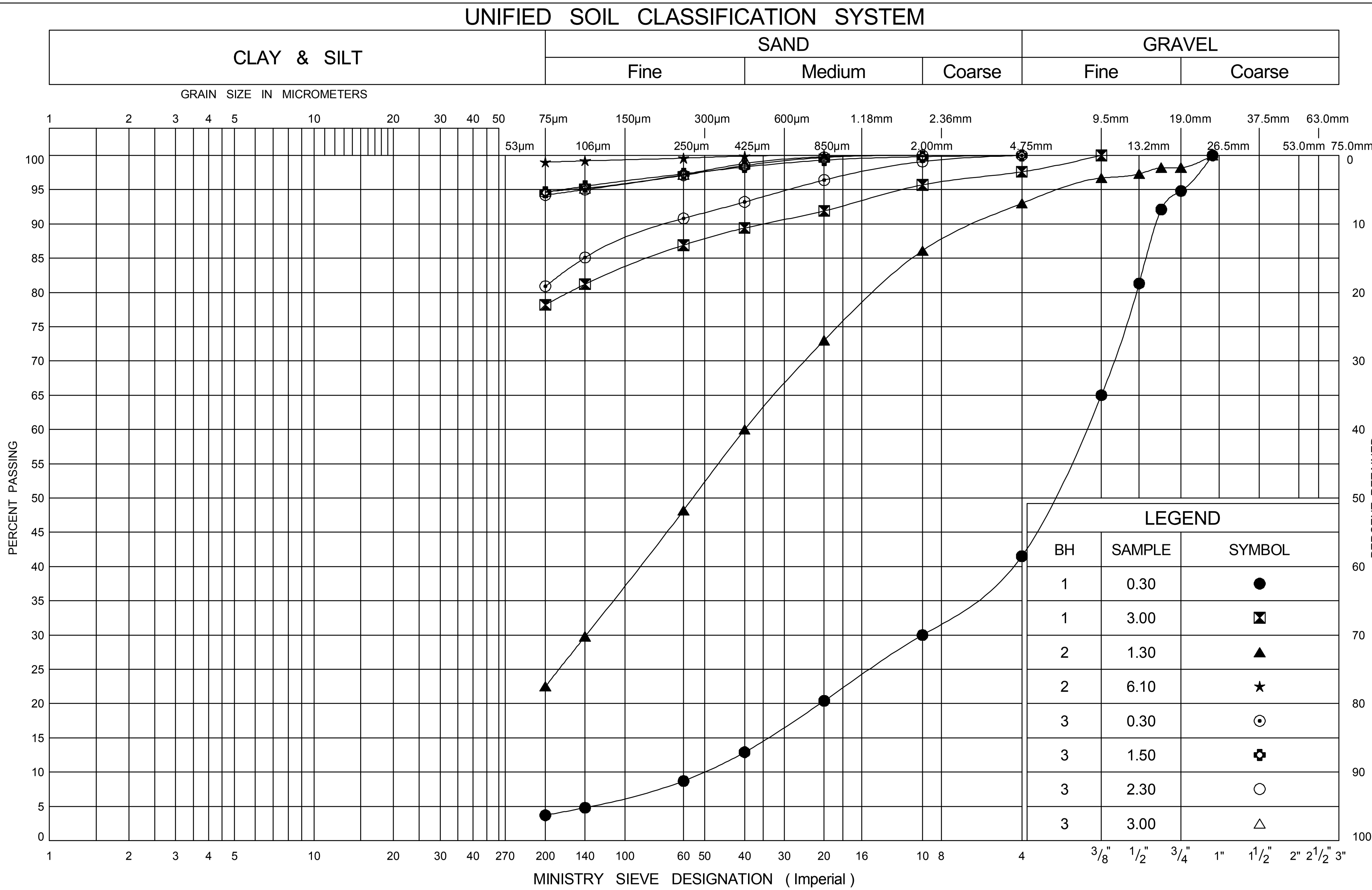
Full Rock Core Wet



-3 metre core of Archean Gneiss

APPENDIX B
Laboratory Test Data

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
1	0.30	●
1	3.00	⧗
2	1.30	▲
2	6.10	★
3	0.30	⊙
3	1.50	✿
3	2.30	○
3	3.00	△

Ministry of Transportation
Ontario

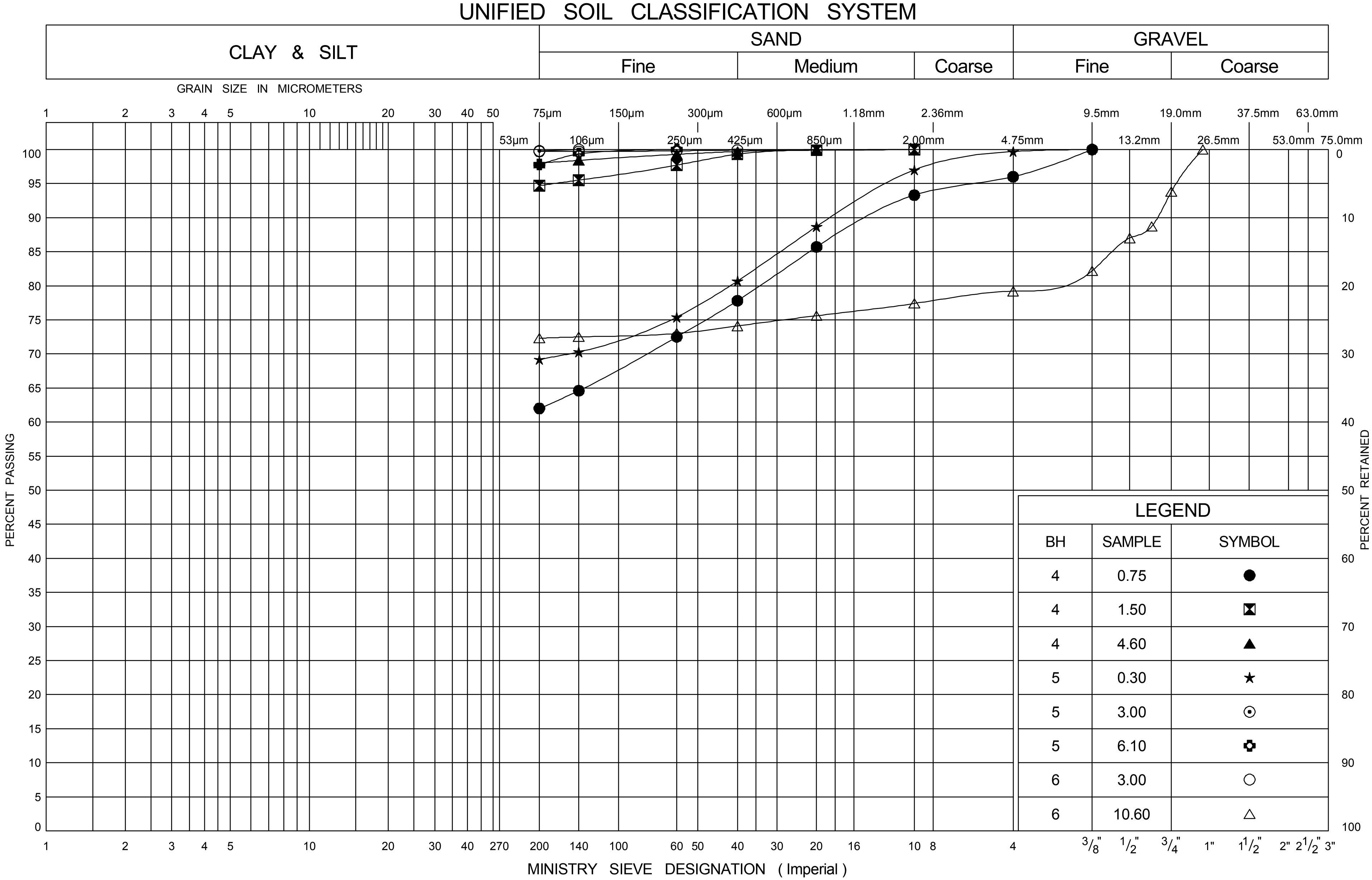
GRAIN SIZE DISTRIBUTION

FIG No

W P

Culvert Investigation

78 12 M



ONTARIO MOT GRAIN SIZE 16-138 MTO - KNIFE CREEK.GPJ ONTARIO MOT.GDT 3/1/17

Ministry of Transportation
Ontario

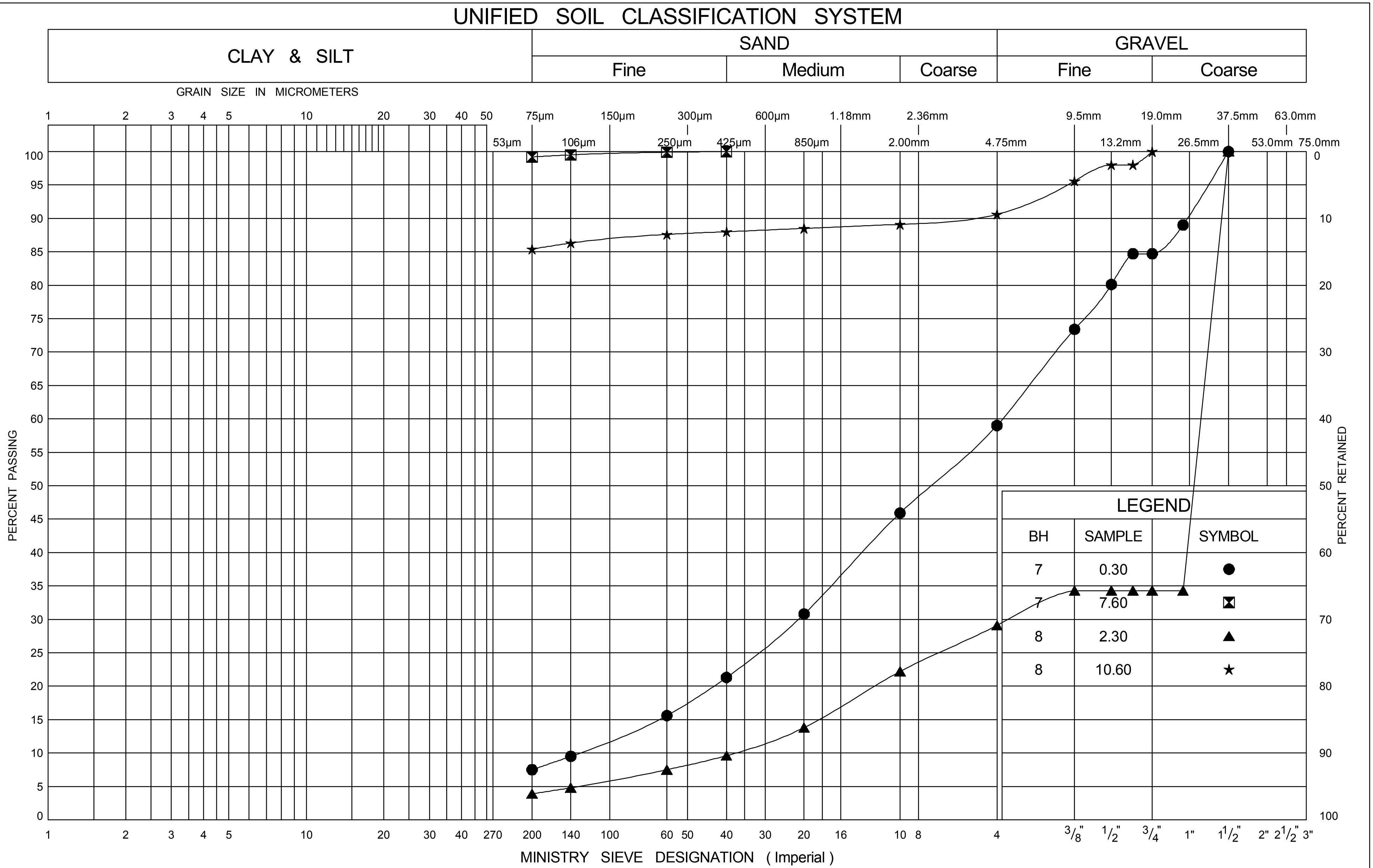
GRAIN SIZE DISTRIBUTION

FIG No

W P

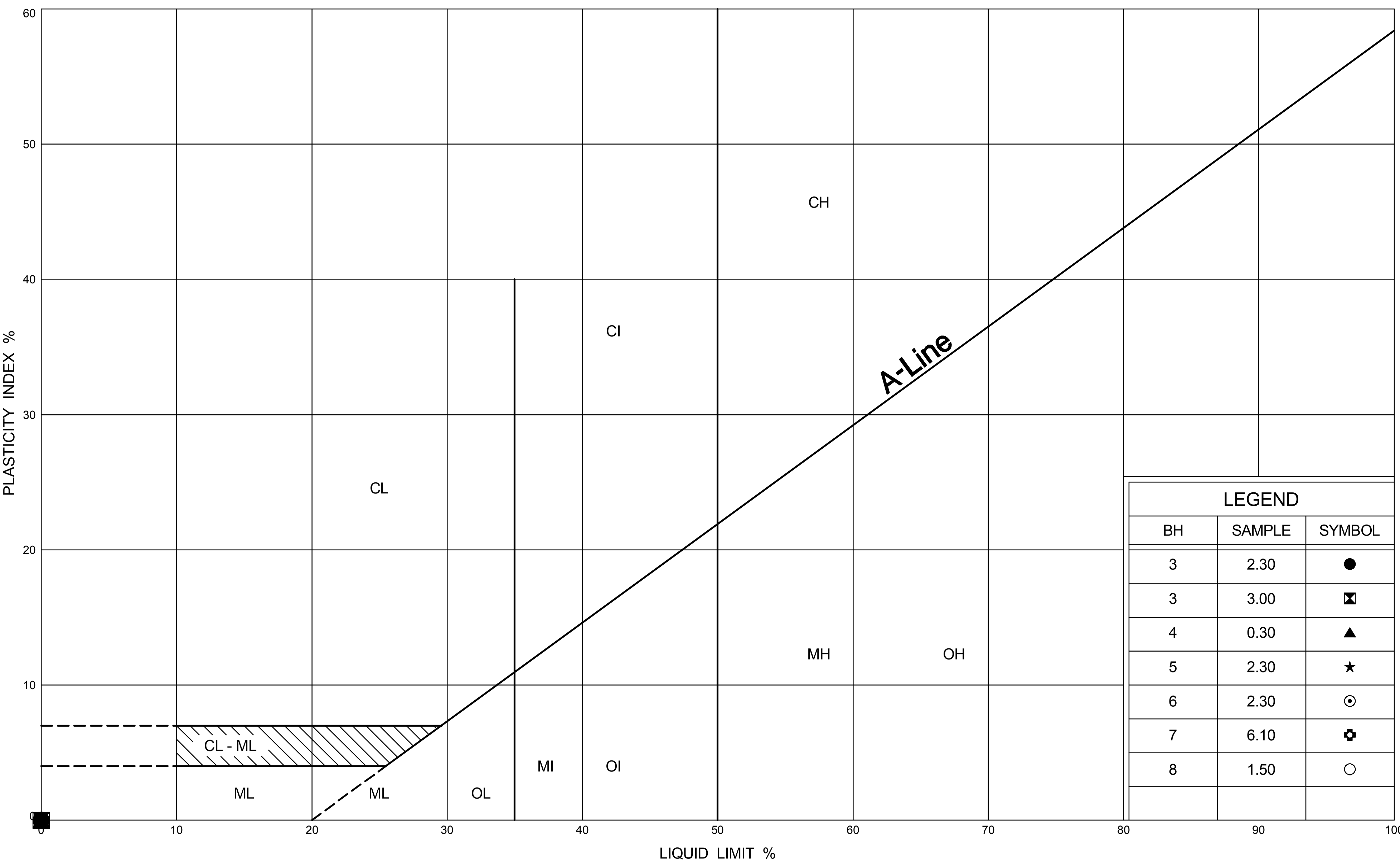
Culvert Investigation

78 12 M



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION	FIG No
		W P
		Culvert Investigation

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
3	2.30	●
3	3.00	⧗
4	0.30	▲
5	2.30	★
6	2.30	⊙
7	6.10	✿
8	1.50	○

Ministry of Transportation
Ontario

PLASTICITY CHART

FIG No
W P
Culvert Investigation

ALS ENVIRONMENTAL ANALYTICAL REPORT

	Sample ID	L1851607-1				
	Description	Soil				
	Sampled Date	20-SEP-16				
	Sampled Time	09:00				
	Client ID	KNIFE CREEK - BH2 SS4				
Grouping	**Analyte**					
SOIL						
Physical Tests	Conductivity (mS/cm)	0.362				
	% Moisture (%)	12.1				
	pH (pH units)	7.78				
	Redox Potential (mV)	123				
	Resistivity (ohm*cm)	2760				
Leachable Anions & Nutrients	Chloride (ppm)	91.8				
	Sulphide (as S) (mg/kg)	<0.20				
Anions and Nutrients	Sulphate (ppm)	42				

APPENDIX C
Borehole Locations and Soil Strata Drawing

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN

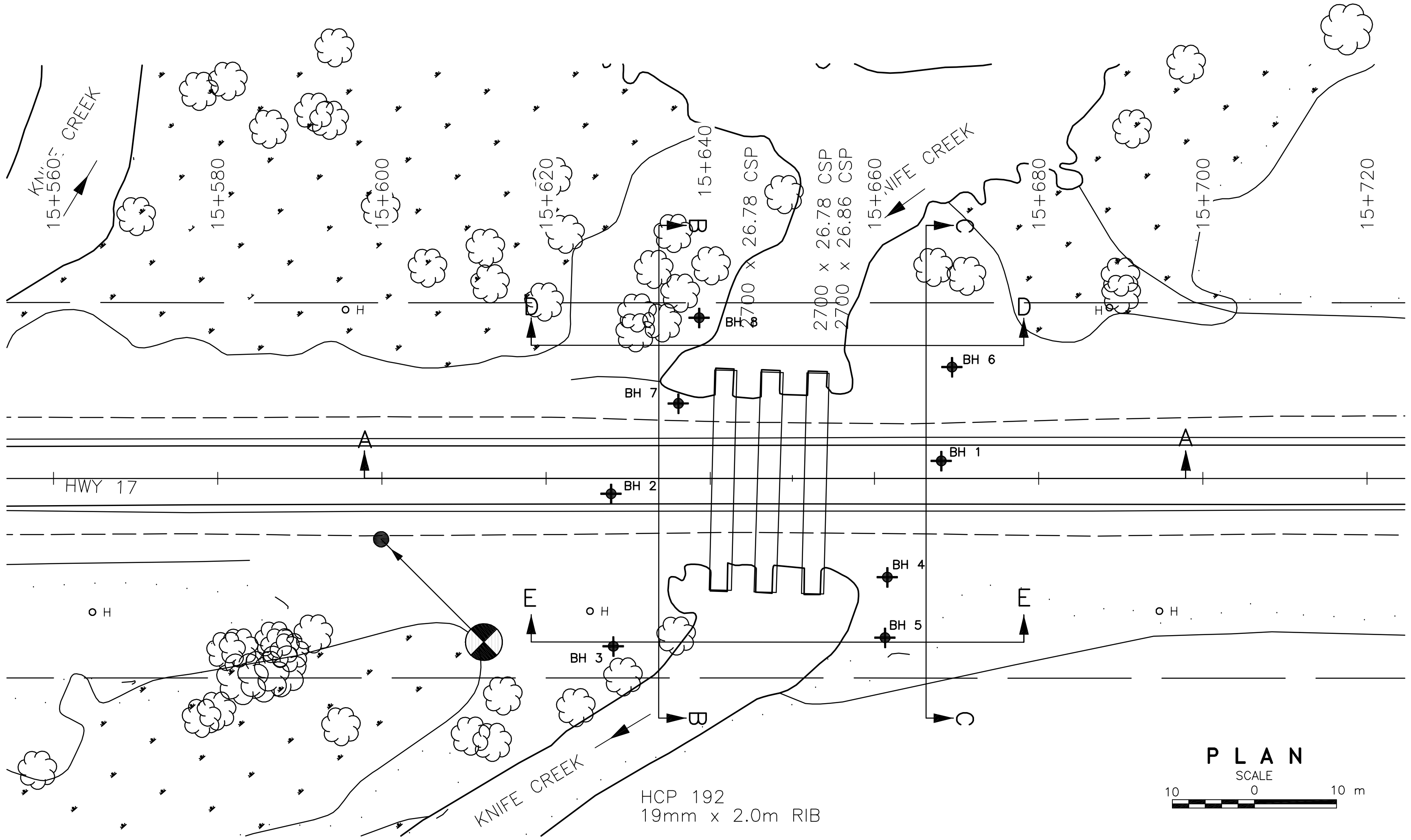
GEOCRES	No. 42C-041
CONT	No. 201x-xx-xxxx
GWP	No. 5119-06-00

HWY 17
CULVERT INVESTIGATION @ KNIFE CREEK
BOREHOLE LOCATIONS AND SOIL STRATA

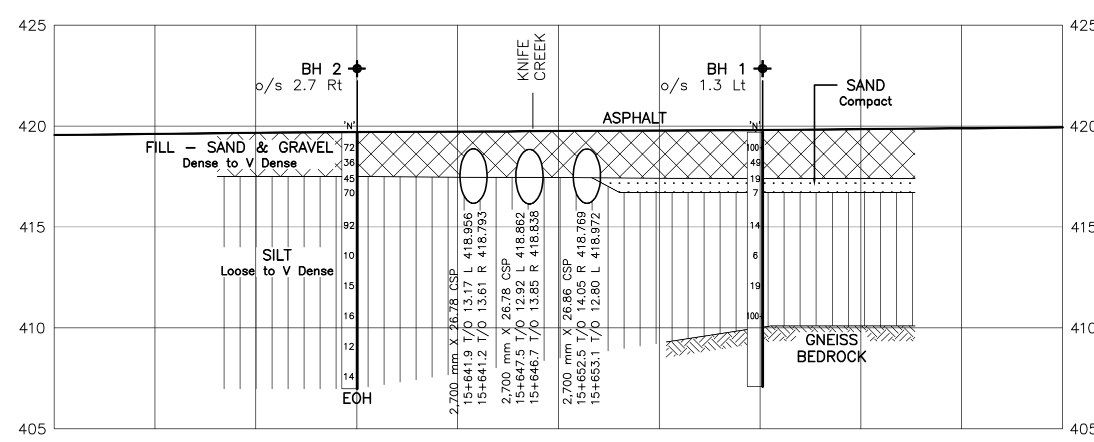
SHEET
1

Ontario
Ministry of Transportation
Northwestern Region
Structural Section

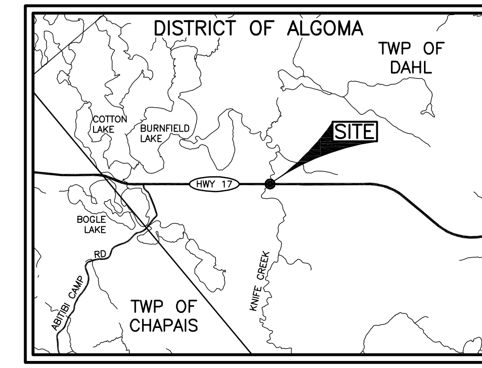
TBT ENGINEERING
CONSULTING GROUP



PLAN
SCALE
10 0 10 m



SECTION A - A
SCALE
HOR 10 0 10 m
VERT 5 0 5 m



KEY PLAN
2.0 km 2.0 km

SOIL STRATA SYMBOLS

- TOPSOIL/ ORGANICS
- SILT
- FILL - SAND OR GRAVEL
- GRAVEL
- SAND
- BEDROCK

LEGEND

✦	Borehole
'N'	Std Pen Test (Blows/0.3m)
▼	Water Level
▽	Water Level on Completion
EOH	End of Borehole
AR	Auger Refusal
○B	Bell Pole
○H	Hydro Pole
● RIB	Round Iron Bar

No	ELEVATION	CO-ORDINATES (MTM) NORTH	EAST
BH 1	419.7	13 5 355 264	228 613
BH 2	419.7	13 5 355 288	228 580
BH 3	418.0	13 5 355 274	228 568
BH 4	418.6	13 5 355 258	228 598
BH 5	418.4	13 5 355 253	228 593
BH 6	418.1	13 5 355 272	228 621
BH 7	418.1	13 5 355 289	228 595
BH 8	417.9	13 5 355 297	228 602

NOTE: ELEVATIONS ARE REFERENCED FROM PLAN PLATE No. 692-17/13-0 WP No. 267-90-00 PROFILE ELEVATION.

—NOTE—
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS

171031	TB	FINAL REVISION 2
170928	TB	UPDATE WITH SURVEY INFORMATION
		DESCRIPTION

CO-ORDINATES Lat 48.331565° Long -85.027971°

DESIGN	CHK	CODE XXXXX-XX	LOAD XX-XXX-XXX	DATE 20161224
DRAWN TB	CHK GM	SITE 38C-154/C		APPENDIX .

MINISTRY OF TRANSPORTATION, ONTARIO

Oct 31, 2017, 12:05pm Login name: tblunden
Drawing Name: I:\Projects\2016\16-138 MTO NER Hwy 17 Post Eng\Foundations\1_Knife Creek\Knife Creek FNDN Final Revision 2.dwg

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

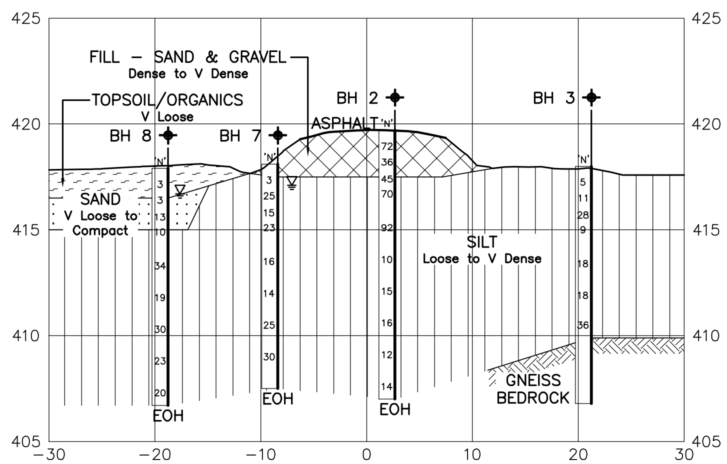
GEOCRES No. 42C-041
CONT No. 201x-xx-xxxx
GWP No. 5119-06-00

HWY 17
CULVERT INVESTIGATION @ KNIFE CREEK
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 2

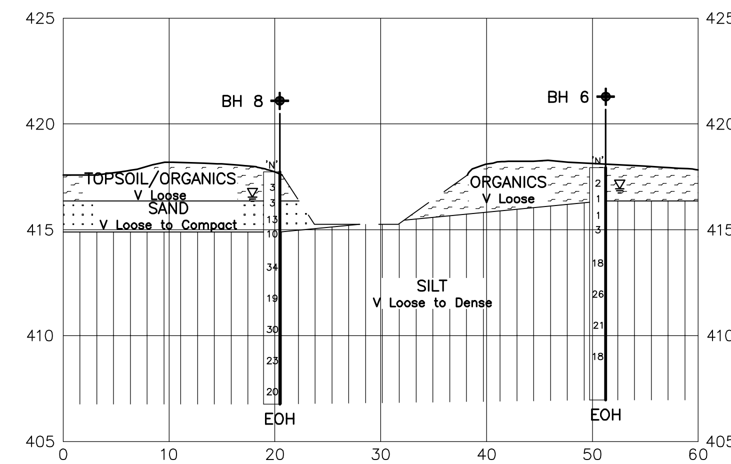
Ontario — Ministry of Transportation, Northwestern Region, Structural Section

TBT ENGINEERING CONSULTING GROUP



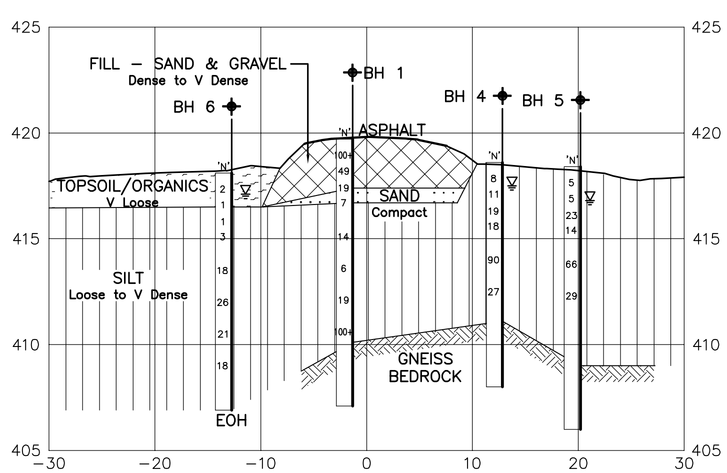
SECTION B – B

SCALE
HOR 10 0 10 m
VERT 5 0 5 m



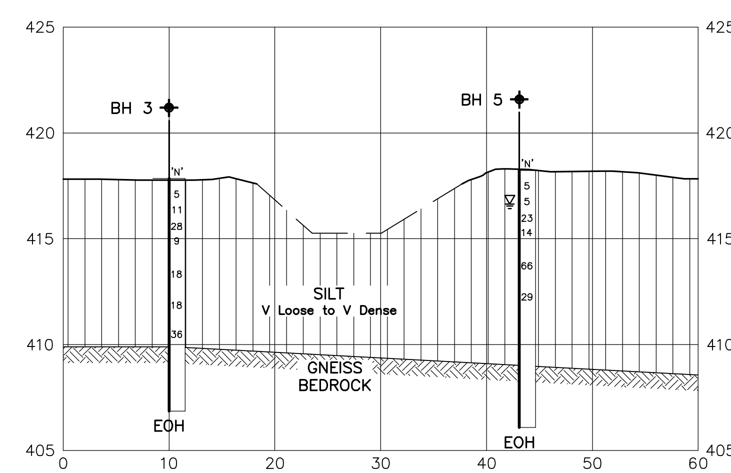
SECTION D – D

SCALE
HOR 10 0 10 m
VERT 5 0 5 m



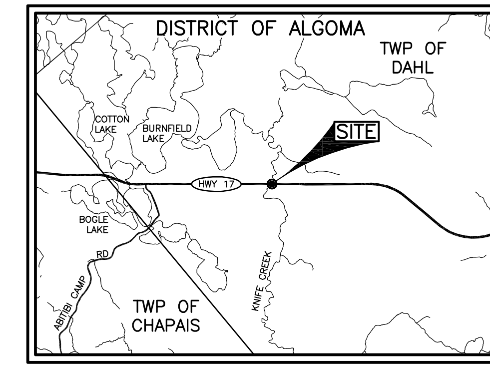
SECTION C – C

SCALE
HOR 10 0 10 m
VERT 5 0 5 m



SECTION E – E

SCALE
HOR 10 0 10 m
VERT 5 0 5 m



KEY PLAN

2.0 km 2.0 km

SOIL STRATA SYMBOLS

- TOPSOIL/ ORGANICS
- SILT
- FILL – SAND OR GRAVEL
- GRAVEL
- SAND
- BEDROCK

LEGEND

✦	Borehole
'N'	Std Pen Test (Blows/0.3m)
▼	Water Level
▽	Water level on completion
EOH	End of Borehole
AR	Auger Refusal

No	ELEVATION	CO-ORDINATES (MTM) NORTH	EAST
BH 1	419.7	13 5 355 264	228 613
BH 2	419.7	13 5 355 288	228 580
BH 3	418.0	13 5 355 274	228 568
BH 4	418.6	13 5 355 258	228 598
BH 5	418.4	13 5 355 253	228 593
BH 6	418.1	13 5 355 272	228 621
BH 7	418.1	13 5 355 289	228 595
BH 8	417.9	13 5 355 297	228 602

NOTE: ELEVATIONS ARE REFERENCED FROM PLAN PLATE No. 692-17/13-0 WP No. 267-90-00 PROFILE ELEVATION.

–NOTE–

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

REVISIONS

		DESCRIPTION
171031	TB	FINAL REVISION 2
170928	TB	UPDATE WITH SURVEY INFORMATION

CO-ORDINATES Lat 48.331565° Long –85.027971°

DESIGN	CHK	CODE XXXXX-XX	LOAD XX-XXX-XXX	DATE 20161224
DRAWN TB	CHK GM	SITE 38C-154/C		APPENDIX .

MINISTRY OF TRANSPORTATION, ONTARIO

Oct 31, 2017, 12:05pm Login name: tblunden
Drawing Name: T:\Projects\2016\16-138 MTO NER Hwy 17 Post Eng\Foundations\1_Knife Creek\Knife Creek FNDN Final Revision 2.dwg