FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN REPORT
SLATE RIVER TRIBUTARY CULVERT REPLACEMENT
HIGHWAY 61
TOWNSHIP OF BLAKE, THUNDER BAY DISTRICT

AGREEMENT NO.: 6013-E-0021
ASSIGNMENT NO.: 9
SITE NO.: 48W-195C
GEOCRES NO. 52A-194
GWP 6305-14-00

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by the Ministry of Transportation (MTO), Geotechnical Section, Northwestern Region to conduct a foundation investigation and produce a preliminary foundation design report for the proposed culvert replacement on Highway 61. This work was carried out under Agreement No.: 6013-E-0021, Geotechnical Retainer, Assignment No. 4 and Assignment No. 9.

This report addresses the field investigation, laboratory test program, factual report on conditions (Part 1) and recommendations for preliminary design and construction for the proposed culvert replacement (Part 2).

2. SITE DESCRIPTION

The site is located on Highway 61, approximately 0.17 km North of Boy Scout Road (latitude 48.2746, longitude -89.4843), LHRS 33540, offset 1.617, Station 23+375, in the Township of Blake, in the District of Thunder Bay.

It is understood that the existing 25.9 m long centerline culvert is a cast-in-place concrete box culvert approximately 6.1 m wide and 1.5 m in height. The existing culvert (Figure 2.3 and 2.4) was originally built in 1899 and inspection by others indicates the culvert is undersized, and is always submerged. The fill thickness above the culvert is approximately 1.0 m and the side slope of the embankment is approximately 2H:1V. The surrounding area is moderately vegetated (Figure 2.1 and 2.2). Photographs were taken by others (Figures 2.1 to 2.4).

Geological information is available from published *Ontario Geological Survey Map #52ASW* by the *Ontario Ministry of Natural Resources* for the Blake Township area. The map indicates that the local area landform is identified as clayey glaciolacustrine plain. The topography in the area is mainly low local relief; plain with dry drainage conditions.

Figure 2.1 Location of existing culvert at Highway 61 (looking North)

Figure 2.2 Location of existing culvert at Highway 61 (looking South)

Figure 2.3 Culvert inlet (looking West)

Figure 2.4 Culvert outlet (looking East)

3. INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out between August 28th and September 5th, 2014 utilizing a CME 750 drill rig equipped for geotechnical drilling and operated by DST. A total of five boreholes were advanced to depths ranging from 3.6 m to 10.8 m. The minimum number and depth of the boreholes was specified by the Ministry of Transportation (MTO).

The borehole locations and stratigraphic sections are shown on the Borehole Location Plan Drawing 1 to 3. Borehole 1 was advanced south of the existing culvert at Station 23+370, 5.1 m right of centreline, and advanced to a depth of 10.8 m below surface. Borehole 2 was advanced North of the existing culvert at Station 23+380, 5.0 m left of centreline, and advanced to a depth of 10.8 m below existing surface. Borehole 3 was advanced North of the existing culvert at Station 23+385, 14.0 m right of centreline, and advanced to a depth of 6.0 m below existing surface. Borehole 4 was advanced at the inlet at Station 23+379, 16.5 m left of centreline, and advanced to a depth of 4.0 m below existing surface. Borehole 5 was advanced at the Inlet at Station 23+372, 16.5 m left of centreline, and advanced to a depth of 3.6 m below existing surface.

The borehole locations are referenced to the MTO Station numbering system as indicated on the drawings provided by MTO. The ground surface elevations at the borehole locations were surveyed by DST personnel and referenced to the existing culvert at Station 23+375. A nail in wooden pole on the south side of the culvert at Station 23+345, 11.0 m Lt was assigned as 0temporary benchmark with elevation of 100.0 m Table 3.1 summarizes the detail of borehole locations and depths.

All boreholes were abandoned using suitable abandonment barrier as described in Ontario Regulation 903 and its amendments. Boreholes were decommissioned by backfilling to the bottom of the road base with cuttings and bentonite chips. From the bottom of the road base, granular materials were replaced to the bottom of the asphalt and the asphalt was sealed with a cold patch.

The fieldwork was supervised on a full-time basis by DST personnel who located the boreholes in the field, performed sampling, in-situ testing and logged the boreholes. Soil samples were obtained from the auger flights and from the split spoon sampler used for the standard penetration test (SPT). The SPT involves driving a 51 mm diameter thick-walled sampler into the soil under the energy of a 63.5 kg weight falling through 760 mm. The number of blows required to drive the sampler 305 mm is known as the standard penetration blow count (N) which provides an indication of the condition or consistency of the soil. The soil samples collected during drilling were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analysis.

Classification and index tests were subsequently performed in the laboratory on samples collected from the boreholes to aid in the selection of engineering properties. Laboratory tests included moisture contents, particle size analyses and Atterberg limits including plastic limit and liquid limit. A total of twenty seven (27) moisture contents, two (2) sieve analyses, one (1) particle size analyses and seven (7) Atterberg limits have been done for this assignment. Laboratory test results are presented in the Boreholes Logs and attached graphical plots in Appendix D (Enclosures).

Table 3.1 Detail of borehole locations

Borehole ID	Station	Elevation (m)	Depth (m)	Offset (m)
BH1	23 + 370	101.4	10.8	5.1 Rt
BH2	23+380	101.2	10.8	5.0 Lt
BH3	23+385	98.6	6.0	14.0 Rt
BH4	23+379	98.7	4.0	16.5.0 Lt
BH 5	23+372	98.7	3.6	16.5 Lt

4. DESCRIPTION OF SUBSURFACE CONDITIONS

The subsurface conditions are presented based on the information obtained during power auger drilling and hand auger drilling.

The generalized stratigraphy of the existing embankment, based on the conditions encountered in Boreholes 1 and 2, consists of asphalt overlying a granular sand layer that is underlain by silty clay.

Table 4.1 Summary of soil strata at the culvert location

Layer	Depth (m)	Elevation (m)	Comments
Asphalt	0.05	101.4 to 101.3 101.1 to 101.0	
Fill- Sand and Crushed Gravel	0.05 to 0.3	101.3 to 101.2 101.0 to 100.8	
Sand	0.3 to 4.6 0.3 to 5.3	101.2 to 96.8 100.8 to 95.8	
Clay-Silty	4.6 to 10.8 5.3 to 10.8	96.8 to 90.6 95.8 to 90.3	

4.1 Asphalt

Asphaltic concrete was encountered at surface in Boreholes 1 and 2 with thickness of 50 mm.

4.2 Topsoil and organics

Topsoil was encountered in Boreholes 3, 4 and 5 at surface with a thickness of approximately 0.1 m (Elev. 98.6 to 98.5 m), 1.5 m (Elev. 98.7 to 97.2 m) and 1.1 m (Elev. 98.7 to 97.6 m) respectively. Standing water was observed in Boreholes 4 and 5.

4.3 Fill – Sand and Crushed Gravel

Sand fill and crushed gravel, trace to some silt was encountered in Boreholes 1 and 2 below the asphalt with a thickness of 0.2 m at depths between 0.1 to 0.3 m (Elev. 101.4 to 101.2m) and depths between 0.1 to 0.3 m (Elev. 101.0 to 100.8 m) respectively. The moisture contents of samples tested range from 4 to 6 %.

4.4 Fill - Sand

Fill Sand with to some gravel and some silt was encountered in the Boreholes 1 and 2 with a thickness of approximately 4.4 m and 5.1 m at depths 0.2 to 4.6 m (Elev. 101.2 to 96.8 m) and 0.2 to 5.3 m (Elev. 100.9 to 95.8 m) respectively.

SPT ‘N’ values vary from 2 to 21, indicating a very loose to compact condition. The moisture contents of the sand material vary from 4 to 17 %. The laboratory test results are summarized in following Tables 4.2

Table 4.2 Summary of particle size analysis

Laboratory Results	
Gravel %	18 to 30
Sand %	55 to 68
Silt %	14 to 15

4.5 Sand

Sand with some gravel was encountered in the Boreholes 3, 4 and 5 with a thickness of approximately 0.5 m, 0.3 m and 0.3 at depths 0.1 to 0.6 m (Elev. 98.5 to 98.0 m), 1.5 to 1.8 m (Elev. 97.2 to 96.9 m) and 1.1 to 1.4 m (Elev. 97.6 to 97.3 m) respectively. Black organics mixed with the sand layer was observed in Borehole 3.

SPT ‘N’ values was found to be 1 in Borehole 3, indicating a very loose condition. The moisture contents of the sand material for borehole 1 was found to be 44.

4.6 Silt-sandy

Sandy silt with some clay was encountered in Borehole 3 at depth of 0.6 m (Elev. 98.0 m) with thickness of 1.7 m. SPT ‘N’ values were found to vary between 1 and 3, indicating a very loose condition. The moisture contents of the tested sample was found to be between 19 to 43. The laboratory test results are summarized in following Tables 4.3

Table 4.3 Summary of particle size analysis-silt

Laboratory Results	
Gravel %	0
Sand %	51
Silt %	31
Clay %	18

4.7 Clay-silty

Silty clay material was encountered in Boreholes 1, 2, 3, 4 and 5 at a depths of 4.6 m (Elev. 96.8

m), 5.3 m (Elev. 95.8 m), 2.3 m (Elev. 96.3 m), 1.8 m (Elev. 96.9 m) and 1.4 m (Elev. 97.3 m) respectively. The thickness of this stratum is not defined as borehole terminus was reached within this stratum. Organics was encountered in Borehole 3 within this stratum.

Atterberg limits tests carried out on samples from Boreholes 1, 2, and 3 indicate that the clay has intermediate to high plasticity with liquid limits ranging from 41 to 70 % and plasticity indexes ranging from 15 to 39 %. The moisture content of the clay ranges from 25 to 72 %. Field vane tests completed in Boreholes 1, 2 and 3 vary between 35 kPa to 90 kPa indicating firm to stiff consistency. The laboratory test results are summarized in following Tables 4.4.

Table 4.4 Summary of Atterberg limits- clay

Laboratory Results – Atterberg Limits	
Liquid Limit %	41 to 70
Plastic Limit %	20 to 44
Plastic Index %	15 to 39

4.8 Groundwater

At the time of the field investigation groundwater was observed in Borehole 1 and Borehole 2 at depth of 2.3 m (Elev. 99.1 m) and 2.2 m (Elev. 98.9 m) respectively. The groundwater levels can be expected to vary with the season and precipitation events.

Table 4.5 Groundwater

Borehole Number	Ground water Depth (m)	Elevation (m)
Borehole 1	2.3	99.1
Borehole 2	2.2	98.9

5. MISCELLANEOUS

Site work was carried out between August 27 and September 5, 2014 utilizing a CME 750 all-terrain drill rig operated by DST personnel. Fieldwork was supervised on a full time basis by Peter Raynak who located the boreholes in the field, performed sampling, in-situ testing and logged the boreholes. Soil samples collected during drilling were identified in the field, placed in labelled containers and transported to DST's laboratory in Thunder Bay for further analysis. Interpretation of the data and preparation of the report was completed by Deep Bansal, P.Eng and reviewed by Prof. Myint Win Bo, P.Eng a designated principal contact for MTO projects.

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PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

6. PROJECT DESCRIPTION

DST Consulting Engineers Inc. (DST) has been retained by the Ministry of Transportation (MTO), Geotechnical Section, Northwestern Region to conduct a foundation investigation and preliminary design report for the proposed culvert replacement on Highway 61. This work was carried out under Agreement No.: 6013-E-0021, Geotechnical Retainer, Assignment No. 4 and Assignment No. 9, Foundation Investigation and Preliminary Design Report of Various Culverts.

It is understood that the existing 25.9 m long centerline culvert is a cast-in-place concrete box culvert approximately 6.1 m wide and approximately 1.5 m in height. The fill thickness above the culvert is approximately 1.0 m and the side slope of the embankment is approximately 2H: 1V. The culvert replacement is recommended to be of a similar box concrete culvert.

The generalized stratigraphy of the existing embankment, based on the conditions encountered in Boreholes 1 and 2, consists of asphalt overlying a granular sand layer that is underlain by silty clay.

This section presents interpretation of the geotechnical data presented in the factual report and provides preliminary geotechnical design recommendations and construction concerns for the proposed culvert replacement

6.1 Replacement Structure

It is of the opinion of DST that the proposed replacement structure at this site should be a cast-in-place or precast concrete box culvert as previously discussed with MTO so as to replace the existing structure with the same type of culvert. However open bottom footing analyses have been provided as a feasible option. A box culvert is the preferred option due to a relatively thick loose fill material at the invert of the culvert. An open bottom footing option will require protection to prevent scouring and undermining along the length of the foundation within the flow path of the culvert under the current conditions. It is understood that open cut excavation will be used to replace the structure. The design of the culvert must be in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC, 2006) and all relevant Ministry of Transportation specification and guidelines.

6.1.1 Earth Excavation

Earth excavation will be required adjacent the existing and replacement structure and may require temporary surface water ditch diversion and temporary support for traffic. This method can more readily accommodate excavation of large boulders, if encountered during excavation. As a minimum, the procedures should be in accordance with OPSS 902 "Construction Specifications for Excavating and Backfilling-Structures". Where temporary protection systems are required they shall be constructed in accordance with OPSS.PROV 539 "Construction Specification for Temporary Protection Systems" and Section 6.1.5 Roadway Protection.

If organic materials are encountered during excavation, the excavations to remove these organics and wood should be completed in accordance with OPSS.PROV 209 "Construction Specification for Embankments Over Swamps and Compressible Soils".

Excavation should be in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA), O.Reg. 213/91. According to O.Reg. 213/91, s.226, the soils in the area of interest classify as Type 3 and Type 4 if located above and below the water table respectively. Type 3 soils generally are stiff to firm and compact to loose or are previously excavated soil, exhibit signs of surface cracking, exhibit signs of seepage, if it is dry, may run easily into a conical pile and have a low degree of internal strength. Type 4 soils generally are soft to very soft and very loose in consistency, very sensitive and upon disturbance are significantly reduced in natural strength, run easily or flow unless it is completely supported before excavation procedure, have almost no internal strength, are wet or muddy and exerts substantial fluid pressure on its supporting system. In accordance with O. Reg. 213/91, s.227 (3), if an excavation contains more than one type of soil, the soil shall be classified with the highest number as described in section 226. These should be assessed and confirmed in the field as construction progresses. Open excavation without shoring could be completed provided that the soils are sloped back sufficiently to maintain sidewall stability and protect workers. As per the OHSA O. Reg 213/91, s 234 it is recommended that the excavation side slopes should not be steeper than 1H: 1V for soils Type 1 to 3 and 3H: 1V for soil Type 4. The stability of the excavation side slopes will be highly dependent on the contractor's methodology and ability to effectively dewater the excavation. Bottom width of excavation should be 4 to 6 m wider than maximum width of proposed replacement culvert.

6.1.2 Preliminary Foundation Design (Concrete Box Culvert)

The culvert should be located approximately at the same elevation and location as the existing culvert. As the proposed culvert is not expected to be heavily loaded, a shallow foundation is considered suitable for this site. The geotechnical resistance was estimated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm. The resistance at ULS was calculated by applying load resistance factor of 0.5 in accordance with the Bridge Design Code (CHBDC) CAN/CSA-S6-06 section 6.6.3.6, Table 6.1. The geotechnical resistance was estimated assuming a strip footing consisting of a width equal to the width of the culvert (6.1 m) and a depth of the culvert base equal to 0 m, which is a temporary condition prior to backfill that will be encountered during construction. Settlement of the structure can be considered negligible due to the marginal expected change in net loading. While ULS is not relevant at final condition due to excessive soil cover SLS is not relevant for temporary condition. Therefore SLS reported here are for final condition. The culvert should be installed to a minimum depth of 2.5 m (Elevation 99.0 m) below top of pavement and bedding material placed on undisturbed native silt or clay soils.

Table 6.1 Concrete Box Culvert - Geotechnical resistances and reactions

Footing Size	Ultimate bearing capacity (kPa)	Resistance at ULS (kPa)	Resistance at SLS (kPa)
B = 6.1 m	180	90	50

6.1.3 Preliminary Foundation Design (Open Footing Culvert)

The culvert will be located at approximately the same vertical and horizontal alignment as the existing structure. As the proposed culvert is not expected to be heavily loaded, a shallow foundation is considered suitable for this site. As the cross sectional area of the existing Concrete culvert will remove the existing soil materials, the overall effect on the culvert foundation soils will be a small decrease in stress at the base of the culvert.

The geotechnical resistance was estimated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm. The resistance at ULS was calculated by applying load resistance factor of 0.5 according to the Bridge Design Code (CHBDC) CAN/CSA-S6-06 section 6.6.3.6, Table 4.1. The geotechnical resistance was estimated assuming a strip footing of various widths with a length equal to 25.9 m.

Table 6.2 Open Footing Culvert Geotechnical resistances and reactions for open footing culverts

Footing Width L=25.9 m	Depth of Soil Cover	Ultimate bearing capacity (kPa)	Factored Resistance at ULS (kPa)	Resistance at SLS (kPa)
B = 1.0 m	0.50	180	90	50
	1.00	300	150	55
	1.50	440	220	60
B = 1.5 m	0.50	200	100	40
	1.00	320	160	45
	1.50	450	225	50
B = 2.0 m	0.50	230	115	35
	1.00	350	175	40
	1.50	480	240	45

Where unsuitable or unstable soils are encountered, the foundation soils must be removed to a firm or hard soils and replaced to the foundation grade with Granular "A" material meeting OPSS.PROV 1010 specifications and compacted to a minimum of 95 % of standard Proctor maximum dry density.

6.1.4 Lateral and Sliding Resistances

The analysis of horizontal and vertical effects of earth loads on the culvert can be performed considering soil parameters given in Table 6.2 and assuming linearly variation of stress change with the depth as described in Section 7.8.5.3.2 in Canadian Highway Bridge Design Code. Temporary bracing and shoring may be designed using the typical soil parameters given in Table 6.2, but the designer/contractor should verify the appropriate soil parameters for the designs of specific bracing and shoring system.

It is recommended that all excavations be either adequately sloped or securely shored and braced to prevent earth caving and to provide a safe and stable work area. The design should incorporate the effects of hydrostatic pressure, traffic surcharge and retained sloping earth conditions in the bracing design.

Table 6.3 Typical soil parameters for earth loads

Soil type	Unit weight (kN/m^3)	Drained Internal friction angle (Deg)	Interface friction angle δ (Deg) Soil-concrete (Soil-Sheet pile)	Intact undrained shear strength (kPa)	Adhesion (kPa)
Granular A[&]	21	35	17 (14)	-	-
Fill Sand	21	35	17 (14)	-	-
Sand	21	35	17 (14)	-	-
Clay - Silty	19	26	17 (11)	39	26
Silt -sandy	19	30	14 (14)	-	-

[&] Please note that parameters of Granular A are dependent on the degree of compaction, mineralogy, angularity of the soil particles and therefore could vary from the listed values.

Table 6.4 Lateral Earth Pressure Coefficients

Earth Pressure Coefficient	Equation*	Granular A[#] Fill-Sand Sand	Clay-silty[#] (Drained condition)	Silt-sandy[#]
Active Earth Pressure (K_a)	$\left(\frac{1-S\quad\emptyset}{1+S\quad\emptyset}\right)$	0.27	0.39	0.33
Passive Earth Pressure(K_p)	$\left(\frac{1+S\quad\emptyset}{1-S\quad\emptyset}\right)$	3.68	2.55	3.0
At rest (K_o)	$(1-S\quad\emptyset)$	0.42	0.56	0.5

* Φ is an angle of internal friction
[#] The earth pressure coefficient provided here are for the normally consolidated soils condition considering fully mobilized condition

For over consolidated (OC) clay the earth pressure coefficient at rest condition should be corrected using a following relationship

$$\mathbf{K_{0\,(OC)} = K_{0(NC)} * (OCR)^{0.5}}$$

Where

$K_{o(oc)}$)= Earth pressure coefficient over consolidated soils

$K_{o(Nc)=}$ Earth pressure coefficient normally consolidated soils

OCR= Over consolidation Ratio

The sliding resistance can we calculated using the following formulae.

$$\mathbf{F_r = W\ (tan¢)}$$

Where

δ = Interface friction angle

W= Total weight of the soil element retained per unit length of the retaining wall

6.1.5 Roadway Protection

Since some temporary roadway protection is required during the structure replacement, installation of a sheet piles system with necessary support may be considered to ensure the stability of the bank and is a feasible option. Alternatively, the use of gabion wall or soldier piles with lagging installed as the excavation progresses may also be considered. Soldier piles, properly designed, will be more capable of accommodating the presence of cobbles if encountered within the embankment fill. The advantages and disadvantages of various options are summarized in Table 7.1. The use of any listed roadway protection option may also require widening of the road platform and/or construction of a temporary embankment to provide sufficient space for traffic to safely traverse during staging. The design of roadway protection may be performed using the typical soil parameters given in Table 6.2, but the designer/contractor should verify the appropriate soil parameters for the designs. As the potential of encountering cobbles exists, the contractor should be prepared to handle this with the selection of adequate driving or vibratory equipment as well as steel thickness.

The construction methodology must be in accordance with all applicable standards and regulations related to the method proposed. The contractor's method and equipment must be suitable for the site conditions and materials used.

6.1.6 Bedding

The foundation soils, clay and silts in particular, will be very susceptible to disturbance and weakening as a result of traffic, standing water and frost. Any foundation soils that could be disturbed shall be protected and therefore use of working mat during construction is recommended. The bottom of the excavation on which the culvert or granular pad is to rest shall not be disturbed. The bedding placement should commence immediately after the final removal of material to the foundation level has been completed. The bedding for the structure should be designed in accordance with Section 7.8 of the CHBDC.

The bedding shall be a minimum of 0.5 m thick and extend to a minimum width (half of the width of culvert) beyond all sides of the culvert. The bedding material should consist of "Granular A" as per Soil Group I in accordance with Table 7.4 of the Canadian Highway Bridge Design Code. The "Granular A" shall be in accordance to OPSS.PROV 1010. The "Granular A" should be placed in layers not exceeding 200 mm in thickness, loose measurement, and each layer compacted to a minimum of 95 % of standard Proctor maximum dry density in accordance with OPSS.PROV 501 "Construction Specification for Compacting". The middle one-third of the culvert width of the top bedding layer, having minimum thickness of 75 mm, shall be loosely placed and uncompacted.

If construction is performed without dewatering bedding material should consist of 19 mm Type I or II clear stone as defined in OPSS.PROV 1004.05.02. Since fine materials are present beneath the clear stone a non-woven geotextile (OPSS 1860.07.05.01 Class II) with the filtration opening size (FOS) less than 135 µm will be required for separation. No compaction is required of the clear stone.

6.1.7 Sidefill and Overfill

The material used for culvert sidefill should not contain debris, organic matter, frozen materials, or large stones of a diameter greater than one-half the thickness of the compacted layers being placed or 100 mm, whichever is smaller. Soils shall be deposited uniformly on each side of the structure in order to prevent lateral displacement. The minimum width of the sidefill should be at least half of the culvert width in each side. The sidefill should consist of Granular A" and compacted to 95% of standard Proctor maximum dry density.

Overfill should consist of "Granular A" and should be compacted to not greater than the compaction or equivalent stiffness of soils in the sidefill zone and bedding. Due to presence of native silt soils underneath the sand fill, the backfill materials should be separated from the adjacent soil with a non-woven Class II geotextile, with a filtration opening size of between 50 to 100 µm, as specified in OPSS 1860 "Material Specifications for Geotextiles".

When the concrete culvert is installed on the undisturbed original ground and fill material is placed around and over the culvert, relative settlements between the fill adjacent to the sides of the culvert and the fill directly over the culvert generates downward frictional forces on the culvert, also effecting a load transfer. This vertical load on the culvert can be determined by multiplying the weight of earth over the top of the box section by the vertical arching factor, λv. Vertical arching factors for Type B1 and B2 box culverts in standard installations can be considered 1.20 and 1.35 respectively as indicated in Section 7.8.4.2.3 of the CHBDC.

$q = \gamma\, h\, b\, \lambda_v$, where

q = vertical load on the culvert

γ = unit weight of soil

h = thickness of soil above the culvert

b = width of the culvert, and

λ_v = vertical arching factor

However, due to the marginal change in net loading above and directly adjacent the culvert replacement, settlements should be considered to be occurring under a recompression condition. Therefore, relative settlements between the fill adjacent the sides of the culvert and the fill directly over the culvert can be considered negligible which results in no or little downdrag force

6.1.8 Dewatering

During construction in order to prevent back up of water from upstream and downstream, a dyke made of sand bags has sometimes been used as a hydraulic barrier. However, a sheet pile vertical cut-off wall will provide better control of both surface and groundwater. A suitable sump and pump system, possibly supported by an efficient wellpoint system, will be required to dewater and stabilize the excavation. A well designed well-point system with a suitable diameter of well point at an appropriate spacing will perform better for working under dry condition and to prevent disturbance of the excavation base through hydraulic heave. It should be noted that depending on the season, depth of excavation and amount of water flow through the creek may vary. The contractor should be prepared to tackle this situation. The contractor should be alerted of the high water table and surface water, for example through a non-standard special provision (NSSP).

A continuous dewatering operation must be provided to keep the excavation stable and free of water. The excavation must be monitored daily throughout the duration of excavation until the completion of backfilling to confirm this. The dewatering system must be maintained and the surrounding area monitored for impacts to items such as, but not limited to, settlement and groundwater usage. The control of water from the dewatering operation should be accordance with OPSS 518 “Construction Specification for Control of Water from Dewatering Operations”.

6.1.9 Erosion Control

Erosion control is essential at inlet and outlet for the successful performance of a culvert. Generally, rip-rap is used to avoid the erosion at inlet and outlet of the culvert. The rip-rap slows down the flow close to the channel bed and prevents culvert failure by the undermining.

To prevent erosion of the surrounding soils at the inlet, rip-rap Treatment shall be applied accordance with OPSD 810.020 "Rip-Rap Treatment for Ditch Inlets" and OPSS 511 "Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting".

The outlet shall be rip-rapped to prevent erosion of the surrounding soils accordance with OPSD 810.010 "Rip-Rap treatment for Sewer and Culvert Outlets" and OPSS 511 "Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting".

To prevent undermining of the bedding, cutoff walls shall be installed along the entrance and exit end bottom sides of culvert. Cutoff wall should be designed based on velocity of the water flow and the type of soil underneath.

The temporary erosion and sedimentation measures during the construction of culvert shall be controlled as described in OPSS 805 "Construction Specification for Temporary Erosion and Sedimentation Control Measures".

6.1.10 Frost Protection

In accordance with OPSD 3090.100 "Foundation Frost Depths for Northern Ontario", the frost penetration at this location is about 2.2 m. The frost susceptible soils shall not be used adjacent to the culvert wall within the depth of frost penetration from the road surface. The soils under the culvert are highly frost susceptible (capable of forming thick ice lenses with the associated pressures and heave).

During winter season, ice may form inside the culvert and a low flow rate may assist the ice formation. It is expected that ice may extend to the culvert invert and frost could therefore extend into the soils below the culverts, possibly as deep as 2.2 m. The frost heave may generate additional stresses on the culvert foundation and walls.

Three design approaches are commonly applied; designing the culvert with enough strength and rigidity to tolerate these pressures (recognizing that the maximum differential pressures and movements as a result of frost lensing cannot be accurately quantified); removing the frost susceptible soils within the frost zone; or providing adequate insulation to reduce frost penetration. As the frost penetration is extended below the invert level of the culvert, the frost protection should be in accordance with OPSD 803.010, 803.030 and 803.031 "Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0m", "Frost Treatment - Pipe Culverts, Frost Penetration Line Below Bedding Grade" and "Frost Treatment - Pipe Culverts, Frost Penetration Line Between Top of Pipe and Bedding Grade".

If sub-excavation for frost effects is carried out in the dry (with adequate dewatering controls), the material can be replaced with Granular B Type 1 material compacted to 95% of standard proctor maximum dry density. If the excavation is in the wet (water is maintained at or above adjacent groundwater table) then the material should be rockfill or clear stone surrounded by geotextile, without the need for compaction. Depending on the structural design of the culvert, partial sub-excavation (less than 2.2 m) may also be considered to reduce differential stresses associated with frost; however the exact pressures and movements cannot be accurately quantified.

Acceptable insulation to prevent frost penetration would be 125 mm Dow Styrofoam Highload 40 Insulation or an equivalent material with a compressive strength of approximately 275 kPa or greater. For a region that has a freezing index greater than 1500 Celsius Degree-Days it is recommended that the insulation be placed beneath the structure and extend 2.44 m from the concrete face of the buried structure.

6.1.11 Embankment Foreslopes

Existing culvert foreslopes are approximately 2H: 1V on both sides of embankment. The foreslopes should be reinstated with a slope not steeper than 2H: 1V if being constructed with granular materials. The foreslopes should be reinstated with a slope not steeper than 1.5H: 1V if being constructed with rock fill. The minimum thickness of rock fill must be greater than 2 m to achieve an adequate FOS for the reinstated rock fill embankment.

6.1.12 Construction Concerns

The main construction issues that need to be addressed for this site are removal of cover/embankment materials, staged removal of the existing culvert, provisions required for temporary roadway protection, diversion of the channel, excavation below the water table and reinstatement of the embankment fill. These items are important for the successful installation of the new culvert.

A Quality Verification Engineer shall be required to inspect the condition of the foundation and surrounding soils before installation of fill materials and ensure the width of excavation and excavation slope walls are suitable, and ensure compliance with materials placed and compaction methods as well as a sheet pile installation.

7. CLOSURE

Based on the information collected from field investigation, parameters interpreted from laboratory test results, groundwater monitoring data and information provided by the client, it is understood that the existing culvert is intended to be replaced with a similar box concrete culvert. Table 7.1 below summarizes the advantages and disadvantages of the use of sheet piles, soldier pile and gabions wall for roadway protection. Since it is a temporary roadway protection, the sheet pile system with necessary support is considered to be a recommended option however design of roadway protection is responsibility of the contractor as per the contract drawings.

Table 7.1 Advantages and disadvantages of using sheet pile versus soldier pile roadway protection

Roadway Protection Option	Advantages	Disadvantages
Sheet Pile	• Relatively non permeable • Increased erosion control capacity	• Lightweight material may encounter difficult driving through cobbles • Higher installation cost • Specialized construction and design required
Soldier Pile	• Heavier gauge materials may be better to be able to accommodate presence of cobbles • Lower cost	• Permeable • Potential for erosion of retained materials • Longer installation time
Gabion Wall	• Ease of Installation • Lower cost • Presence of cobbles and boulders is not an issue	• Permeable • Potential for erosion of retained materials

8. REFERENCES

Canadian Foundation Engineering Manual. 2006. Fourth Edition, Canadian Geotechnical Society.

Canadian Highway Bridge Design Code. 2006, CAN/CSA-S6-06, A National Standard of Canada, Canadian standards Association.

Municipal and Provincial Common, Volume 1 - General & Construction Specifications, "*Ontario Provincial Standard for Roads & Public Work*s" Spec No. *OPSS* 511, 517, 518, 805, 902.

Municipal and Provincial Common, Volume 2 - Material Specifications, "*Ontario Provincial Standard for Roads & Public Work*s" Spec No. *OPSS* 1860.

Municipal and Provincial Common, Volume 3 - Drawings for Roads, Barriers, Drainage, Sanitary Sewers, Watermains and Structures, "*Ontario Provincial Standard for Roads & Public Work*s" Spec No. *OPSD* 203.040, 803.010, 803.030, 803.031, 810.010, 810.020, 3090.100.

Provincial-Orientated, Volume 5 - MTO General Conditions of Contract and General & Construction Specifications, "*Ontario Provincial Standard for Roads & Public Work*s" Spec No. *OPSS.PROV* 209, 501, 510, 539.

Provincial-Orientated, Volume 6 - Material Specifications, "*Ontario Provincial Standard for Roads & Public Work*s" Spec No. *OPSS.PROV* 1004, 1010.

The Surveys and Design Office, Highway Engineering Division, Ministry of Transportation, 1990, Pavement Design and Rehabilitation Manual.

9. LIMITATIONS OF REPORT

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix 'A', and this forms an integral part of this report.

For DST CONSULTING ENGINEERS INC.

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Reviewed by:

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C.Geol, C. Eng, Eur Geol, Eur Eng
Senior Vice President / Senior Principal

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S [illegible] C [illegible] S [illegible] L [illegible] S [illegible] C

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The data, conclusions and recommendations which are presented in this report and the quality thereof are based on a scope of work authorized by the Client. Note that no sampling program, no matter how comprehensive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that [illegible]S[illegible] Consulting engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the test holes. The borings and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stages to verify that the design is consistent with our recommendations and that assumptions made in our analysis are valid.

Unless otherwise noted, the information contained herein in no way reflects on the environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work.

Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and [illegible]S[illegible] Consulting engineers cannot warrant their accuracy. Similarly, [illegible]S[illegible] cannot warrant the accuracy of information supplied by the client.

Appendix B

DESCRIPTION OF TERMS

DST consulting engineers

EXPLANATION OF TERMS USED IN REPORT

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

DYNAMIC CONE PENETRATION TEST (DCPT): CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51 mm 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 0.3 m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS

TEXTURAL CLASSIFICATION OF SOILS

BOULDERS	COBBLES	GRAVEL	SAND	SILT	CLAY
GREATER THAN 200 mm	75 TO 200 mm	4.75 TO 75 mm	0.075 TO 4.75 mm	0.002 TO 0.075 mm	LESS THAN 0.002 mm

COARSE GRAIN SOIL DESCRIPTION (50% GREATER THAN 0.075 mm)

TERMINOLOGY	TRACE OR OCCASIONAL	SOME	WITH	ADJECTIVE (e.g. SILTY OR SANDY)	AND (e.g. SAND AND SILT)
	LESS THAN 10%	10 TO 20%	20 TO 30%	30 TO 40%	40 TO 60%

CONSISTENCY*: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (C_U) AND SPT 'N' VALUES AS FOLLOWS

C_U (kPa)	0 – 12	12 – 25	25 – 50	50 - 100	100 - 200	> 200
N (BLOWS / 0.3 m)	<2	2 - 4	4 - 8	8 - 15	15 - 30	>30
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

N (BLOWS / 0.3 m)	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, 100 mm+ IN LENGTH EXPRESSED AS A PERCENTAGE 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

LEGEND OF RECORDS FOR BOREHOLES: SYMBOLS AND ABBREVATIONS FOR SAMPLE TYPE

SS	SPLIT SPOON SAMPLE	WS	WASH SAMPLE
TW	THIN WALL SHELBY TUBE SAMPLE	AS	AUGER (GRAB) SAMPLE
PH	SAMPLER ADVANCED BY HYDRAULIC PRESSURE	TP	THIN WALL PISTON SAMPLE
WH	SAMPLER ADVANCED BY SELF STATIC WEIGHT	PM	SAMPLER ADVANCED BY MANUAL PRESSURE
SC	SOIL CORE	RC	ROCK CORE
▼	WATER LEVEL	$SENSITIVITY = \frac{UNDISTURBED\ SHEAR\ STRENGTH}{REMOLDED\ SHEAR\ STRENGTH}$	

*HIERARCHY OF SOIL STRENGTH PREDICTION: **1)** LABORATORY TRIAXIAL TESTING. **2)** FIELD INSITU VANE TESTING. **3)** LABORATORY VANE TESTING. **4)** SPT VALUES. **5)** POCKET PENETROMETER.

Appendix C

DRAWINGS

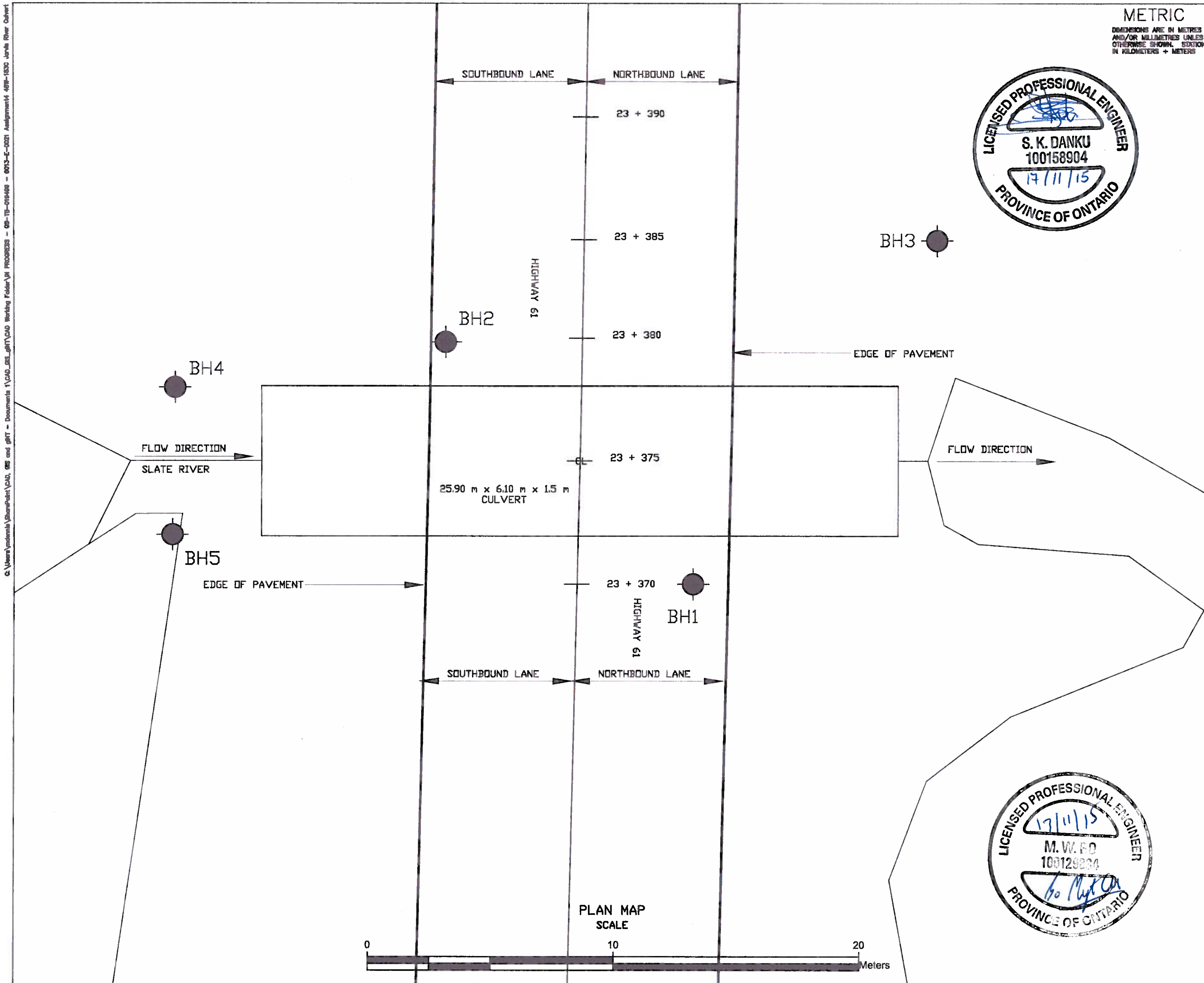
METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETERS + METERS

CONT No
WP No 6305–14–00
SITE No 48W–195/C
GEOCRES No 52A–194

CULVERT REPLACEMENT
SLATE RIVER CULVERT
STA 23+370 TO STA 23+385
Survey______ Revised______

SHEET



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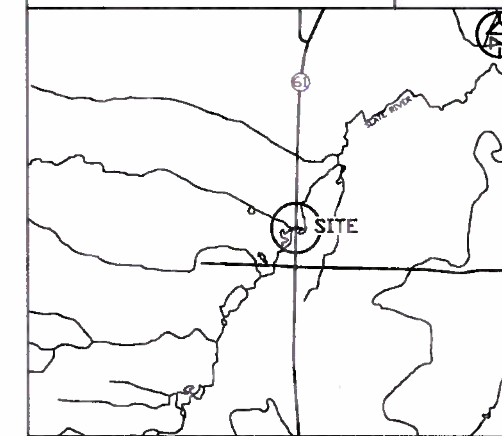

KEY MAP

LEGEND

Borehole

No.	Elevation	Northing	Easting	Station	Offset
BH1	101.4	5349847 m N	315704 m E	23+370	5.1 m RT
BH2	101.2	5349880 m N	315694 m E	23+380	5.0 m LT
BH3	98.6	5349862 m N	315714 m E	23+385	14.0 m RT
BH4	98.7	5349853 m N	315683 m E	23+379	16.5 m LT
BH5	-	-	-	23+372	16.5 m LT

NOTE:
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

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DRAWING 1

METRIC

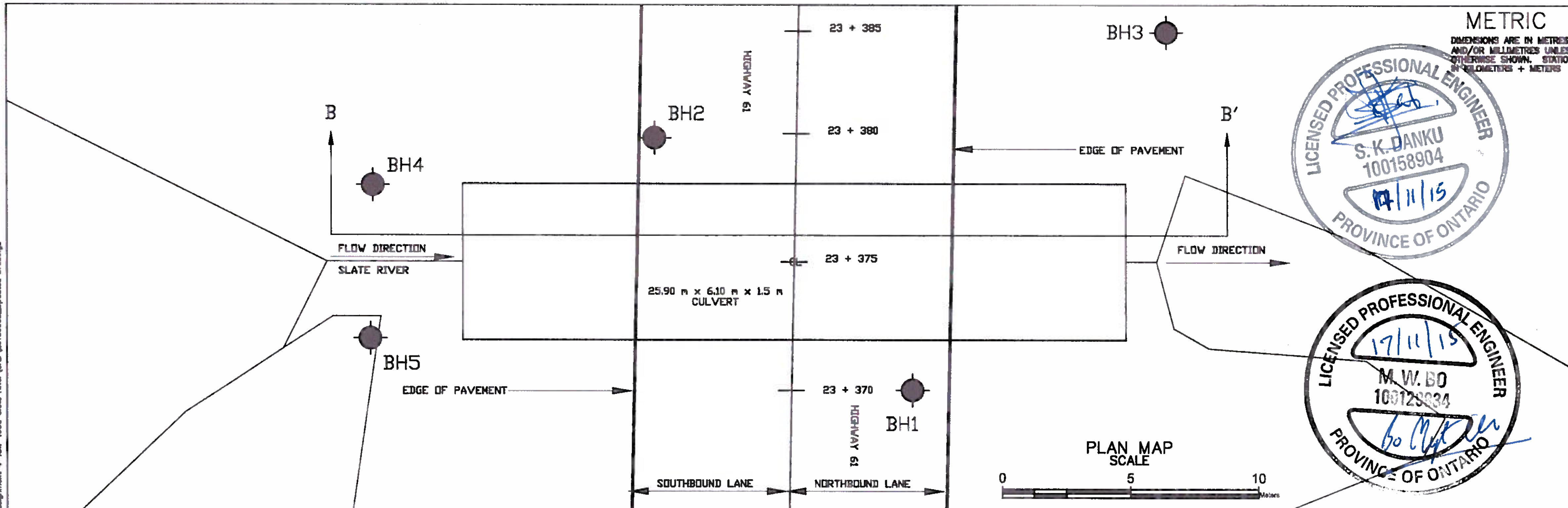
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETERS + METERS

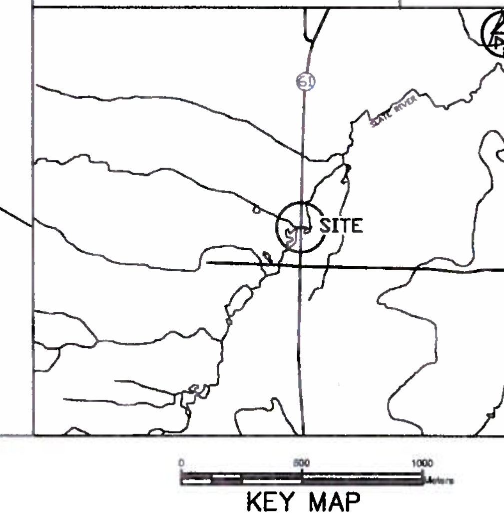
CONT	No
WP	No 6305–14–00
SITE	No 48W–195/C
GEOCRES	No 52A–194

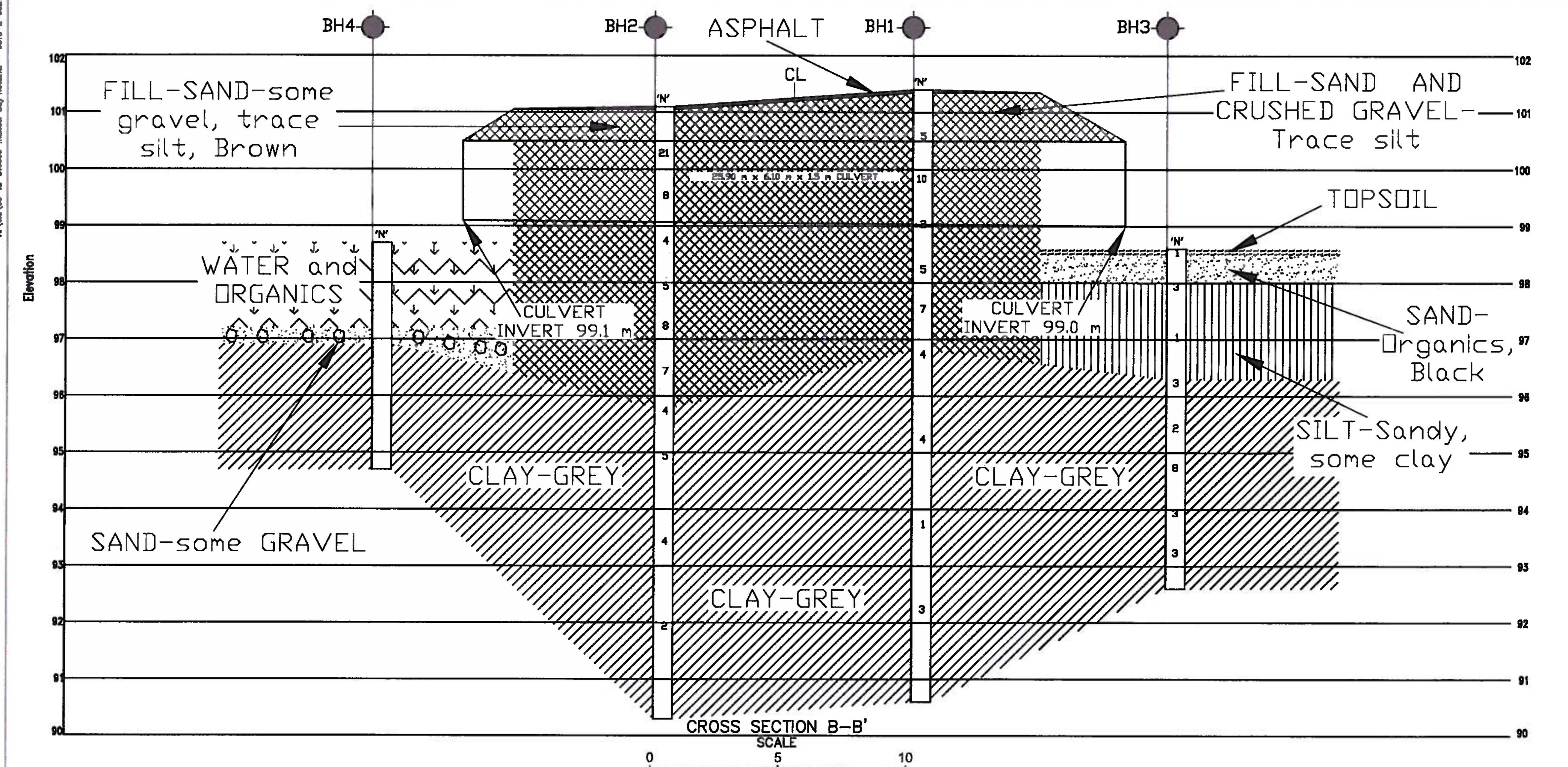
CULVERT REPLACEMENT
SLATE RIVER CULVERT
STA 23+370 TO STA 23+385

Survey______ Revised______

SHEET







LEGEND

- Borehole
- 'N' Blows/0.3m (Std. Pen Test, 475 J/Blow)

Fill, Organics, Topsoil, Till, Bedrock, Sand, Silt, Clay, Sand & Gravel, Boulders

No.	Elevation	Northing	Easting	Station	Offset
BH1	101.4	5349847 m N	315704 m E	23+370	5.1 m RT
BH2	101.2	5349880 m N	315694 m E	23+380	5.0 m LT
BH3	98.6	5349882 m N	315714 m E	23+385	14.0 m RT
BH4	98.7	5349853 m N	315683 m E	23+379	16.5 m LT
BH5	-	-	-	23+372	16.5 m LT

NOTE: The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed by interpolation and may not represent actual conditions.

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DRAWING 2

Appendix D

ENCLOSURES

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BH1

METRIC

W.P. 6013-E-0021 LOCATION Slate River Tributary Culvert STA 23+370 RT 5.1 m ORIGINATED BY PR

DIST Thunder Bay HWY 61 BOREHOLE TYPE Hollow Stem Auger 80 mm COMPILED BY DB

DATUM Local DATE 2014 08 27 CHECKED BY DM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
101.4	GROUND SURFACE					
101.4 0.1	ASPHALT					
101.2 0.3	FILL-SAND & CRUSHED GRAVEL-Trace silt	AS1	AS			
	FILL-SAND-some gravel, trace silt, BROWN□L□□S□ t□C□M□AC□	SS2	SS	5		
		SS3	SS	10	▽	
		SS4	SS	2		
		SS5	SS	5		18 68 (14)
		SS6	SS	7		
96.8 4.6	CLAY-Silty, GREY□S□□□	SS7	SS	4		
		SS8	SS	4		
		SS9	SS	1		
		SS10	SS	3		
90.6 10.8	END OF BOREHOLE					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (101, 100, 99, 98, 97, 96, 95, 94, 93, 92, 91); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL × LAB VANE 50 100 150 200 250; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

Field vane readings: $+^6$ at approx. elev. 95.8; + at approx. elev. 94.7; $+^6$ at approx. elev. 92.8; $+^6$ at approx. elev. 90.8

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

ON_MOT-HIGH VANES GS-TB-019500 SLATE RIVER.GPJ DST_MIN.GDT 11/24/14

ENCLOSURE 1

RECORD OF BOREHOLE No BH2

METRIC

W.P.	6013-E-0021	LOCATION	Slate River Tributary Culvert STA 23+380 LT 5.0 m	ORIGINATED BY	PR
DIST	Thunder Bay HWY 61	BOREHOLE TYPE	Hollow Stem Auger 80 mm	COMPILED BY	DB
DATUM	Local	DATE	2014 08 27	CHECKED BY	DM

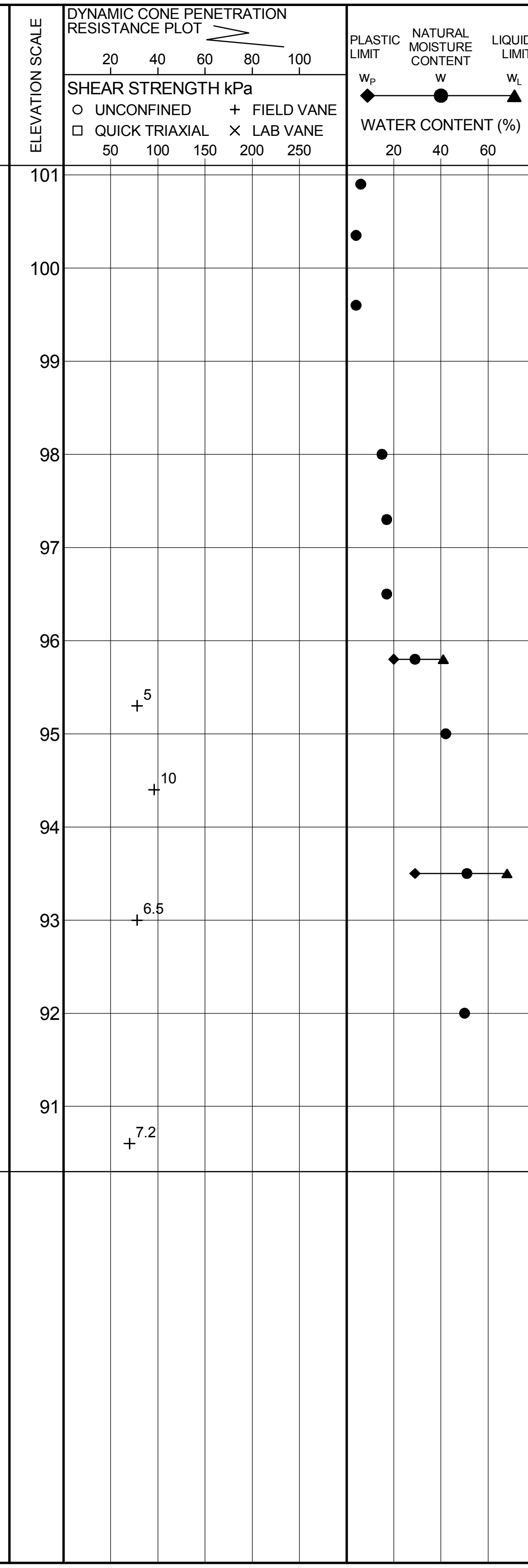
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
101.1	GROUND SURFACE
101.0 0.1	ASPHALT
100.8 0.3	FILL-SAND & CRUSHED GRAVEL-Trace silt
	FILL-SAND-some gravel, trace silt, BROWN□L□□S□ □□ C□M□AC□
95.8 5.3	CLAY-Silty, GREY□S□□□
90.3 10.8	END OF BOREHOLE

SAMPLES

NUMBER	TYPE	"N" VALUES
AS1	AS	
SS2	SS	21
SS3	SS	8
SS4	SS	4
SS5	SS	5
SS6	SS	8
SS7	SS	7
SS8	SS	4
SS9	SS	5
SS10	SS	4
SS11	SS	2

GROUND WATER CONDITIONS: ∇ at elevation approx. 99



UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
30	55	(15)	

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

RECORD OF BOREHOLE No BH3

1 OF 1 **METRIC**

W.P. 6013-E-0021 LOCATION Slate River Tributary Culvert STA 23+385 RT 14.0 m ORIGINATED BY PR

DIST Thunder Bay HWY 61 BOREHOLE TYPE Hollow Stem Auger 80 mm COMPILED BY DB

DATUM Local DATE 2014 09 05 CHECKED BY DM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
98.6	GROUND SURFACE					
98.5 / 0.1	TOPSOIL					
	SAND-organics, BLACK□□□□Y L□□S□	SS1	SS	1		
98.0 / 0.6	SILT-sandy, some clay, Very Loose				98	0 51 31 □8
		SS2	SS	3		
		SS3	SS	1	97	
96.3 / 2.3	CLAY-Silty, GREY/REDISH□□□□M □□ S□□□	SS4	SS	3	96	
	-Trace Organics	SS5	SS	2	95	
		SS6	SS	8		
					94	
		SS7	SS	3		
		SS8	SS	3	93	
92.6 / 6.0	END OF BOREHOLE					

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

Field vane sensitivity labels: 2, 8, 6, 4

NR = NO RECOVERY   +3, ×3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

ON_MOT GS-TB-019500 SLATE RIVER.GPJ DST_MIN.GDT 1/9/15

RECORD OF BOREHOLE No BH4

METRIC

W.P. 6013-E-0021 LOCATION Slate River Tributary Culvert STA 23+379 LT 16.5 m ORIGINATED BY PR

DIST Thunder Bay HWY 61 BOREHOLE TYPE Hand Auger Hole COMPILED BY DB

DATUM Local DATE 2014 09 05 CHECKED BY DM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, □ QUICK TRIAXIAL, × LAB VANE (50 100 150 200 250)	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
98.7	GROUND SURFACE										
	WATER and ORGANICS						98				
97.2 / 1.5	SAND-some gravel						97				
96.9 / 1.8	CLAY-silty						96				
							95				
94.7 / 4.0	END OF BOREHOLE										

ON_MOT-HIGH VANES GS-TB-019500 SLATE RIVER.GPJ DST_MIN.GDT 11/24/14

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BH5

METRIC

W.P. 6013-E-0021 LOCATION Slate River Tributary Culvert STA 23+372 LT 16.5 m ORIGINATED BY PR

DIST Thunder Bay HWY 61 BOREHOLE TYPE Hand Auger Hole COMPILED BY DB

DATUM Local DATE 2014 09 05 CHECKED BY DM

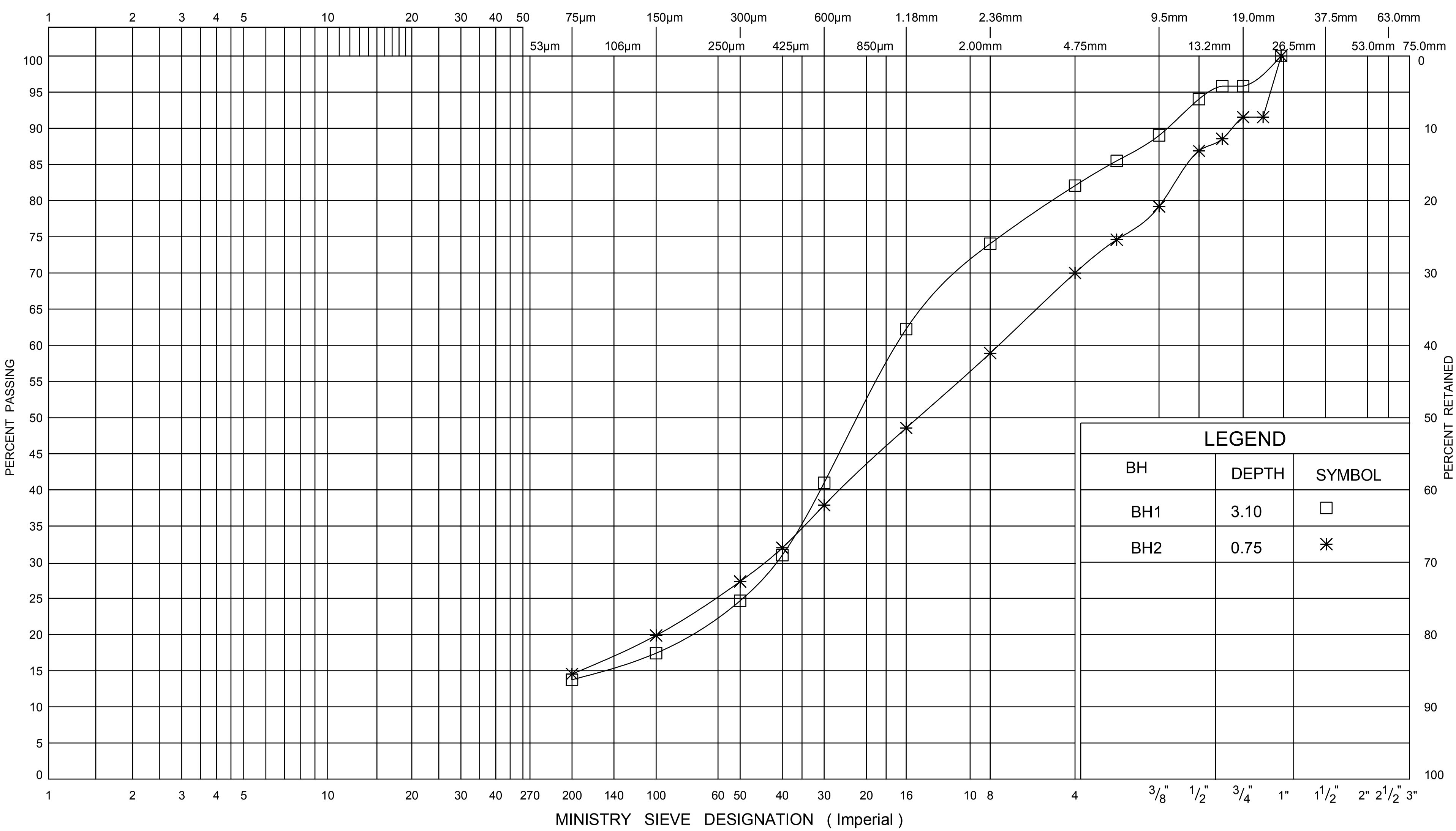
ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (50 100 150 200 250) ○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT w_P / NATURAL 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
98.7	GROUND SURFACE										
	WATER and ORGANICS						98				
97.6 / 1.1	SAND-some gravel										
97.3 / 1.4	CLAY-silty						97				
							96				
95.1 / 3.6	END OF BOREHOLE										

ON_MOT-HIGH VANES GS-TB-019500 SLATE RIVER.GPJ DST_MIN.GDT 11/24/14

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

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



LEGEND

BH	DEPTH	SYMBOL
BH1	3.10	□
BH2	0.75	✳

Ministry of Transportation Ontario

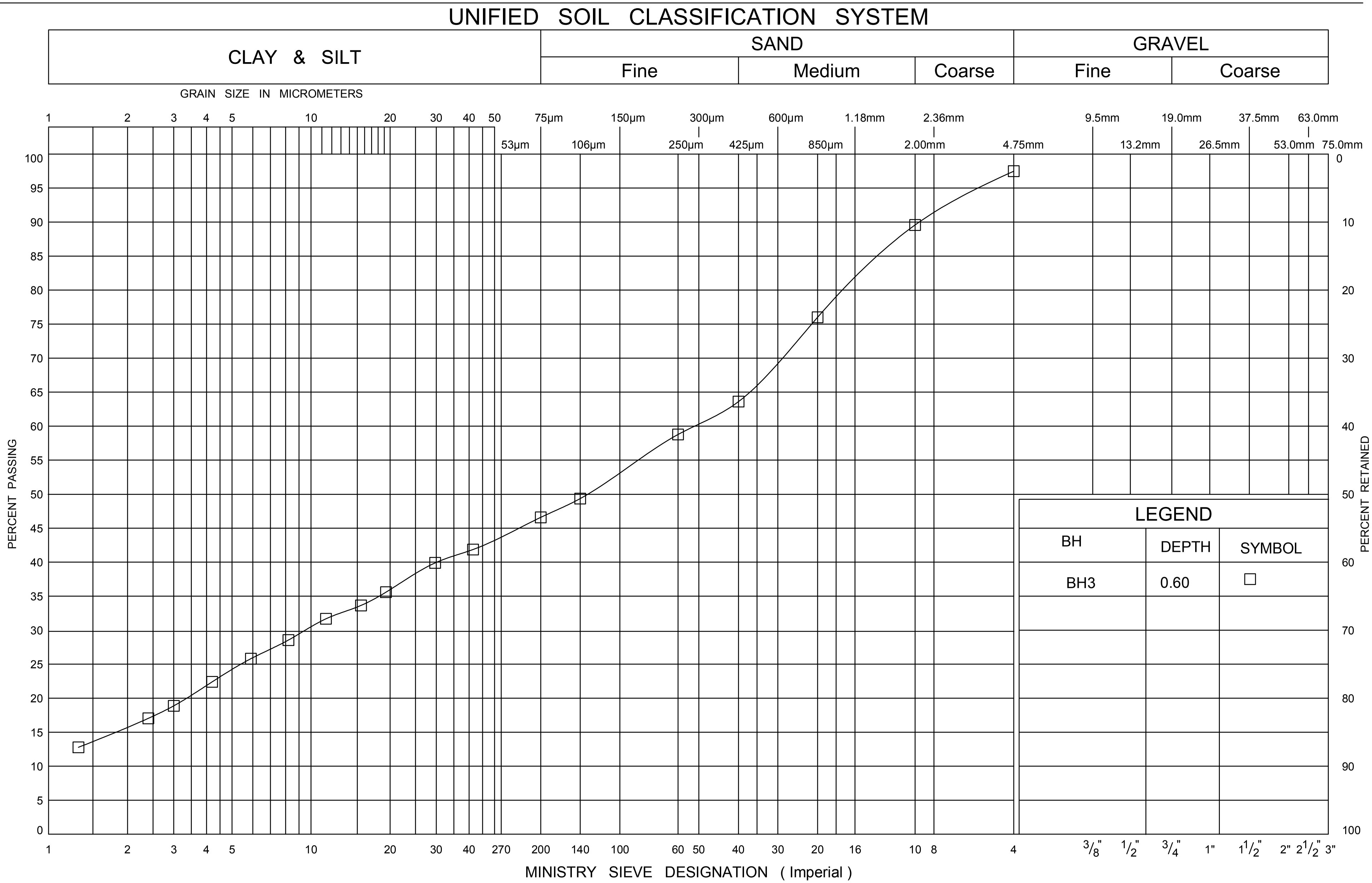
GRAIN SIZE DISTRIBUTION

□□LL-SA□□

ENCLOSURE □

W P 6013-E-0021

HWY 61



Ministry of Transportation Ontario

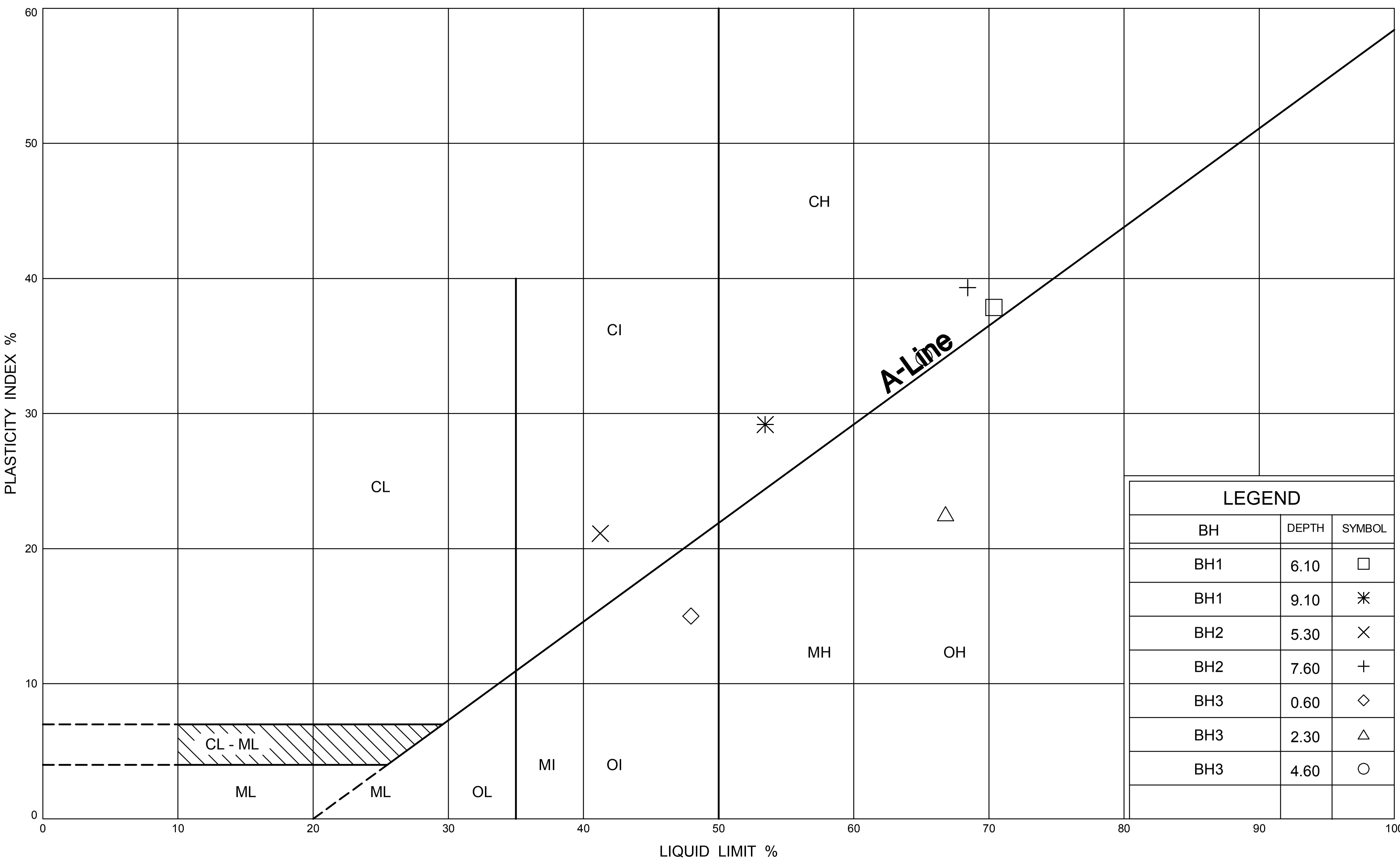
GRAIN SIZE DISTRIBUTION

S L

ENCLOSURE

W P 6013-E-0021

HWY 61



LEGEND		
BH	DEPTH	SYMBOL
BH1	6.10	□
BH1	9.10	✳
BH2	5.30	×
BH2	7.60	+
BH3	0.60	◇
BH3	2.30	△
BH3	4.60	○

Ministry of Transportation Ontario

PLASTICITY CHART
CLAY-Silty

ENCLOSURE 8

W P 6013-E-0021

HWY 61