



**FOUNDATION INVESTIGATION AND
PRELIMINARY DESIGN REPORT
JARVIS RIVER TRIBUTARY CULVERT REPLACEMENT
HIGHWAY 61
TOWNSHIP OF BLAKE, THUNDER BAY DISTRICT
AGREEMENT NO.: 6013-E-0021
ASSIGNMENT NO.: 9
SITE NO.: 48W-183C
GEOCRES NO. 52A-192
GWP 6304-14-00**

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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 0.4 km South of Cloud Lake Road (latitude 48.1769, longitude -89.4614), LHRs 33520, offset 13.710, Station 12+175, in the Township of Blake, in the District of Thunder Bay.

It is understood that the existing 47.2 m long centerline culvert is a cast-in-place concrete box culvert approximately 4.88 m wide and 3.05 m in height. The existing culvert (Figure 2.3 and 2.4) was originally built in 1899 and inspection by others indicates there is an extensive deterioration of concrete and severely corroded rebar in soffit. The fill thickness above the culvert is approximately 5.0 m and the side slope of the embankment is approximately 2H:1V. The surrounding area is moderately vegetated and wooded (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 Northeast)



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



Figure 2.3 Culvert inlet (looking Southeast)



Figure 2.4 Culvert outlet (looking West)

3. INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out between August 28, 2014 and September 4, 2014 utilizing a CME 750 drill rig equipped for geotechnical drilling and operated by DST. A total of four boreholes were advanced to depths ranging from 5.9 m to 17.2 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 Drawings 1 to 3. Borehole 1 was advanced south of the existing culvert at station 12+170, 5.0 m right of centreline, and advanced to a depth of 10.3 m below existing surface. Borehole 2 was advanced North of the existing culvert at station 12+185, 5.2 m left of centreline, and advanced to a depth of 17.2 m below existing surface. The remaining two boreholes were advanced with portable hand equipment at the inlet and outlet of the existing culvert. Borehole 3 was advanced at the outlet at station 12+182, 20.0 m right of centreline, and advanced to a depth of 5.9 m below existing surface. Borehole 4 was advanced at the inlet at station 12+165, 17.5 m left of centreline, and advanced to a depth of 5.9 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 12+175. A nail in a telephone pole at station 12+187 on the north side of the culvert was assigned as temporary 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 forty three (43) moisture contents, Two (2) sieve analyses, four (4) particle size analyses and two (2) Atterberg limits have been carried out for this assignment. Laboratory test results are presented in the Boreholes Logs and in graphical plots attached Appendix D (Enclosures).

Table 3.1 Detail of borehole locations

Borehole ID	Station	Elevation (m)	Depth (m)	Offset (m)
BH1	12+170	107.0	10.3	5.0 Rt
BH2	12+185	106.9	17.2	5.2 Lt
BH3	12+182	98.5	5.9	20.0 Rt
BH4	12+165	99.0	5.9	17.5 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 sand layer that is underlain by silt to clayey silt stratum.

Table 4.1 Summary of soil strata at the culvert location

Layer	Depth (m)	Elevation (m)	Comments
Asphalt	0 to 0.05	107.0 to 106.9 106.9 to 106.8	
Fill- Sand	0.05 to 7.6 0.05 to 6.1	106.9 to 99.4 106.8 to 100.8	
Silt –trace clay to clayey	7.6 to 10.3 6.1 to 17.2	99.4 to 96.7 100.8 to 89.7	

4.1 Asphalt

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

4.2 Topsoil

Topsoil was encountered at surface in Boreholes 3 and 4 with a thickness of approximately 0.2 m (Elev. 98.5 to 98.3 m) and 0.1 m (Elev. 99.0 m to 98.9 m) respectively.

4.3 Fill – Sand

Sand fill with some to with gravel, trace to some silt and cobbles was encountered in Boreholes 1 and 2 below the asphalt with a thickness of 7.6 m and 6.1 m at depths between 0.05 to 7.6 m (Elev. 107.0 to 99.4 m) and depths between 0.05 to 6.1 m (Elev. 106.9 to 100.8 m) respectively.

SPT 'N' values vary from 8 to 40, indicating a loose to dense condition. The moisture contents of samples tested range from 5 to 10 %. The results of laboratory tests are summarized in Table 4.2.

Table 4.2 Summary of sand fill sieve analyses

Laboratory Results - Sieve Analyses	
Gravel %	19 to 39
Sand %	43 to 61
Fines %	18 to 20

4.4 Silt -clayey

Silt clayey, trace to some sand, trace gravel was encountered in Boreholes 2, 3 and 4 at a depths 6.1 m (Elev. 100.8 m), 0.2 m (Elev. 98.3 m) and 0.1 m (Elev. 98.9 m) respectively. The thickness of this stratum was found to be between 2.8 to 4.6 m. The interbedded layer of silt and clayey silt was encountered within this stratum.

Atterberg limits tests carried out on samples from Boreholes 3 and 4 indicate that the plastic silt has liquid limits ranging from 45 to 62 % and plasticity indexes ranging from 15 to 25 %. The moisture content of the plastic silt ranges from 23 to 56 %. Field vane tests completed in Boreholes 2 and 3 showing 45 kPa to 100 kPa indicating a firm to stiff consistency. The laboratory test results are summarized in following Tables 4.3 and Table 4.4

Table 4.3 Summary of silt particle size analyses

Laboratory Results – Particle Size Analysis	
Gravel %	0 to 3
Sand %	10 to 11
Silt %	60 to 71
Clay %	15 to 30

Table 4.4 Summary of atterberg limits- silt

Laboratory Results – Atterberg Limits	
Liquid Limit %	45 to 62
Plastic Limit %	30 to 37
Plastic Index %	15 to 25

4.5 Silt

Silt trace to some clay, trace to some sand, trace gravel was encountered in Boreholes 1, 2, 3 and

4 at a depths 7.6 m (Elev. 99.4 m), 10.7 m (Elev. 96.2 m), 3.0 m (Elev. 95.5 m) and 3.7 m (Elev. 95.3 m) respectively. The thickness of this stratum is unknown as borehole terminus was reached within this stratum. The interbedded layer of silt and clayey silt was encountered within this stratum. The moisture content of the silt ranges from 18 to 30 %. The laboratory test results are summarized in following Tables 4.5.

Table 4.5 Summary of silt particle size analyses

Laboratory Results – Particle Size Analysis	
Gravel %	0
Sand %	1 to 3
Silt %	86 to 90
Clay %	8 to 13

4.6 Auger Refusal

Auger refusal on possible boulder was encountered in Borehole 1 at depth of 10.3 m (Elev. 96.7 m).

4.7 Groundwater

At the time of the field investigation groundwater was observed in Boreholes 1, 2, 3 and 4 and groundwater depths are summarized in Table 4.5. The groundwater levels can be expected to vary with the season and precipitation events.

Table 4.6 Groundwater Depths

Borehole	Groundwater Depth	Groundwater Elev.
Borehole 1	8.0	99.0
Borehole 2	11.0	95.9
Borehole 3	2.6	95.9
Borehole 4	1.5	97.5

5. MISCELLANEOUS

Site work was carried out between August 28, 2014 and September 4, 2014 utilizing a CME 750 drill rig equipped for geotechnical drilling and operated by DST. 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. has been retained by the Ministry of Transportation (MTO), Northwestern Region, to conduct a foundation investigation and preliminary design report for the replacement of the Jarvis River Tributary Culvert on Highway 61. This work was carried out under Agreement No. 6013-E-0021, Assignment No. 4 and Assignment No. 9, Foundation Investigation and Preliminary Design Report for the Replacement of Various Culverts.

Existing structure at this location is a 4.88 m in width x 3.05 m in height x 47.0 m in length cast-in-place concrete box culvert with a depth of soil cover of approximately 5.0 m. The culvert was identified with extensive deterioration of concrete and severely corroded rebar in the soffit of the culvert. The fill thickness above the culvert is approximately 5.0 m and the side slope of the embankment identified through visual inspection 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, consists of asphalt overlying a sand layer that is underlain by silt to clayey silt stratum.

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 loose silt 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. The design of the replacement structure must be in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC, 2006) and all relevant Ministry of Transportation specifications 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 will 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 2006 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 (4.88 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 10.4 m (Elevation 96.6 m) below top of pavement and bedding material placed on undisturbed native silt 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 = 4.9 m	210	105	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 47.2 m.

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

Footing Width L=47.2 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	70
	1.00	300	150	75
	1.50	440	220	80
B = 1.5 m	0.50	200	100	50
	1.00	330	115	55
	1.50	450	225	60
B = 2.0 m	0.50	230	115	40
	1.00	360	180	45
	1.50	490	245	50

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.

Culvert foundation can be constructed either with dewatering of excavation or without lowering the water in the excavation. If the construction of foundation is carried out in the wet, foundation construction can be performed by Tremie concrete placement. In this case, soil below foundation should be carefully prepared to minimize the disturbance. Alternatively, use of precast concrete footings can be considered.

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	Top Elevation (m)	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	106.9 to 106.8	21	32	17 (14)	-	-
Silt –clayey (plastic)	98.3 to 100.8	19	26	14 (11)	45	35
Silt (non-plastic)	96.2 to 95.5	19	30	14 (11)	-	-

[&] 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 [#]	Silt [#] (non plastic)	Silt [#] (plastic)
Active Earth Pressure (K_a)	$\left(\frac{1 - \sin \phi}{1 + \sin \phi} \right)$	0.27	0.30	0.33	0.39
Passive Earth Pressure (K_p)	$\left(\frac{1 + \sin \phi}{1 - \sin \phi} \right)$	3.68	3.27	3.0	2.55
At rest (K_0)	$(1 - \sin \phi)$	0.42	0.47	0.50	0.56

* 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) clayey silts the earth pressure coefficient at rest condition should be corrected using a following relationship

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

Where

$K_{0(OC)}$ = Earth pressure coefficient over consolidated soils

$K_{0(NC)}$ = Earth pressure coefficient normally consolidated soils

OCR = Over consolidation Ratio

The sliding resistance can be calculated using the following formulae.

$$F_r = W (\tan \delta)$$

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 and rock fill 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 and boulders 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, 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 is recommended during construction. 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 2006 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. 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 , specified in OPSS 1860.

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, v_v . 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.

$$q = \gamma h b v_v, \text{ where}$$

q = vertical load on the culvert

γ = unit weight of soil

h = thickness of soil above the culvert

b = width of the culvert, and

v_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 well-point 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 river 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 in 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 in 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 walls 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 2.0 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 2.0 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 2.0 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 the 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 shoring structure 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 pile, soldier pile and gabion walls 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 the responsibility of the contractor as per the contract drawings.

Table 7.1 Advantages and disadvantages of various roadway protection methods

Roadway Protection Option	Advantages	Disadvantages
Sheet Pile	<ul style="list-style-type: none"> • Relatively non permeable • Increased erosion control capacity 	<ul style="list-style-type: none"> • Lightweight material may encounter difficult driving through cobbles • Higher installation cost • Specialized construction and design required
Soldier Pile	<ul style="list-style-type: none"> • Heavier gauge materials may be better to be able to accommodate presence of cobbles • Lower cost 	<ul style="list-style-type: none"> • Permeable • Potential for erosion of retained materials • longer installation time
Gabion Wall	<ul style="list-style-type: none"> • Ease of Installation • Lower cost • Presence of cobbles and boulders is not an issue 	<ul style="list-style-type: none"> • 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 Works*" Spec No. OPSS 511, 517, 518, 805, 902.
- Municipal and Provincial Common, Volume 2 - Material Specifications, "*Ontario Provincial Standard for Roads & Public Works*" 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 Works*" 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 Works*" Spec No. OPSS.PROV 209, 501, 510, 539.
- Provincial-Orientated, Volume 6 - Material Specifications, "*Ontario Provincial Standard for Roads & Public Works*" 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.

Prepared by:



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Manager

Reviewed by:



Selorm Danku P. Eng.
Geotechnical Engineer

Reviewed By:



Dr. M W Bo, PhD., P. Eng, P.Geo, Int PE,
C.Geol, C. Eng, Eur Geol, Eur Eng
Senior Vice President / Senior Principal

Appendix

M

Appendix B

DESCRIPTION OF TERMS

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 \bar{N} .

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 ABBREVIATIONS 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.

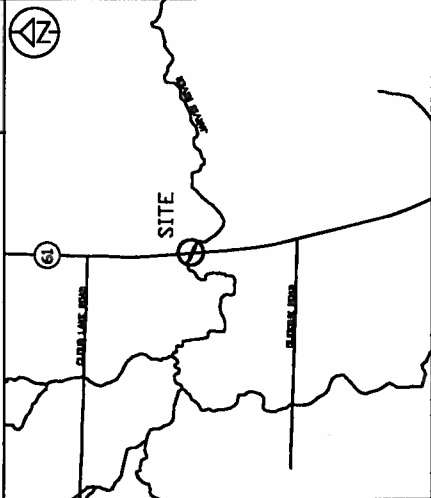
Appendix C

DRAWINGS

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SPECIFIED
IN DIMENSIONS + METERS

CONT	No	
GWP	No 6304-14-00	
SITE	No 48W-183/C	
GEORES	No 52A-192	

CULVERT REPLACEMENT JARVIS RIVER CULVERT	SHEET
STA 12+165 TO STA 12+185	Survey _____ Revised _____



KEY MAP

LEGEND

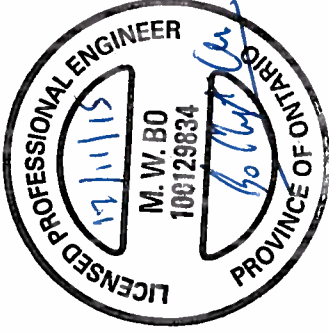
Borehole

No.	Elevation	Northing	Easting	Station	Offset
BH1	107.0	5338548 m N	317024 m E	12+170	5.0 m RT
BH2	106.9	5338556 m N	317014 m E	12+185	5.2 m LT
BH3	98.5	5338556 m N	317047 m E	12+182	20.0 m RT
BH4	99.0	5338554 m N	317000 m E	12+165	17.5 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



BH3

EDGE OF PAVEMENT

JARVIS RIVER
FLOW DIRECTION

PLAN MAP
SCALE



HIGHWAY 61

BH2

12+185

12+180

12+175

12+170

12+165

HIGHWAY 61

NORTHBOUND LANE
SOUTHBOUND LANE

47.20 m X 4.88 m CULVERT
FLOW DIRECTION

EDGE OF PAVEMENT

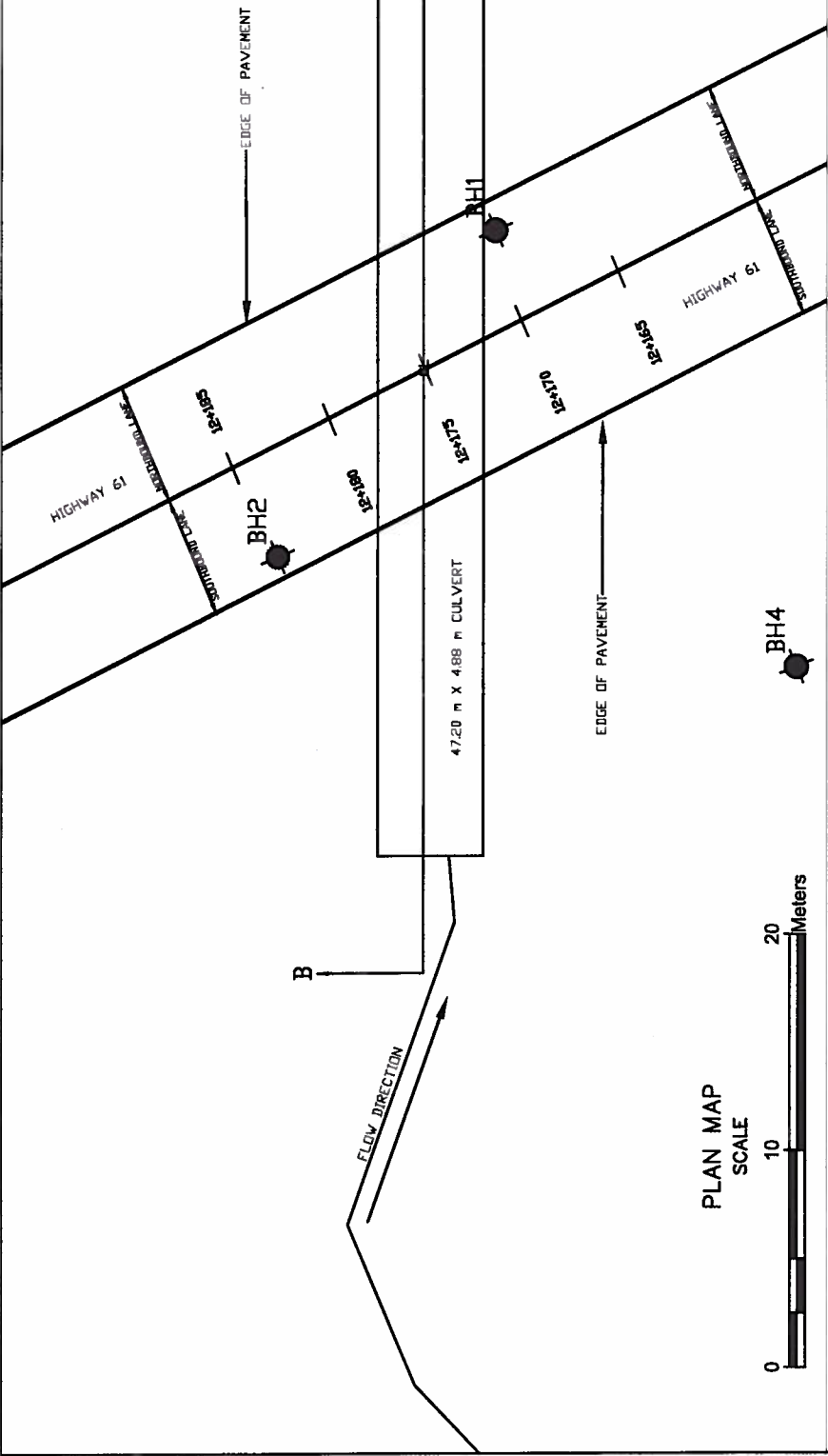
BH4

BH1

METRIC
DIMENSIONS ARE IN METRES
DIMENSIONS IN FEET AND INCHES
DIMENSIONS IN KILOMETRES + METRES



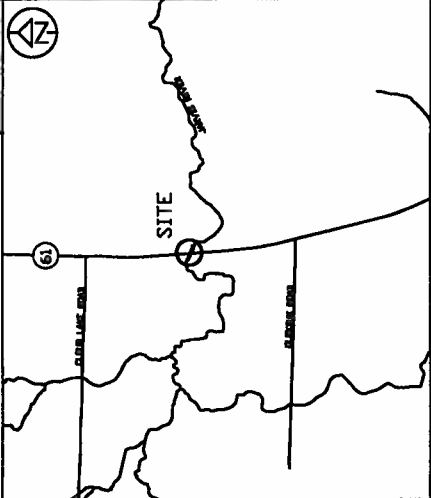
BH3



PLAN MAP
SCALE
10 20
Meters

CONT No
GWP No 6304-14-00
SITE No 48W-183/C
GEOGRES No 52A-192

SHEET
CULVERT REPLACEMENT
JARVIS RIVER CULVERT
STA 12+165 TO STA 12+185
Survey _____ Revised _____



KEY MAP

LEGEND

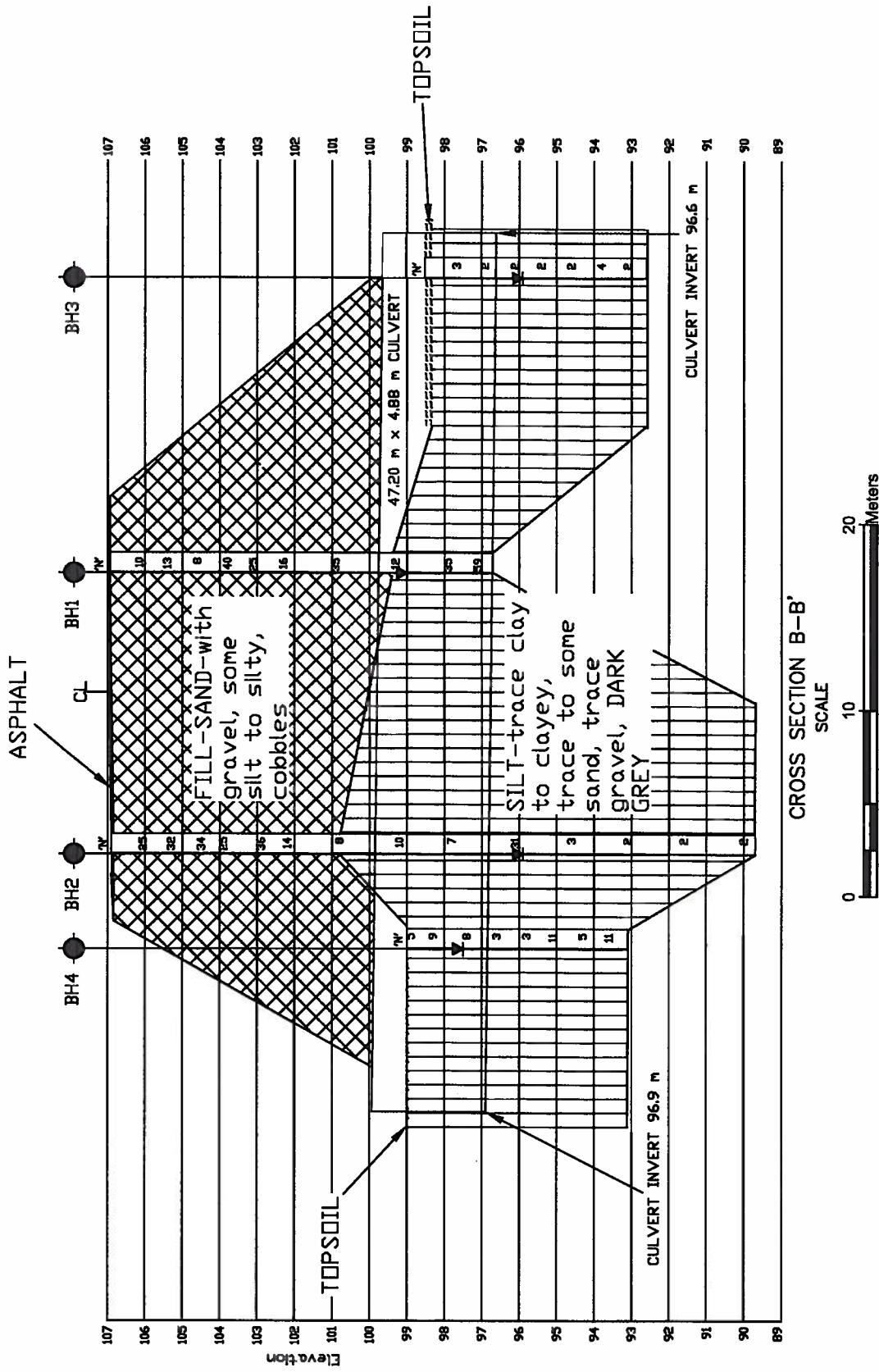
- Borehole
- 'N' Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Water level at time of investigation
- Fill
- Organics
- Topsoil
- Till
- Bedrock
- Sand
- Silt
- Clay
- Sand & Gravel
- Boulders

No.	Elevation	Nothing	Ending	Station	Offset
BH1	107.0	5338643 m N	317024 m E	12+170	5.0 m RT
BH2	106.9	5338668 m N	317014 m E	12+186	5.2 m LT
BH3	98.5	5338686 m N	317047 m E	12+182	20.0 m RT
BH4	98.0	5338654 m N	317000 m E	12+185	17.5 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



CROSS SECTION B-B'
SCALE
10 20
Meters

Appendix D
ENCLOSURES

RECORD OF BOREHOLE No BH1

1 OF 1

METRIC

W.P. 6013-E-0023 LOCATION Jarvis River Culvert Hwy 61: STA. 12+170, 5.0m RT ORIGINATED BY PR
DIST Thunder Bay HWY 61 BOREHOLE TYPE Hollow Stem Auger 80 mm COMPILED BY MD
DATUM LOCAL DATE 2014 08 28 CHECKED BY DB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
107.0	GROUND SURFACE																
106.9	ASPHALT																
106.3	FILL - SAND-some grave, trace silt		AS1	AS													
	FILL-SAND - some to with gravel, some silt, cobbles, BROWN L S		SS2	SS	10												
			SS3	SS	13												
			SS4	SS	8												
			SS5	SS	40												
			SS6	SS	25												
			SS7	SS	16												
			SS8	SS	35												
99.4																	
7.6	SILT-Clayey to trace clay, trace to some sand, trace gravel, STI, BROWN		SS9	SS	12												
			SS10	SS	35												
	-BROWN/BLACK																
96.7			SS11	SS	59												
10.3	END OF BOREHOLE Auger Refusal Possible Boulder																

ON MOT GS-TB-019499 - JARVIS RIVER HWY 61_BHLOGS.GPJ DST_MIN.GDT 11/18/14

NR = NO RECOVERY

+ 3, X 3: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH2

1 OF 1

METRIC

W.P. 6013-E-0023 LOCATION Jarvis River Culvert Hwy 61: STA. 12+185, 5.2m LT ORIGINATED BY PR
DIST Thunder Bay HWY 61 BOREHOLE TYPE Hollow Stem Auger 80 mm COMPILED BY MD
DATUM LOCAL DATE 2014 08 28 CHECKED BY DB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
	GROUND SURFACE							20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT		
								20 40 60 80 100	W _p	W	W _L		
								20 40 60 80 100					
106.9	ASPHALT		AS1	AS									
106.8	FILL - SAND- some gravel, trace silt		SS2	SS	25		106						19 61 (20)
106.5	FILL-SAND-Some silt to silty, some gravel, cobbles, BROWN COMPACT		SS3	SS	32		105						
			SS4	SS	34		104						
			SS5	SS	25		103						
			SS6	SS	36		102						
			SS7	SS	14		101						
100.8							100						
6.1	SILT-Clayey to trace clay, trace to some sand, trace gravel, GREY/DARK GREY, STIFF		SS8	SS	8		99						3 11 71 15
			SS9	SS	10		98						
			SS10	SS	7		97						0 10 60 30
			SS11	SS	31		96						
			SS12	SS	3		95						0 3 87 10
			SS13	SS	2		94						
			SS14	SS	2		93						
							92						
							91						
89.7							90						0 2 90 8
17.2	END OF BOREHOLE		SS15	SS	2								

ON_MOT_GS-TB-019499 - JARVIS RIVER HWY 61_BHLOGS.GPJ DST_MIN.GDT 11/18/14

NR = NO RECOVERY

+ 3, X 3: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

ENCLOSURE 2

RECORD OF BOREHOLE No BH3

1 OF 1

METRIC

W.P. 6013-E-0023 LOCATION Jarvis River Culvert Hwy 61: STA. 12+182, 20.0m RT ORIGINATED BY PR
DIST Thunder Bay HWY 61 BOREHOLE TYPE Hollow Stem Auger 80 mm COMPILED BY MD
DATUM LOCAL DATE 2014 09 02 CHECKED BY DB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	□ QUICK TRIAXIAL	× LAB VANE						
98.5	GROUND SURFACE						20	40	60	80	100	20	40	60	kN/m ³	GR SA SI CL	
98.3	TOPSOIL		AS1	AS		▽	98									0 1 86 13	
0.2	SILT-Clayey to trace clay, GREY/DARK GREY, FIRM to STIFF W _d □□□□□□		SS2	SS	3		97			+							
			SS3	SS	2		96			+							
			SS4	SS			95										
			SS5	SS	2		94										
			SS6	SS	2		93										
			SS7	SS	4												
			SS8	SS	2												
92.6	END OF BOREHOLE																
5.9																	

ON_MOT_GS-TB-019499 - JARVIS RIVER HWY 61_BHLOGS.GPJ DST_MIN.GDT 11/18/14

NR = NO RECOVERY

+³, ×³: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

ENCLOSURE 3

RECORD OF BOREHOLE No BH4

1 OF 1

METRIC

W.P. 6013-E-0023 LOCATION Jarvis River Culvert Hwy 61: STA. 12+165, 17.5,m LT ORIGINATED BY PR
DIST Thunder Bay HWY 61 BOREHOLE TYPE Hollow Stem Auger 80 mm COMPILED BY MD
DATUM LOCAL DATE 2014 09 04 CHECKED BY DB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20	40	60	80	100					
99.0	GROUND SURFACE																
98.9	TOPSOIL		SS1	SS	5	▽	98										
	SILT-Clayey to trace clay, some sand, trace gravel, S □ T □ □ STI □ BROWN/DARK GREY		SS2	SS	9												
	-DARK GREY		SS3	SS	8												
	□ Tr □ □ W □ d		SS4	SS	3												
			SS5	SS	3												
			SS6	SS	11												
			SS7	SS	5												
			SS8	SS	11												
93.1	END OF BOREHOLE																
5.9																	

ON_MOT_GS-TB-019499 - JARVIS RIVER HWY 61_BHLOGS.GPJ DST_MIN.GDT 11/18/14

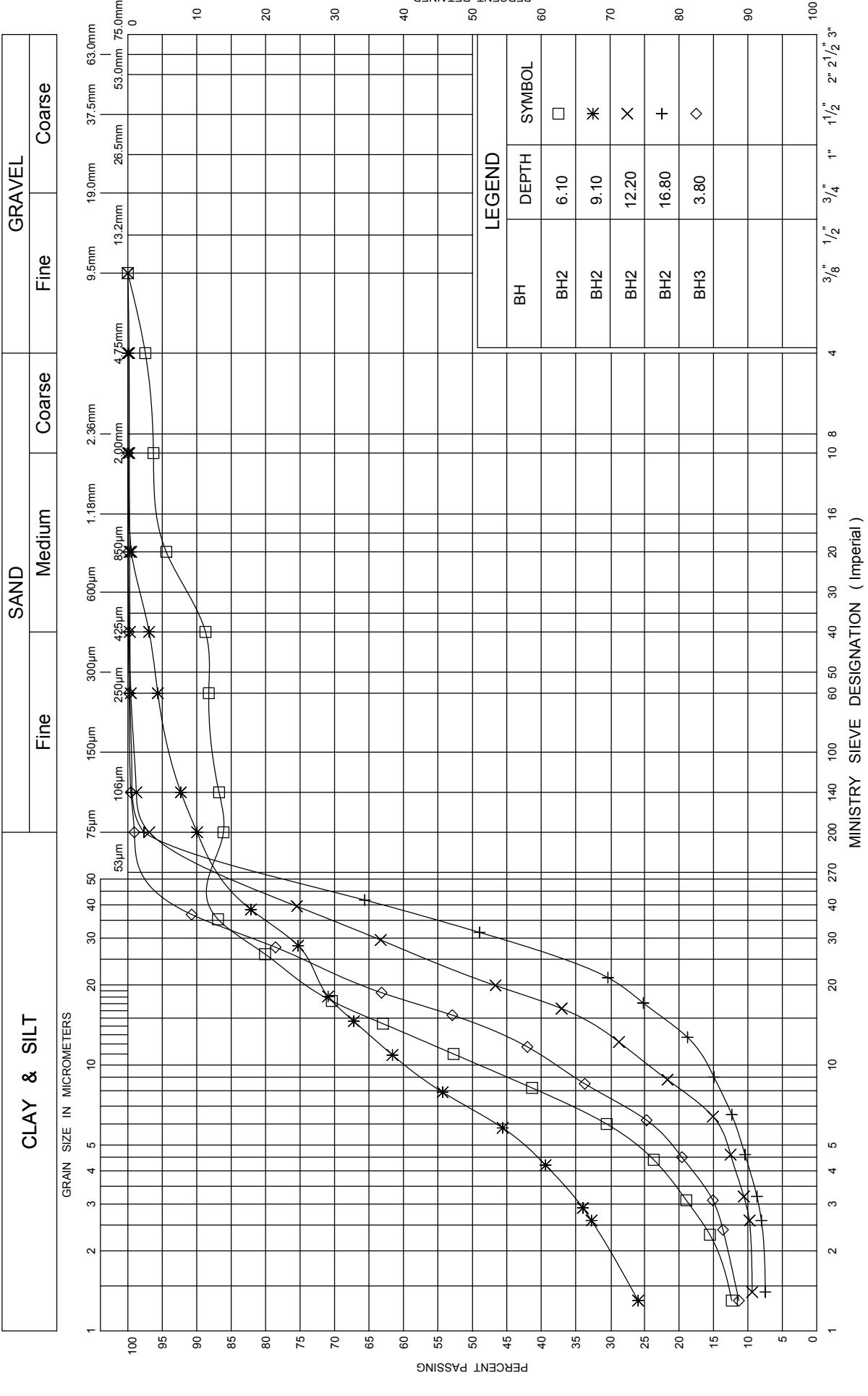
NR = NO RECOVERY

+ 3, × 3: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

ENCLOSURE 4

UNIFIED SOIL CLASSIFICATION SYSTEM

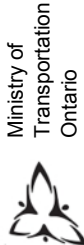


GRAIN SIZE DISTRIBUTION
SILT

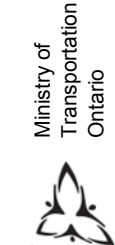
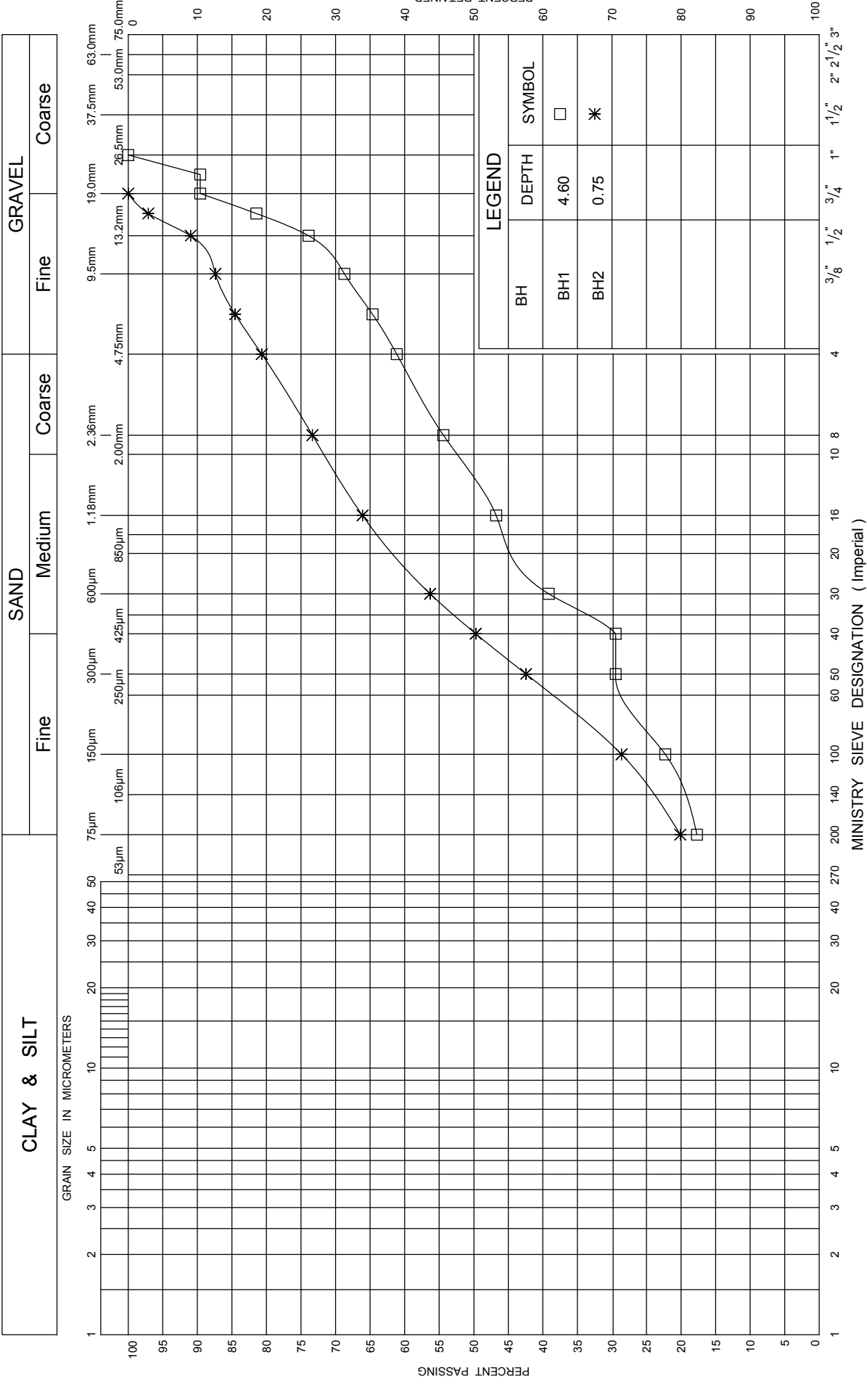
ENCLOSURE ☐

WP 6013-E-0023

HWY 61



UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

SA □ □

ENCLOSURE □

WP 6013-E-0023

HWY 61

