



**FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN REPORT
CEDAR CREEK CULVERT REPLACEMENT
HIGHWAY 595
TOWNSHIP OF O'CONNOR, THUNDER BAY DISTRICT
AGREEMENT NO.: 6013-E-0021
ASSIGNMENT NO.: 9
SITE NO.: 48W-171C
GEOCRES NO. 52A-193
GWP 6353-14-00**

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Table of Contents

1. INTRODUCTION	1
2. SITE DESCRIPTION	2
3. INVESTIGATION PROCEDURES AND LABORATORY TESTING.....	5
4. DESCRIPTION OF SUBSURFACE CONDITIONS	7
4.1 Surface Treatment.....	7
4.2 Fill – Sand.....	7
4.3 Fill – Silt	8
4.4 Fill-Silty Clay	8
4.5 Silty Clay	9
4.6 Gravel	9
4.7 Sand	10
4.8 Till	10
4.9 Auger Refusal.....	10
4.10 Groundwater	11
5. MISCELLANEOUS	12
6. PROJECT DESCRIPTION	13
6.1 Replacement Structure	14
6.1.1 Earth Excavation	14
6.1.2 Foundation Design.....	15
6.1.3 Lateral and Sliding Resistances	16
6.1.4 Roadway Protection	18
6.1.5 Sidefill and Overfill	19
6.1.6 Dewatering	20
6.1.7 Erosion Control	21
6.1.8 Frost Protection.....	21
6.1.9 Embankment Foreslopes	22
6.1.10 Construction Concerns	22
7. CLOSURE.....	23
8. REFERENCES	24
9. LIMITATIONS OF REPORT	25

Appendices

LIMITATIONS OF REPORT	'A'
DESCRIPTION OF TERMS.....	'B'
DRAWINGS	'C'
ENCLOSURES	'D'

List of Tables

Table 3.1	Detail of borehole locations	6
Table 4.1	Summary of soil strata at the culvert location	7
Table 4.2	Summary of sand and gravel fill sieve analyses	8
Table 4.3	Summary of silt fill particle size analyses	8
Table 4.4	Summary of Atterberg limits- clay	9
Table 4.5	Summary of clay particle size analyses	9
Table 4.6	Summary of Atterberg limits- clay	9
Table 4.7	Summary of gravel sieve analyses.....	10
Table 4.8	Summary of till sieve analyses	10
Table 4.9	Summary of Groundwater	11
Table 6.1	Geotechnical resistances and reactions for footings on bedrock.....	16
Table 6.2	Typical soil parameters for earth loads	17
Table 6.3	Lateral Earth Pressure Coefficients	17
Table 6.4	Coefficient of Friction for sliding resistance	18
Table 6.5	Sliding Resistance per unit length of the footing.....	18

List of Figures

Figure 2.1	Location of existing culvert at Highway 595 (looking North)	3
Figure 2.2	Location of existing culvert at Highway 595 (looking South)	3
Figure 2.3	Culvert outlet (looking West)	4
Figure 2.4	Culvert inlet (looking East)	4

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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 foundation design report for the proposed culvert replacement on Highway 595. 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 595, approximately 1.45 km South of Highway 590 (latitude 48.3861, longitude -89.7008), LHRS 62920, offset 10.830, Station 20+860, in the Township of O'Connor, in the District of Thunder Bay.

It is understood that the existing 11.5 m long centerline culvert is a three sided cast-in-place concrete box culvert approximately 6.1 m wide and 2.3 m in height. The existing culvert (Figure 2.3 and 2.4) has an original date of construction that is unknown and inspection conducted by others indicates there is delamination and spalls, wide cracks in the retaining walls and exposed rebar in the soffit. The fill thickness above the culvert is approximately 0.7 m and the side slope of the embankment is approximately 1.5H: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 O'Connor area. The map indicates that the local area landform is identified as a till, clay ground moraine. The topography in the area is mainly low local relief; plain with dry drainage conditions.



Figure 2.1 Location of existing culvert at Highway 595 (looking North)



Figure 2.2 Location of existing culvert at Highway 595 (looking South)



Figure 2.3 Culvert outlet (looking West)



Figure 2.4 Culvert inlet (looking East)

3. INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out on August 20th and 21st, 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 0.5 m to 5.0 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 and Drawings 2 to 3. Borehole 1 was advanced South of the existing culvert at station 20+856, 1.5 m right of centreline, and advanced to a depth of 5.0 m below surface. Borehole 2 was advanced North of the existing culvert at station 20+864, 1.7 m left of centreline, and advanced to a depth of 3.5 m below existing surface. Borehole 3 was advanced 2.0 m North of Borehole 2 at station 20+866 to confirm the soil conditions. The remaining two boreholes were advanced with portable hand equipment at the inlet and outlet of the existing culvert. Borehole 4 was advanced at the outlet at station 20+860, 7.5 m right of centreline, where bedrock was encountered on the surface. Borehole 5 was advanced at the inlet at station 20+860, 7.5 m left of centreline, and advanced to a depth of 0.5 m below existing surface.

The borehole locations are referenced to the MTO Station numbering system as indicated on the drawings provided by the Ministry. The ground surface elevations at the borehole locations were surveyed by DST personnel and referenced to the existing culvert at Station 20+860. The nail on telephone/hydro pole # 282 on the north side of the culvert at approximately Station 20+894, 10.0 m right 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 eighteen (18) moisture contents, three (3) sieve analyses, two (2) particle size analyses and five (5) Atterberg limits have been done for this assignment. Laboratory test results are presented in the Boreholes Logs and graphical plots attached Appendix D (Enclosures).

Table 3.1 Detail of borehole locations

Borehole ID	Station	Elevation (m)	Depth (m)	Offset (m)
BH1	20+856	100.1	5.0	1.5 Rt
BH2	20+864	100.1	3.5	1.7 Lt
BH3	20+866	100.1	4.3	1.7 Lt
BH4 (HA)	20+860	95.4	0.0	7.5 Rt
BH5 (HA)	20+860	95.0	0.5	7.5 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, 2, and 3, consists of surface treatment and cold mix patching overlying a fill consisting of sand, silt and clay underlain by silty clay which is again underlain by a till material overlaying bedrock.

Table 4.1 Summary of soil strata at the culvert location

Layer	Depth (m)	Elevation (m)	Comments
Surface Treatment	0.0 to 0.1 0.0 to 0.2	100.0 to 99.9 100.1 to 99.9	Cold mix patching
Fill- Sand	0.1 to 0.8 0.2 to 0.8	99.9 to 99.3 99.9 to 99.4	
Fill- Silt	0.8 to 1.5	99.3 to 98.5	
Fill- Silty Clay	0.8 to 3.1	99.4 to 97.0	
Silty Clay	1.5 to 4.6	98.5 to 95.4	
Till	4.6 to 5.0 3.1 to 3.5	95.4 to 95.0 97.0 to 96.6	

4.1 Surface Treatment

Surface treatment with cold mix patching was encountered at surface in Boreholes 1, 2, and 3 with thicknesses ranging from 60 to 200 mm.

4.2 Fill – Sand

Sand fill with some gravel, trace to some silt was encountered in Boreholes 1, 2, and 3 below the asphalt at depth of 0.1 m, 0.2 m and 0.2 m with a thickness of 0.7 (Elev. 99.9 to 99.3 m), 0.6 (Elev. 99.9 to 99.4 m), and 0.3 m (Elev. 100.0 to 99.7 m) respectively.

The moisture contents of the sand and gravel material vary from 3 to 5 %. The results of laboratory tests are summarized in Table 4.2.

Table 4.2 Summary of sand and gravel fill sieve analyses

Laboratory Results - Sieve Analyses	
Gravel %	30
Sand %	59
Fines %	11

4.3 Fill – Silt

Silt fill with sand, trace gravel, trace clay and cobbles was encountered in Boreholes 1 and 3 at depth of 0.8 m and 0.5 m with a thickness of 0.7 m (Elev. 99.3 to 98.5 m) and 1.0 m (Elev. 99.7 to 98.6 m) respectively.

SPT 'N' value found to be 9, indicating a loose condition. The moisture contents of the silt material vary from 12 to 20 %. The results of laboratory tests are summarized in Table 4.3.

Table 4.3 Summary of silt fill particle size analyses

Laboratory Results – Particle Size Analyses	
Gravel %	2
Sand %	36
Silt %	52
Clay %	10

4.4 Fill-Silty Clay

Silty Clay Fill was encountered in Borehole 2 at depth of 0.8 m with thickness of 2.3 m (Elev. 99.4 to 97.0 m). Atterberg limits tests carried out on samples from Borehole 2 indicate that the clay has low to medium plasticity. The moisture content of the clay ranges from 17 to 29 %. Field vane tests completed in Boreholes 2 showing 65 kPa indicating a stiff consistency. The laboratory test results are summarized in following Tables 4.4 and 4.5.

Table 4.4 Summary of Atterberg limits- clay

Laboratory Results – Atterberg Limits	
Liquid Limit %	26 to 37
Plastic Limit %	16 to 23
Plastic Index %	9 to 14

Table 4.5 Summary of clay particle size analyses

Laboratory Results – Particle Size Analyses	
Gravel %	0
Sand %	30
Silt %	36
Clay %	34

4.5 Silty Clay

A silty clay material was encountered in Boreholes 1 and 3 at a depths of 1.5 m (Elev. 98.5 m) and 1.5 m (Elev. 98.6 m) with thickness of 3.1 m (Elev. 98.5 to 95.4 m), and 1.6 m (Elev. 98.6 to 97.0 m) respectively. In Boreholes 3 silty clay was again encountered at depths of 3.7 m (Elev. 96.4 m) with thickness of 0.3 m (Elev. 96.4 to 96.1 m).

Atterberg limits tests carried out on samples from Boreholes 1 and 3 indicate that the clay has low to high plasticity. The moisture content of the clay ranges from 18 to 35 %. Field vane test completed in Boreholes 1 showing 100 kPa indicating a stiff consistency. The laboratory test results are summarized in following Tables 4.6

Table 4.6 Summary of Atterberg limits- clay

Laboratory Results – Atterberg Limits	
Liquid Limit %	33 to 51
Plastic Limit %	22 to 27
Plastic Index %	9 to 23

4.6 Gravel

Gravel with some sand, trace silt was encountered in Borehole 3 at depth of 3.1 m with thickness of 0.7 m (Elev. 97.0 to 96.3 m). The moisture content of the tested sample was found to be 10%. The results of laboratory tests are summarized in Table 4.7.

Table 4.7 Summary of gravel sieve analyses

Laboratory Results – Sieve Analyses	
Gravel %	79
Sand %	15
Fines %	6

4.7 Sand

Sand with some gravel, cobbles and boulders was encountered in Borehole 5 at the surface. The thickness of this stratum is not defined as borehole terminus was reached within this stratum due to auger refusal on possible bedrock at depth of 0.5 m (94.8 m).

4.8 Till

Till material with sand and gravel, with silt and trace clay was encountered in Borehole 1 and 3 at depth of 4.6 m (Elev. 95.4 m) and 4.0 m (Elev. 96.1 m). The thickness of this stratum is not defined as borehole terminus was reached within this stratum due to auger refusal on possible bedrock.

The moisture content of the tested sample was found to be 20%. The results of laboratory tests are summarized in Table 4.8.

Table 4.8 Summary of till sieve analyses

Laboratory Results – Sieve Analyses	
Gravel %	44
Sand %	36
Fines %	20

4.9 Auger Refusal

Auger Refusal on possible bedrock was encountered in Borehole 1, 3, 4 and 5 at depth of 5.0 m (Elev. 95.0 m), 4.3 m (Elev. 95.8 m), 0.0 m (Elev. 95.0 m) and 0.5 m (Elev. 94.8 m) respectively. For Borehole 2 auger refusal on possible concrete abutment was encountered at depth of 3.5 m (Elev. 96.6 m).

4.10 Groundwater

At the time of the field investigation groundwater was observed in Boreholes 1 and 2 and groundwater depths are summarized in Table 4.9. The groundwater levels can be expected to vary with the season and precipitation events.

Table 4.9 Summary of Groundwater

Borehole	Groundwater Depth (m)	Groundwater Elevation (m)
Borehole 1	3.0	97.0
Borehole 2	3.0	97.1

5. MISCELLANEOUS

Site work was carried out during the week of August 20th, 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. has been retained by the Ministry of Transportation (MTO), Northwestern Region, to conduct a foundation investigation and design report for the replacement of the Cedar Creek Culvert on Highway 595. This work was carried out under Agreement No. 6013-E-0021, Assignment No.4 and Assignment No. 9, Foundation Investigation and Design Report for the Replacement of Various Culverts.

Existing structure at this location is a 6.1 m in width x 2.3 m in height x 11.5 m in length three sided (open bottom) cast-in-place concrete box culvert with a depth of soil cover of approximately 0.7 m. The culvert was identified with delamination and spalling, wide cracks in the retaining walls and exposed rebar in the soffit. The fill thickness above the culvert is approximately 0.7 m and the side slope of the embankment is approximately 1.5H:1V. The culvert replacement is recommended to be of a similar three sided (open bottom) concrete box culvert.

The generalized stratigraphy of the existing embankment, based on the conditions encountered in boreholes, consists of a surface treatment overlying a sand layer that is underlain by silt, followed by a silty clay layer which is again underlain by a till followed by a possible bedrock.

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

6.1 Replacement Structure

The proposed replacement structure at this site is a three sided (open bottom) cast-in-place or precast concrete box culvert. It is understood that open cut excavation will be used to replace the structure. 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".

where 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. 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. 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. 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 Foundation Design

The culvert will be located approximately at the same elevation and location as the existing culvert. Due to presence of bedrock at relatively shallow depth a shallow foundation on bedrock 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 footings should be installed on bedrock at depth of 4.3 m to 5.0 m (Elevation 95.0 to 95.8 m) below the road pavement.

Footings founded on bedrock with tight joints may be designed on the basis of an ultimate bearing capacity of 1 MPa, subject to the recommendations in this report. 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. Since the bedrock is considered incompressible and no settlement issues are expected therefore SLS values is considered equal to ULS values. Minimum foundation widths of 0.5 m are recommended. Bedrock depths may vary unpredictably.

All foundations systems should be founded on sound bedrock (rough to very rough surface, unweathered to slightly weathered, moderately close to wide spacing discontinuities and free of loose rock fragments) on a surface that has been suitably prepared with tight joints. Footings should not be founded partially on bedrock and partially on soil. All bearing surfaces should be clear of all loose, fractured or highly weathered rock prior to concrete placement. Where the bedrock slopes, foundations should be designed to resist lateral movements. A friction factor of 0.7 can be used for mass concrete on the original bedrock surface, and 0.8 for mass concrete on freshly excavated bedrock.

The use of rock anchor set into the bedrock is recommended where the rock surface slopes at greater than 15° from the horizontal and for the uplift tension. The bearing surface should be inspected by the Quality Verification Engineer to confirm the competency of the rock and design assumptions. Highly fractured rock or rock of lesser strength that is exposed during excavation may require special attention such as removal and/or pressure grouting.

Bedrock subgrade preparation requires the following procedures under the direction of the Quality Verification Engineer:

- Sub-excavate any zones of weak rock.
- Clear all bearing surfaces of soil and loose rock as identified by excavator equipment.

- Assess any open or soil-filled joints. Unless advised otherwise by the Geotechnical Engineer, treat by cleaning cracks and joints to a minimum depth equal to 3 times the width and filling with cement mortar or slush grout, dependent on crack width.
- Apply dental concrete to level out rock surface sufficiently for uniform foundation support.

All existing topsoil and/or other deleterious materials (including construction debris) must be removed prior to the start of subgrade preparation.

Bearing areas will require very careful preparation. Following excavation all bearing surfaces should be cleaned of all organic, loose, disturbed, or slough material prior to concreting or placing compacted fill material. Bearing surfaces should be protected at all times from rain, freezing temperatures and the ingress of groundwater before, during and after construction.

All foundation excavations and bearing surfaces should be inspected by a quality verification engineer to confirm the integrity of the bearing surface.

Table 6.1 Geotechnical resistances and reactions for footings on bedrock

Depth (m)	Width of Footing (B) (m)	Ultimate Bearing Capacity (kPa)	Resistance at ULS (kPa)	Reaction at SLS (kPa)
Surface	0.5	500	250	250
Surface	1.0	1000	500	500
Surface	1.5	2,250	1,125	1,125

6.1.3 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 6.2 Typical soil parameters for earth loads

Soil type	Unit weight (kN/m ³)	Internal drained friction angle (Deg)	Soil/concrete Interface friction angle δ (Deg)	Intact undrained shear strength (kPa)	Adhesion (kPa)
Till	22	35	22	-	-
Granular A	21	35	22	-	-
Fill-Sand	21	35	17	-	-
Clay-Silty	19	26	14	45	35
Silt	19	30	14	-	-
Sand	20	32	17	-	-
Gravel	21	34	22	-	-

& 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 6.3 Lateral Earth Pressure Coefficients

Earth Pressure Coefficient	Equation*	Granular A Sand Fill Till [#]	Clay-silty [#]	Silt [#]	Sand [#]	Gravel [#]
Active Earth Pressure (K_a)	$\left(\frac{1 - \sin\phi}{1 + \sin\phi} \right)$	0.27	0.39	0.33	0.30	0.28
Passive Earth Pressure (K_p)	$\left(\frac{1 + \sin\phi}{1 - \sin\phi} \right)$	3.68	2.55	3.0	3.25	3.53
At rest (K_0)	$(1 - \sin\phi)$	0.42	0.56	0.5	0.47	0.44

* 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) soils 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 following table provides a coefficient of friction for various soils with cast in place or pre cast concrete that may be used in the assessment of sliding resistances.

Table 6.4 Coefficient of Friction for sliding resistance

Soils	Coefficient of Friction
Granular A , Till and Gravel	0.40
Sand	0.30
Silt	0.25
Clay	0.25
Bedrock	0.7 to 0.8

The sliding resistance for the different sizes of concrete footings on the bedrock was estimated and summarized in following Table 6.5.

Table 6.5 Sliding Resistance per unit length of the footing

Footing Width	Sliding Resistances (kN/m) ^{&}
0.5 m	36
1.0 m	73
1.5 m	110

[&] the friction coefficient of 0.7 was assumed between bedrock and concrete footing.

6.1.4 Roadway Protection

Roadway protection for this project should be constructed 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.

Since some temporary roadway protection is required during the structure replacement, installation of a gabion wall may be considered to ensure the stability of the bank and is a feasible option. This approach will require widening of road embankment at the location of culvert replacement with temporary culvert extension. Since the bedrock is encountered at shallow depths the use of sheet pile or soldier piles is not considered as an economical option due to difficulty of driving the piles through the bedrock. Alternatively a detour during construction may be considered which will allow full replacement of culvert at same time.

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. 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.

6.1.5 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. 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 "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, λ_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.

$q = \gamma h b \lambda_v$, 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

Due to extremely low compressibility of the bedrock no settlement is expected therefore no issue of downdrag force is expected.

6.1.6 Dewatering

During Construction to prevent water from entering the construction area, a dyke made of sand bags has sometimes been used as a hydraulic barrier. The water passage system should be designed in accordance with OPSD 221.010 "Temporary water passage system – culvert in watercourse". An adequately designed and properly installed pump system will be required to pump creek flow downstream. It should be noted that depending on the season, precipitation, depth of excavation and amount of water flow through the creek dewatering requirements may vary. The contractor should be prepared to tackle this situation. The contractor should be alerted of the high water table and surface water through a non-standard special provision (NSSP).

Where dewatering is performed, all dewatering operations should be completed in accordance with OPSS 517 "Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation". If construction is to be completed in the dry 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, groundwater usage, etc. The

control of water from the dewatering operation should be accordance with OPSS 518 "Construction Specification for Control of Water from Dewatering Operations". However, as the location is underlain by bedrock there will not have any issue of settlement due to increase in effective stress.

Water shall be disposed of so as not to be injurious to public health or safety, property, the environment, fisheries, or any part of the work completed or under construction. Dewatering operations shall be directed to a sediment control device or natural attenuation area prior to discharge to watercourses. If a natural attenuation area is used, a minimum 15 m setback shall be maintained from the receiving watercourse. When water is discharged to a watercourse, the water discharged shall be done in a manner that does not cause erosion or other damage to adjacent lands.

When required, a permit issued by the Ministry of the Environment (MOE) for taking water from a groundwater source shall be obtained. However, as the underlying soils has low to medium hydraulic conductivity, effort required for dewatering may not be substantial provided surface water diversion is managed with proper design.

6.1.7 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".

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".

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".

6.1.8 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.

Bedrock is a non-frost susceptible material in a sound condition. Therefore no requirement for frost depth deformation for foundation purposes.

6.1.9 Embankment Foreslopes

Existing culvert foreslopes are approximately 1.5H: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.

6.1.10 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 bedding and other backfills and ensure the width of trench and trench wall slopes are suitable, and ensure compliance with material placements and compaction methods as well as a gabion wall 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, the proposed replacement structure at this site is recommended to be a open bottom concrete box culvert. Due to presence of shallow bedrock the use of sheet pile wall and soldier piles is not feasible. Since it is a temporary roadway protection, the Gabion wall is considered to be a recommended option however design of roadway protection is the responsibility of the contractor as per the contract drawings.

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:



Deep Bansal, P. Eng
Geotechnical Engineer

Reviewed by:

A handwritten signature in blue ink, appearing to read "Bernardo Villegas".

Bernardo Villegas, M.Sc
Manager

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 'A'
LIMITATIONS OF REPORT

LIMITATIONS OF REPORT

GEOTECHNICAL STUDIES

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 scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the testholes 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 DST Consulting Engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavation, 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 stage 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 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 testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. 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 conclusion 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 DST Consulting Engineers Inc. cannot warranty their accuracy. Similarly, DST cannot warranty the accuracy of information supplied by the client.

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.

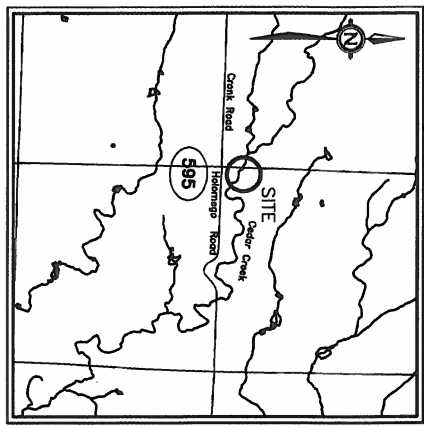
Appendix C

DRAWINGS

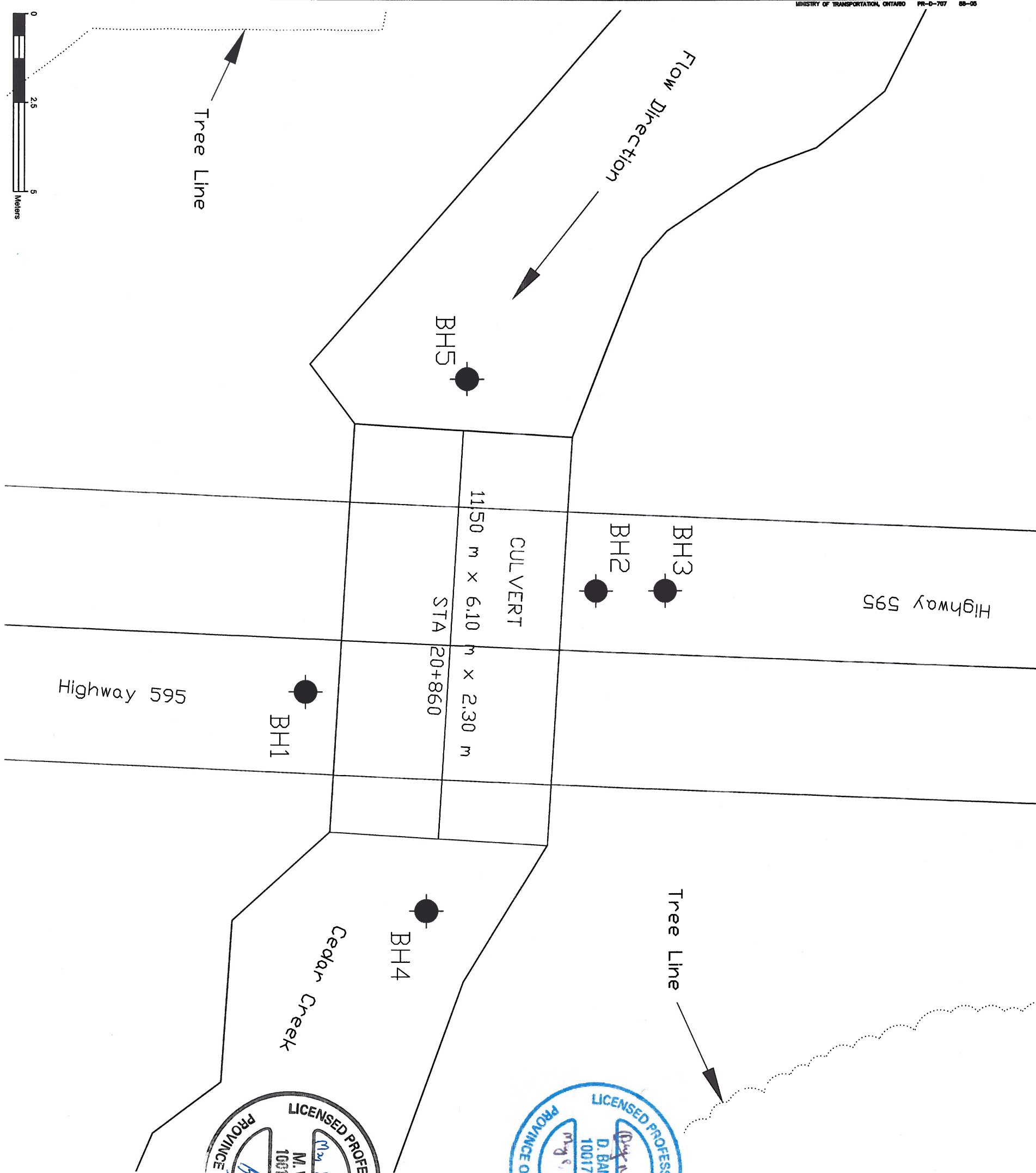
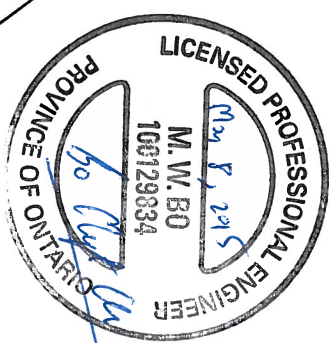
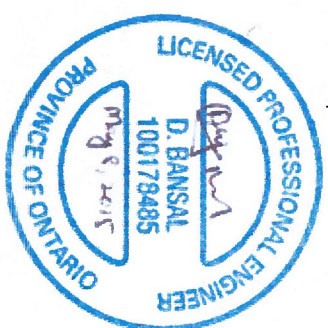
METRIC
DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETERS + METERS

CONT No 6013-E-0023
GWP No 6353-14-00
SITE No 52A-193

CULVERT REPLACEMENT
CEDAR CREEK CULVERT
STA 9+995 TO STA 10+006
Survey 13-06 Revised



KEY PLAN
1.0 km 0 1.0 km



LEGEND

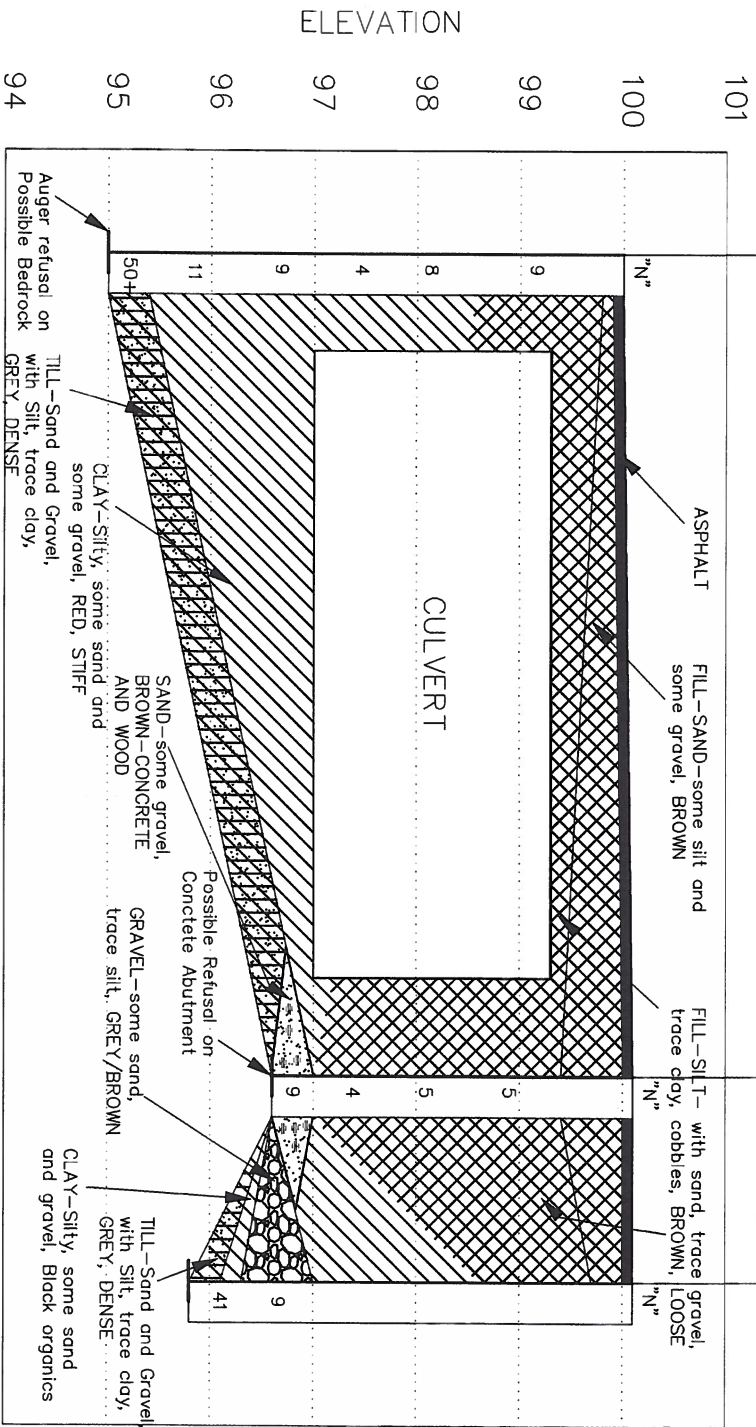
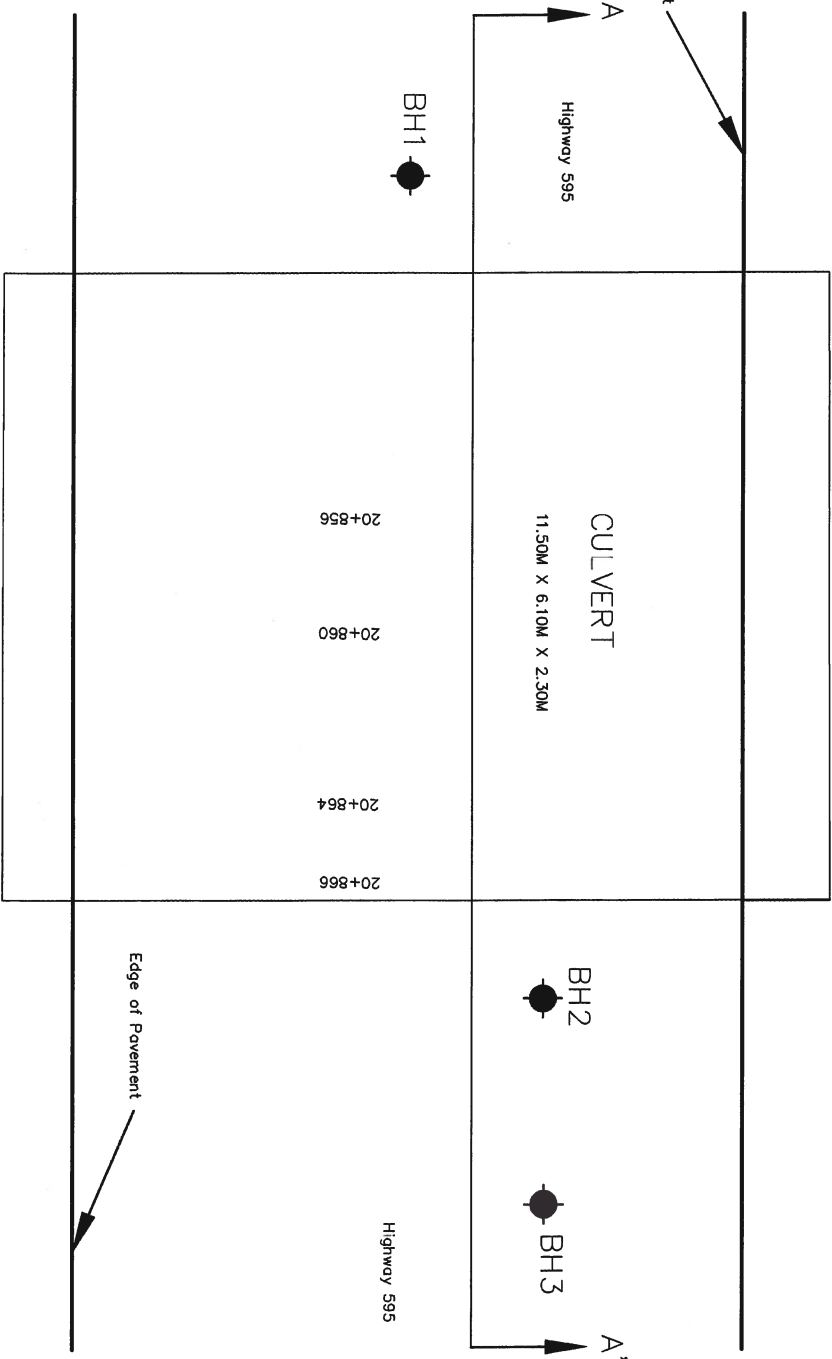
- Borehole
- Borehole with CPT
- Asphalt Core
- Rock Probe
- Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Water level at time of investigation.
- Fill
- Organics
- Topsoil
- Till
- Bedrock
- Sand
- Silt
- Clay
- Sand & Gravel
- Boulders

No.	Elevation	Northing	Easting	Station	Offset
BH1	100.0	5362627 m N	300041 m E	20+865	1.5 m RT
BH2	100.1	5362624 m N	300035 m E	20+864	1.7 m LT
BH3	100.1	5362633 m N	300037 m E	20+866	1.7 m LT
BH4	95.0	5362624 m N	300032 m E	20+860	7.5 m RT
BH5	95.3	5362632 m N	300023 m E	20+860	7.5 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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Edge of Pavement



PROFILE ALONG SECTION A-A'



METRIC

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

CONT No 6353-14-00

GWP No 52A-193

GEORECS No 52A-193



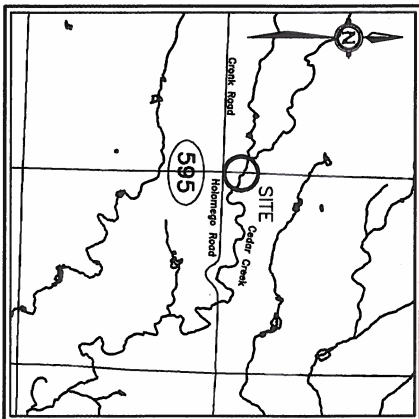
CULVERT REPLACEMENT

CEDAR CREEK CULVERT

STA 20+856 TO STA 20+866

Survey 13-06 Revised

SHEET



LEGEND

◆	Borehole	◆	Sand
●	Borehole with CPT	◆	Silt
●	Asphalt Core	◆	Clay
●	Rock Probe	◆	Sand & Gravel
◆	Blows/0.3m (Std. Pen Test, 475 J/Blow)	◆	Boulders
◆	Water level at time of investigation.		
◆	Fill		
◆	Organics		
◆	Topsoil		
◆	Till		
◆	Bedrock		

No.	Elevation	Nothing	Easting	Station	Offset
BH1	100.0	5382627 m N	300041 m E	20+856	1.5 m RT
BH2	100.1	5382642 m N	300035 m E	20+864	1.7 m LT
BH3	100.1	5382638 m N	300037 m E	20+866	1.7 m LT
BH4	95.0	5382624 m N	300052 m E	20+860	7.5 m RT
BH5	95.3	5382632 m N	300023 m E	20+860	7.5 m LT

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

NOTE: Drawings and survey completed by DST.

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DRAWING 2

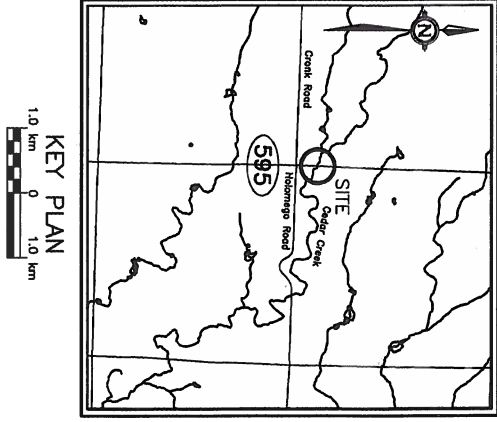
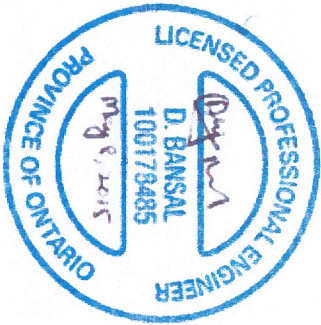
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETERS + METERS

CONT No
GWP No 6353-14-00
GEORECS No 52-193



CULVERT REPLACEMENT
CEDAR CREEK CULVERT
STA 20+856 TO STA 20+866
Survey 13-06 Revised

SHEET



LEGEND

- Borehole
- Borehole with CPT
- Asphalt Core
- Rock Probe
- Blows/0.3m (Std. Pen Test, 475 J/Blow)
- Water level at time of investigation.

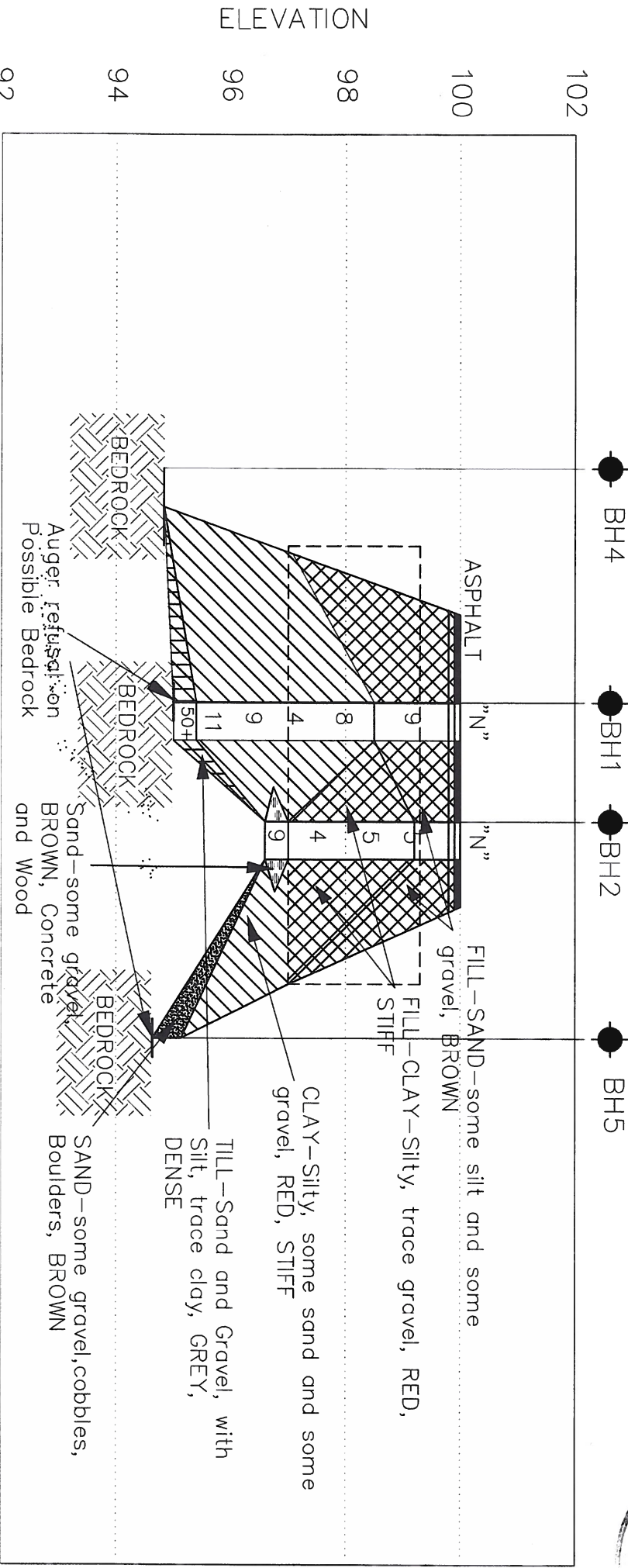
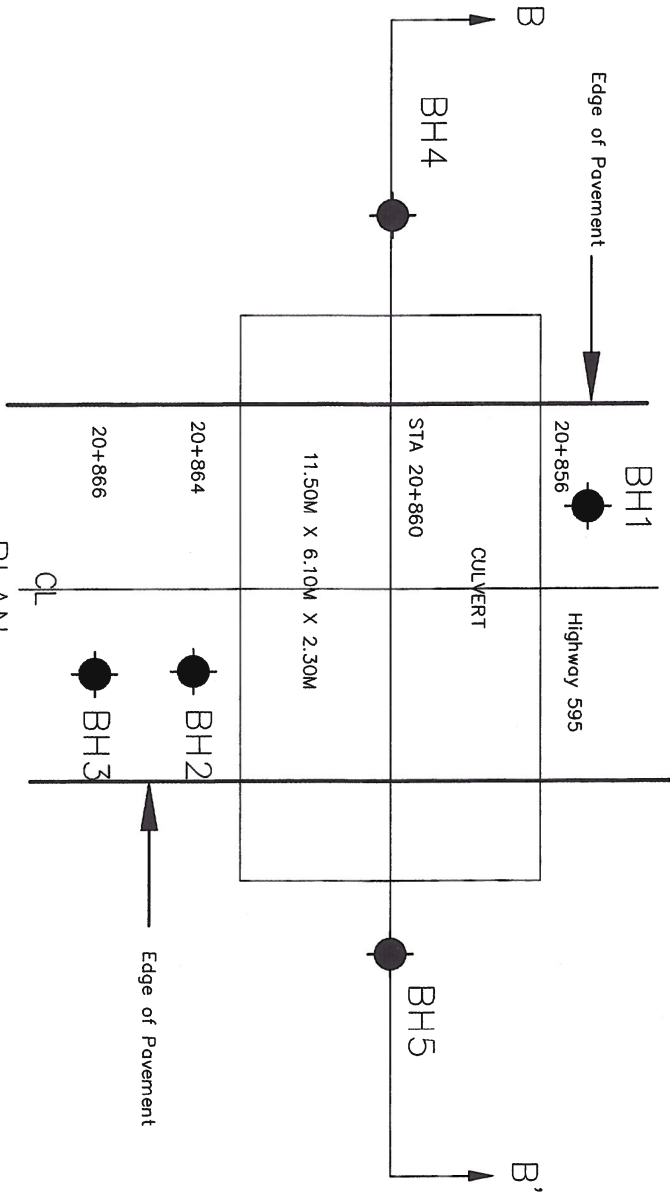
- Fill
- Organics
- Topsoil
- Till
- Bedrock
- Sand
- Silt
- Clay
- Sand & Gravel
- Boulders

No.	Elevation	Nothing	Easting	Station	Offset
BH1	100.0	5362627 m N	300041 m E	20+856	1.5 m RT
BH2	100.1	5362642 m N	300035 m E	20+864	1.7 m LT
BH3	100.1	5362638 m N	300037 m E	20+866	1.7 m LT
BH4	95.0	5362624 m N	300052 m E	20+860	7.5 m RT
BH5	95.3	5362632 m N	300023 m E	20+860	7.5 m LT

NOTE:
Boundaries between soil types 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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NOTE:
Drawing and survey completed by DST.



PROFILE ALONG SECTION B-B'



Appendix D
ENCLOSURES

RECORD OF BOREHOLE No BH1

1 OF 1

METRIC

W.P. 6013-E-0021 LOCATION Cedar Creek Culvert STA 20+856, 1.5 RT ORIGINATED BY PR
DIST HWY 595 BOREHOLE TYPE Hollow Stem Auger (80 mm ID) COMPILED BY MD
DATUM LOCAL DATE 2014 08 20 CHECKED BY DM

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							
100.0	GROUND SURFACE							20	40	60	80	100			
99.9	ASPHALT		AS1	AS											
99.3	FILL-SAND & CRUSHED GRAVEL- Trace silt														
0.8	FILL-SILT- with sand, trace gravel, trace clay, cobbles, BROWN, LOOSE		SS2	SS	9		99								
98.5	CLAY-Silty, some sand and some gravel, RED, STIFF														
1.5			SS3	SS	8		98								2 36 52 10
			SS4	SS	4										
							97								
			SS5	SS	9										
			SS6	SS	11		96								
95.4	TILL- Sand and gravel, with silt, trace clay, GREY, DENSE														44 36 (20)
95.0			SS7	SS	50+		95								
5.0	END OF BOREHOLE Auger Refusal on Possible Bedrock														

NR = NO RECOVERY

+³, X³: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

ENCLOSURE 1

RECORD OF BOREHOLE No BH2

1 OF 1

METRIC

W.P. 6013-E-0021 LOCATION Cedar Creek Culvert 595 STA 20+864, 1.7 LT ORIGINATED BY PR
DIST HWY 595 BOREHOLE TYPE Hollow Stem Auger (80 mm ID) COMPILED BY MD
DATUM LOCAL DATE 2014 08 20 CHECKED BY DM

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					
100.1	GROUND SURFACE																
99.9	ASPHALT		AS1	AS													
0.2	FILL-SAND-some silt and some gravel, BROWN																
99.4																	
0.8	FILL-CLAY-silty, trace gravel, RED, STIFF		SS2	SS	5												
			SS3	SS	5												
			SS4	SS	4												
97.0																	
3.1	SAND- some gravel, BROWN		SS5	SS	9												
96.6	-CONCRETE AND WOOD																
3.5	END OF BOREHOLE Possible Refusal on Concrete Abutment																

NR = NO RECOVERY

+³, X³: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

ENCLOSURE 2

ONL MOT GS-TB-019503_BH_LOGS_CEDAR_CREEK.GPJ DST_MIN.GDT 1/9/15

RECORD OF BOREHOLE No BH3

1 OF 1

METRIC

W.P. 6013-E-0021 LOCATION Cedar Creek Culvert 595 STA 20+866, 1.7 LT ORIGINATED BY PR
DIST HWY 595 BOREHOLE TYPE Hollow Stem Auger (80 mm ID) COMPILED BY MD
DATUM LOCAL DATE 2014 08 20 CHECKED BY DM

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 (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa														
								○ UNCONFINED	+ FIELD VANE	□ QUICK TRIAXIAL	× LAB VANE						WATER CONTENT (%)					
100.1	GROUND SURFACE						20	40	60	80	100					GR	SA	SI	CL			
100.0	ASPHALT		AS1	AS																		
99.7	FILL-SAND-some gravel, some silt, BROWN																					
0.5	FILL-SILT- with sand, some gravel,trace clay, BROWN/REDDISH		AS2	AS																		
98.6	CLAY-Silty-trace gravel, RED, STIFF		AS3	AS																		
1.5																						
			AS4	AS																		
97.0	GRAVEL-some sand, trace silt, GREY/BROWN, Loose																					
3.1			SS5	SS	9																	
96.3																						
96.8	CLAY-Silty, some sand and gravel, Black organics		SS6	SS	41																	
4.0	TILL- Sand and gravel, with silt, trace clay																					
95.8	END OF BOREHOLE Auger Refusal on Possible Bedrock																					
4.3																						

NR = NO RECOVERY

+³, X³: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

ENCLOSURE 3

ONL MOT GS-TB-019503_BH_LOGS_CEDAR_CREEK.GPJ DST_MIN.GDT 1/9/15

RECORD OF BOREHOLE No BH4

1 OF 1

METRIC

W.P. 6013-E-0021 LOCATION Cedar Creek Culvert 595 STA 20+860, 7.5 RT ORIGINATED BY PR
 DIST HWY 595 BOREHOLE TYPE Hollow Stem Auger (80 mm ID) COMPILED BY MD
 DATUM LOCAL DATE 2014 08 21 CHECKED BY DM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
95.0	GROUND SURFACE						20	40	60	80	100	20	40	60		
	BEDROCK ON SURFACE END OF BOREHOLE															

ONL MOT GS-TB-019503_BH_LOGS_CEDAR_CREEK.GPJ DST_MIN.GDT 1/9/15

NR = NO RECOVERY +³, X³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ENCLOSURE 4

RECORD OF BOREHOLE No BH5

1 OF 1

METRIC

W.P. 6013-E-0021 LOCATION Cedar Creek Culvert 595 STA 20+860, 7.0 LT ORIGINATED BY PR
 DIST HWY 595 BOREHOLE TYPE Hollow Stem Auger (80 mm ID) COMPILED BY MD
 DATUM LOCAL DATE 2014 08 21 CHECKED BY DM

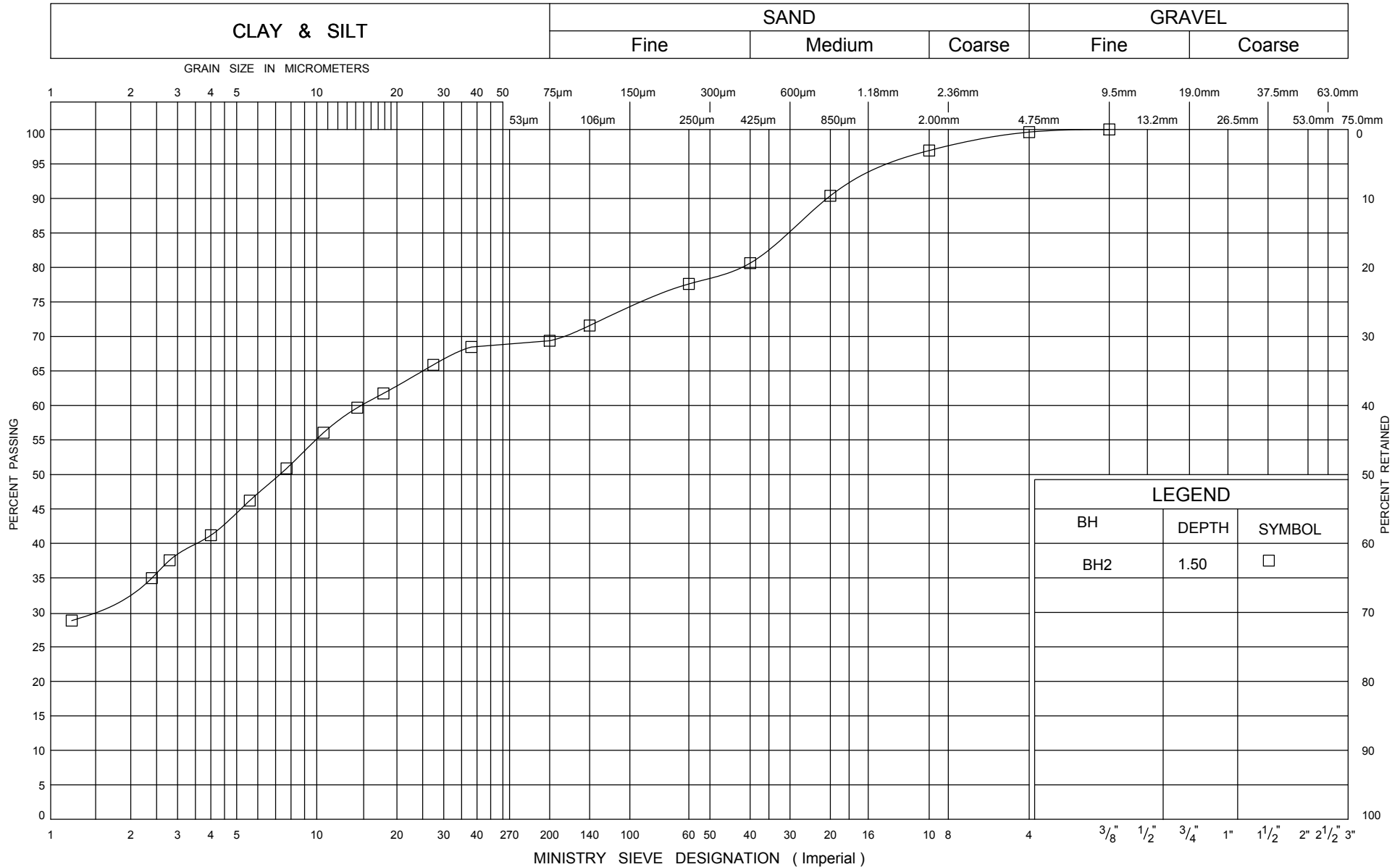
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100						
95.3	GROUND SURFACE																
94.8	SAND- some gravel, cobbles, boulders, BROWN																
0.5	END OF BOREHOLE Auger Refusal on Possible Bedrock																

ONL MOT GS-TB-019503_BH_LOGS_CEDAR_CREEK.GPJ DST_MIN.GDT 1/9/15

NR = NO RECOVERY +³, X³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

ONTARIO MOT GRAIN SIZE GS-TB-019503_BH_LOGS_CEDAR_CREEK.GPJ DST_MIN.GDT 11/25/14

UNIFIED SOIL CLASSIFICATION SYSTEM



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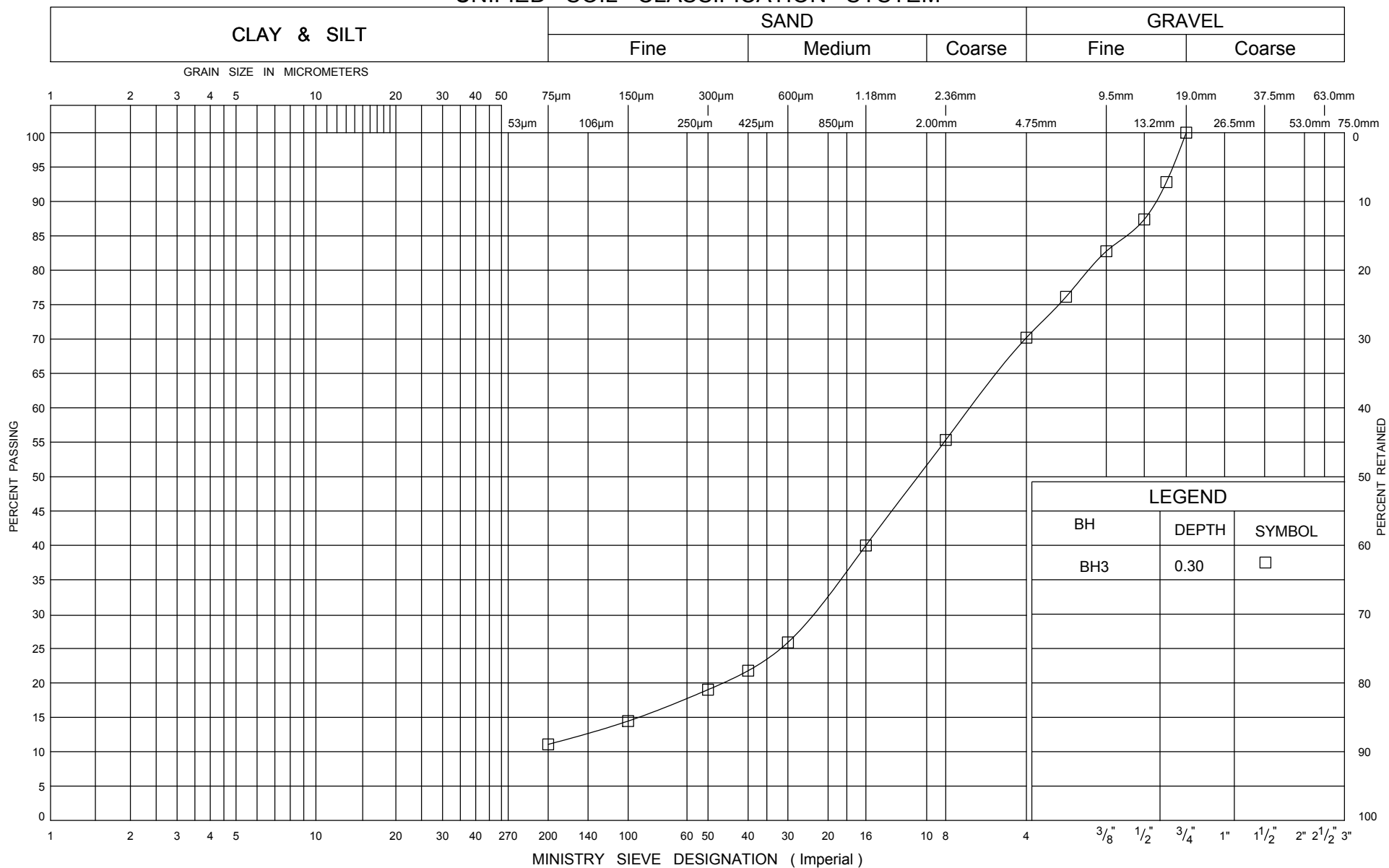
GRAIN SIZE DISTRIBUTION FILL-CLAY

ENCLOSURE 6

W P 6013-E-0021

HWY 595

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GRAIN SIZE DISTRIBUTION

FILL-Sand

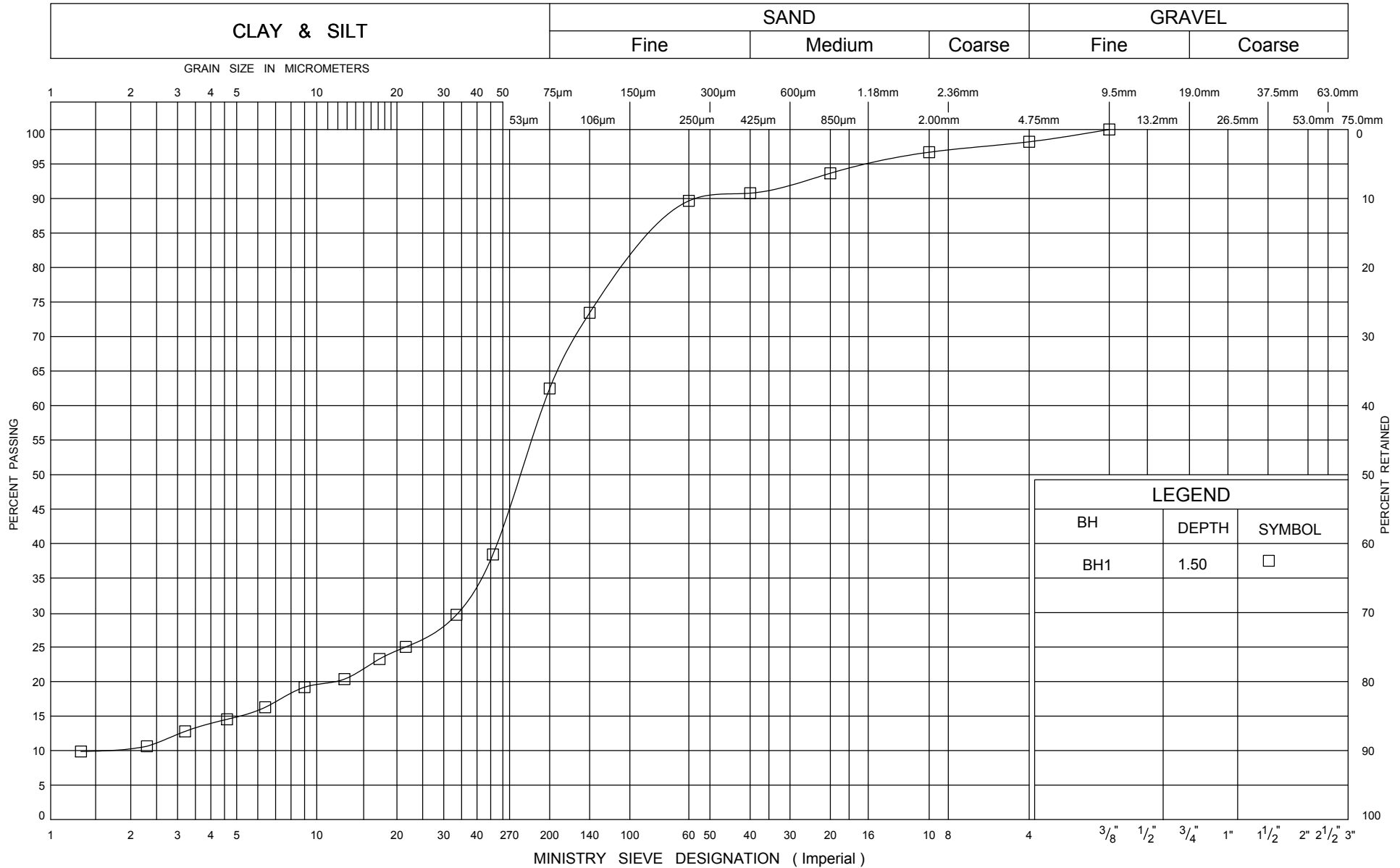
ENCLOSURE 7

W P 6013-E-0021

HWY 595

ONTARIO MOT GRAIN SIZE GS-TB-019503_BH_LOGS_CEDAR_CREEK.GPJ DST_MIN.GDT 11/25/14

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GRAIN SIZE DISTRIBUTION FILL-SILT

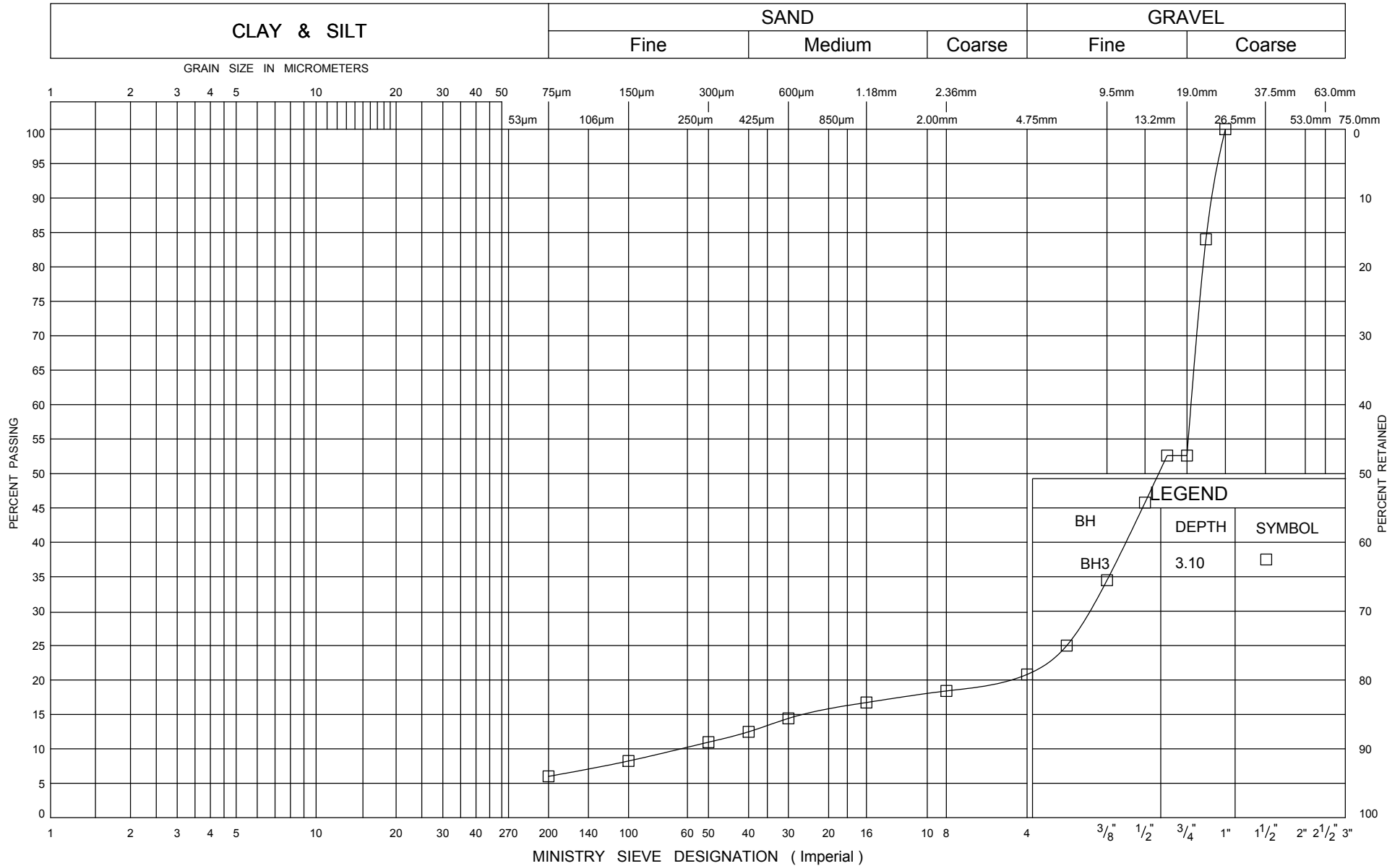
ENCLOSURE 8

W P 6013-E-0021

HWY 595

ONTARIO MOT GRAIN SIZE GS-TB-019503_BH_LOGS_CEDAR_CREEK.GPJ DST_MIN.GDT 11/25/14

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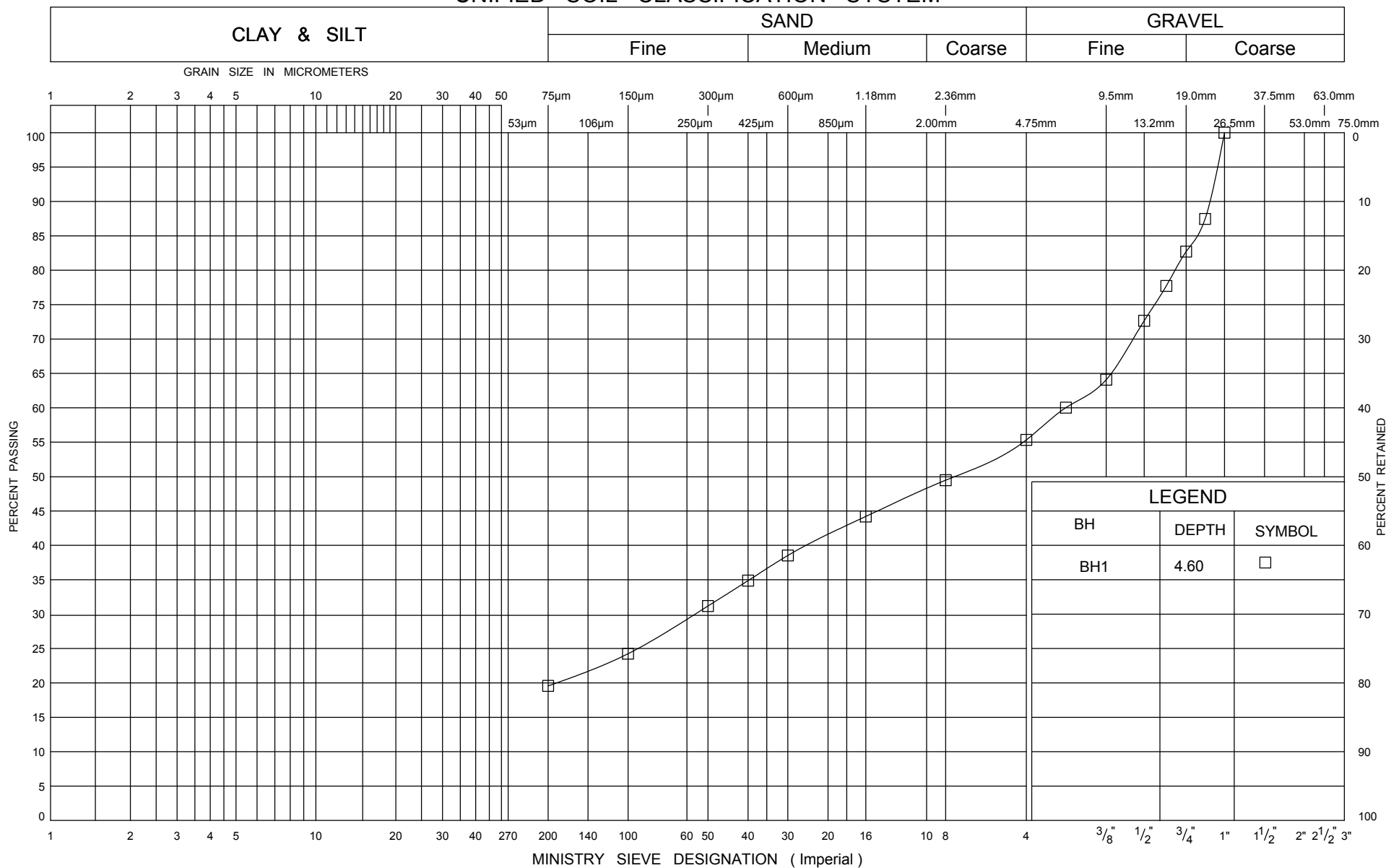
GRAIN SIZE DISTRIBUTION GRAVEL

ENCLOSURE 9

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HWY 595

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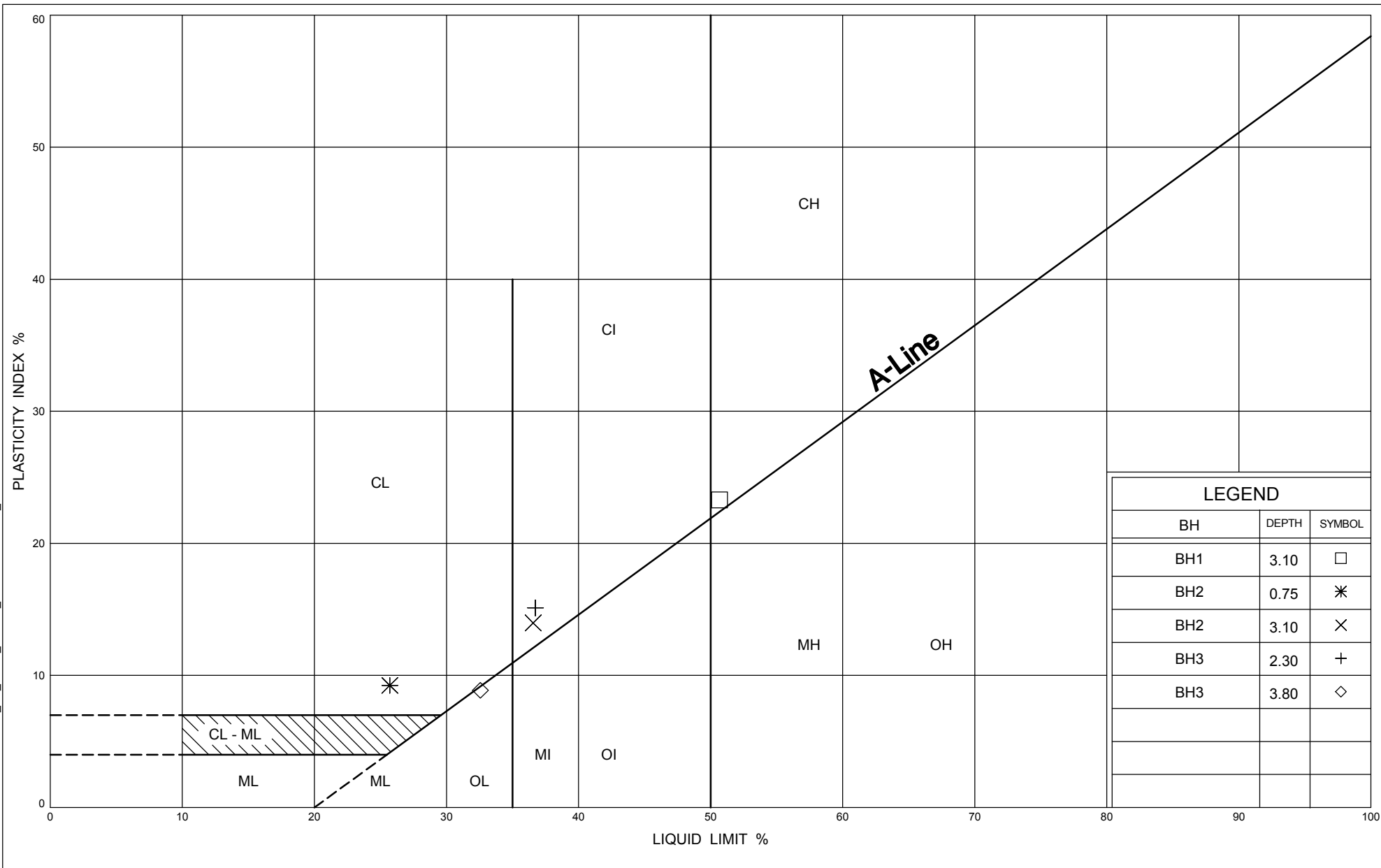
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GRAIN SIZE DISTRIBUTION TILL

ENCLOSURE 10

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HWY 595



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PLASTICITY CHART CLAY-Silty

ENCLOSURE 11

W P 6013-E-0021

HWY 595