



**FOUNDATION INVESTIGATION AND DESIGN REPORT
LITTLE GRASSY TRIBUTARY CULVERT, OLSON ROAD
75 M NORTH-WEST OF HIGHWAY 621
PRATT TOWNSHIP
AGREEMENT NO.: 5014-E-0049
SITE NO.: 45-282/C
GEOCRES NO. 52D-22
GWP 6065-13-00**

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

DST Consulting Engineers Inc. (DST) has been retained by the prime consultant, Ainley Group, to conduct a foundation investigation and design report for the proposed culvert replacement on Olson Road 75 m North-West of Highway 621. This work was carried out under Agreement No.: 5014-E-0049. This report addresses the field investigation, laboratory test program, factual report on conditions (Part 1) and recommendations for design and construction for the proposed culvert replacement (Part 2).

2. SITE DESCRIPTION

The site is located on Olson Road, approximately 75 m North-West of Highway 621 (Latitude 48.912234, Longitude -94.409477), in the Township of Pratt.

It is understood that the existing structure is a triple cell timber box culvert with an overall span of 14 m, width of 4.8 m, minimum clearance of 1.51 m, and 2° skew. This culvert currently carries one lane of rural northwest-southeast traffic. The existing box culvert deck is in a fair to poor condition with the timber spans sagging and separating. The timber barrel soffit walls are in a fair to poor condition exhibiting checking, splitting, leaning towards the west, and the timbers separating. The inlet and outlets of the culvert are also in a fair to poor condition with the northwest and northeast wing walls rotated and separated respectively. The existing timber bridge can be seen in (Figure 2-1 to 2-2). Inspection of the triple cell timber culvert was conducted by others.

Geological information is available from published *Ontario Geological Survey Map #52DNE* by the *Ontario Ministry of Natural Resources* for the Stratton area. The map indicates that the local area landform is identified as glaciolacustrine plains. Sediments in glaciolacustrine plains consist of varved and massive, fine grained materials deposited in glacial lake basins of varying size and

depth. The proportions of clay, silt, and sand deposited at any particular location in these basins varies with depth of water in the former lake, distance from former shorelines, and the size of particles washed into the lake. Most clay, silt, and sand lacustrine deposits contain minor inclusions of till and scattered dropstones which were drafted into the lake on pieces of ice. In places, wave and current action in former glacial lakes eroded the surface of till deposits, producing thin patches of washed sand, gravel and boulders that rest on till or bedrock. In other areas, bedrock knobs and ridges are surrounded by pockets of glaciolacustrine sediment. Glacial lake deposits usually consist of clay and silt which accumulated in deep offshore waters at a depth where the bottom was no longer affected by wave action (generally below 10 m). Closer to shore, and at points where rivers discharged sandy materials into the lake, the deposits usually consist of fine and medium sand with minor silt. Most of these sandy lacustrine deposits accumulated in deltaic environments.



Figure 2-1 Location of existing culvert Olson Road (downstream)



Figure 2-2 Location of existing culvert Olson Road (looking North-West)

3. INVESTIGATION PROCEDURES AND LABORATORY TESTING

Site work was carried out on August 29th to September 9th, 2015 utilizing a CME 750 drill rig equipped for geotechnical drilling. A total of six boreholes were advanced to depths ranging from 10.7 m to 18.3 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 1 and 2 in Appendix C. Borehole 1 was advanced 2.6 m North of the existing bridge and 2.0 m West of outlet to a depth of 17.7 m below the existing surface. Borehole 2 was advanced 5.0 m North of the bridge and 2.2 m East of the bridge to a depth of 12.5 m below the existing surface. Borehole 3 was advanced 15 m North of the bridge and 3.0 m West of outlet to a depth of 11.9 m below the existing surface. Borehole 4 was advanced 2.0 m South of the bridge and 2.0 m East of inlet to a depth of 18.3 m below the existing surface. Borehole 5 was advanced 5.0 m South of the bridge and 0.6 m East of outlet to a depth of 12.2 m below the existing surface. Borehole 6 was advanced 15 m South of the bridge and 2.0 m East of inlet to a depth of 10.7 m below the existing surface.

The ground surface elevations at the borehole locations were surveyed by DST personnel and referenced to benchmark 328.565 m (N = 5420262.7, E = 201491.8) as indicated on the

drawings provided by the Ministry. 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. 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. In addition, in-situ vane shear testing were performed in cohesive soils at selected depths using an MTO vane (65 mm x 150 mm). 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 analyses.

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, Atterberg limits including plastic limit and liquid limit, unconfined compression tests and chemical tests. A total of forty four (44) moisture contents, five (5) particle size analyses, twelve (12) Atterberg limits, five (5) point load tests, and a set of chemical tests have been carried out 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 Location

Borehole ID	Station	Elevation (m)	Depth (m)
BH1	10 + 090	328.6	17.7
BH2	10 + 093	328.6	12.5
BH3	10 + 102	328.4	11.9
BH4	10 + 078	328.6	18.3
BH5	10 + 076	329.1	12.2
BH6	10 + 064	328.7	10.7

4. DESCRIPTION OF SUBSURFACE CONDITIONS

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

The generalized stratigraphy of the existing embankment and approaches at the existing culvert, based on the conditions encountered in the boreholes consists of sand and gravel fill at surface underlain by silty clay which further overlies various deposits of sand, silty clay, clayey sand, sand and gravel, and cobbles and boulders. Topsoil was encountered in Borehole 5. Bedrock was encountered in Boreholes 1 and 4. Summary of the soil strata is provided in Table 4-1.

Table 4-1 Summary of soil strata at the existing bridge

Layer	Depth (m)	Elevation (m)	Comments
Topsoil	0.0 to 0.2	329.1 to 328.9	BH5
Fill - Sand and Gravel	0.0 to 1.0	328.6 to 327.6	BH1
	0.0 to 0.8	328.6 to 327.8	BH2
	0.0 to 0.8	328.4 to 327.6	BH3
	0.0 to 0.6	328.7 to 328.1	BH6
Fill – Silty Sand	0.0 to 0.8	328.6 to 327.8	BH4
Sandy Silty Clay	0.2 to 10.7	328.9 to 318.4	BH5
Silty Clay	1.0 to 9.1	327.6 to 319.5	BH1
	0.8 to 6.1	327.8 to 322.5	BH2
	0.8 to 11.9	327.6 to 316.5	BH3
	0.8 to 9.7	327.8 to 318.9	BH4
	0.6 to 9.8	328.1 to 318.9	BH6
Sand	6.1 to 12.5	322.5 to 316.1	BH2
Silt and Sand	9.1 to 13.1	319.5 to 315.5	BH1
Silty Sand	9.8 to 10.7	318.9 to 318.0	BH6
Clayey Sand	9.7 to 12.2	318.9 to 316.4	BH4
Sand and Gravel	10.7 to 12.2	318.4 to 316.9	BH5
Cobbles and Boulders	12.2 to 13.7	316.4 to 314.9	BH4
Bedrock	13.1 to 17.7	315.5 to 310.9	BH1
	13.7 to 18.3	314.9 to 310.3	BH4

4.1 Topsoil

Topsoil was encountered in Borehole 5 (Elev. 329.1 to 328.9 m) with a thickness of 0.2 m.

4.2 Fill – Sand and Gravel

Sand and gravel fill with trace silt was encountered in Boreholes 1, 2, 3, and 6 at surface at depths 0.0 to 1.0 m (Elev. 328.6 to 327.6 m), 0.0 to 0.8 m (Elev. 328.6 to 327.8 m), 0.0 to 0.8 m (Elev. 328.4 to 327.6 m), and 0.0 to 0.6 m (Elev. 328.7 to 328.1m) with thicknesses of 1.0 m, 0.8 m, 0.8 m, and 0.6 m respectively.

The SPT 'N' values vary from 10 to 25, indicating a compact condition. The moisture contents of the sand and gravel fill vary from 3 to 8 %.

4.3 Fill – Silty Sand

Silty sand with some gravel fill was encountered at the surface of Borehole 4 at the depth from 0.0 to 0.8 m (Elev. 328.6 to 327.8 m) with a thickness of 0.8 m.

The SPT 'N' value was 17, indicating a compact condition. The moisture contents of a selected sample tested was 9 %. The laboratory test results are summarized in Table 4-2.

Table 4-2: Summary of Sieve Analysis- Fill – Sand

Laboratory Results – Sieve Analysis	
Gravel %	13
Sand %	54
Fines %	33

4.4 Sandy Silty Clay

Various portions of sand, silt and clay with some to trace gravel was encountered below the topsoil in Borehole 5 at the depth from 0.2 to 10.7 m (Elev. 328.9 to 318.4 m) with a thickness of 10.5 m.

Atterberg limits tests carried out on samples from Boreholes indicate that the sandy silty clay has medium to high plasticity. Field vane test completed in the borehole show shear strength of 42 kPa indicating a firm consistency. The SPT 'N' values vary from 2 to 15. The moisture content of the clay ranges from 16 to 53 %. The laboratory test results are summarized in the following Tables 4-3 and 4-4.

Table 4-3 Summary of Atterberg Limits- Sandy Silty Clay

Laboratory Results – Atterberg Limits	
Liquid Limit %	49 to 55
Plastic Limit %	18 to 29
Plastic Index %	26 to 31

Table 4-4: Summary of Sieve Analysis- Sandy silty clay

Laboratory Results – Sieve Analysis	
Gravel %	8 to 11
Sand %	38 to 41
Fines %	48 to 54

4.5 Silty Clay

Silty clay was encountered below the sand and gravel fill in Borehole 1,2,3,4, and 6 at the depths from 1.0 to 9.1 m (Elev. 327.6 to 319.5 m), 0.8 to 6.1 m (Elev. 327.8 to 322.5 m), 0.8 to 11.9 m (Elev. 327.6 to 316.5 m), 0.8 to 12.2 m (Elev. 327.8 to 316.4 m), and 0.6 to 9.8 m (Elev. 328.1 to 318.9 m) with a thicknesses of 8.1m, 5.3 m, 11.1 m, 11.4 m, and 9.2 m respectively.

Atterberg limits tests carried out on samples from Boreholes indicate that the clay has medium to high plasticity. Field vane tests completed in Boreholes show shear strength between 48 and 132 kPa indicating a firm to very stiff consistency. The SPT 'N' values vary from 1 to 16. The moisture content of the clay ranges from 13 to 64 %. The laboratory test results are summarized in following Tables 4-5.

Table 4-5: Summary of Atterberg Limits- Silty Clay

Laboratory Results – Atterberg Limits	
Liquid Limit %	41 to 78
Plastic Limit %	17 to 27
Plastic Index %	20 to 57

4.6 Sand

Sand with to some gravel and some fines was encountered below the silty clay in Boreholes 2, and below the sandy silty clay in Borehole 5 at the depth from 6.1 to 12.5 m (Elev. 322.5 to 316.1 m) and 10.7 to 12.2 m (Elev. 318.4 to 316.9 m) with thicknesses of 6.4 m and 1.5 m respectively.

The SPT 'N' values vary from 7 to 100+, indicating a loose to very dense condition. The moisture contents of a selected samples tested range from 17 to 18 %. The laboratory test results are summarized in Table 4-6.

Table 4-6 Summary of Sieve Analysis- Sand

Laboratory Results – Sieve Analysis	
Gravel %	10 to 27
Sand %	62 to 90
Fines %	4 to 11

4.7 Silt and Sand

Silt and sand with trace gravel was encountered below the silty clay in Borehole 1 at the depth from 9.1 to 13.1 m (Elev. 319.5 to 315.5 m) with a thickness of 4.0 m.

The SPT 'N' values vary from 6 to 100+, indicating a loose to very dense condition. The moisture contents of a selected sample tested was 34 %. The laboratory test results are summarized in Table 4-7.

Table 4-7: Summary of Sieve Analysis- Silt and Sand

Laboratory Results – Sieve Analysis	
Gravel %	3
Sand %	46
Fines %	51

4.8 Silty Sand

Silty sand was encountered below the silty clay in Boreholes 6 at the depth 9.8 to 10.7 m (Elev. 318.9 to 318.0 m) with a thickness of 0.9 m.

The SPT 'N' value 100+, indicating a very dense condition. The moisture contents of a selected sample tested was 21 %.

4.9 Clayey Sand

Clayey sand with some gravel was encountered below the silty clay in Borehole 4 at the depth from 9.7 to 12.2 m (Elev. 318.9 to 316.4 m) with a thickness of 2.5 m.

The SPT 'N' values was 20, indicating a compact condition. The moisture contents of a selected sample tested was 19 %.

4.10 Sand and Gravel

Sand and gravel with trace silt was encountered below the sandy silty clay in Borehole 5 at the depth from 10.7 to 12.2 m (Elev. 318.4 to 316.9 m) with a thickness of 1.5 m.

The SPT 'N' values vary from 18 to 50+, indicating a compact to very dense condition. The moisture content of a selected sample tested was 15 %.

4.11 Cobbles and Boulders

Cobbles and boulders was encountered in Borehole 4 at the depth from 12.2 to 13.7 m (Elev. 316.4 to 314.9 m) with a thickness of 1.5 m.

4.12 Bedrock

Bedrock was encountered in two (2) Boreholes, Borehole 1 and 4 at depth below 13.1 m (Elev. 315.5 m) and 13.7 m (Elev. 314.9 m) respectively.

The type of Bedrock is a metamorphic rock Gneiss diorite based on our visual classification of rock cores and this is also consistent with our review of surficial bedrock geology of the area. This is a rock composed mainly of translucent white plagioclase and dark hornblende with a little biotite giving it a grey colour. The appearance of the stone varies from dark (hornblende and biotite make up almost 50 %) to very light (5 % hornblende). The cores recovered had no signs of physical, chemical or biological weathering. The rock cores can be seen in Figures 4-1 and 4-2.



Figure 4-1 Little Grassy River rock core Borehole 1



Figure 4-2 Little Grassy River rock core Borehole 4

In Borehole 1, Total Core Recovery (TCR) and Solid Core Recovery (SCR) were 85 % to 100%. Rock Quality Designation (RQD) was found between 41% and 73% indicating Poor to Fair

Rock. In Borehole 4, Total Core Recovery (TCR) and Solid Core Recovery (SCR) were 92% to 98%. Rock Quality Designation (RQD) was found between 0% and 66% indicating a Very Poor to Fair Rock. The unconfined compressive strength of rock, derived from the Point Load Test, varies between 53 to 67 MPa at Borehole 1 and 30 to 136 MPa at Borehole 4 indicating an Average to Very strong rock.

4.13 Groundwater

At the time of the field investigation groundwater was observed in Boreholes 1 and 6. The groundwater levels can be expected to vary with the season and precipitation events.

Table 4-8: Groundwater depth

Borehole	Groundwater Depth (m)	Groundwater Elev. (m)
Borehole 1	0.9	327.7
Borehole 6	1.5	327.2

4.14 Chemical Test Results

Selected soil samples were submitted to ALS Laboratories Thunder Bay for chemical analyses (pH, sulphate, conductivity, resistivity and Chloride) to assess the potential for corrosion and sulphate attack on buried structures. The conductivity and resistivity of samples are determined in the lab by tumbling soil samples in de-ionized (DI) water at the ratio of 2:1 water to soil. After tumbling the sample is analysed by conductivity meter to determine the conductivity. The inverse of the conductivity is taken and calculated for resistivity.

The results are presented below in Table 4-9 and a copy of the Laboratory Certificate of Analysis is provided in **Appendix D**.

Table 4-9: Chemical Test Results

Sample ID	Moisture (%)	Sulphate (mg/kg)	Chloride (mg/kg)	pH	Conductivity (umhos/cm)	Resistivity (ohm - cm)
BH1 @ 2.7 m depth	27.3	84	<20	7.19	215	4200

The analytical results of the soil samples were compared with applicable Canadian Standards Association (CSA) standards as shown in **Table 4-10** below

The chemical sulphate content analyses for representative soil sample tested indicate a sulphate concentration of 84 mg/kg or 0.0084 % in soil. The results were compared with Canadian

Standards Association (CSA) Standards A23.1 for sulphate attack potential on concrete structures and possess a “negligible” risk for sulphate attack on concrete material and accordingly, conventional GU or MS Portland cement may be used in the construction of the proposed concrete elements.

The pH value for the soil samples was reported to be 7.19, indicating a durable condition against corrosion. These results were evaluated using Table 2 of Building Research Establishment (BRE) Digest 363 (July 1991). The pH is greater than 5.5 indicating the concrete will not be exposed to attack from acids. The chloride content of the selected soil sample was also compared with the threshold level and present negligible concrete corrosion potential. Soil resistivity and conductivity was found to be 4200 ohm-cm and 215 umhos / cm respectively for the sample analysed from BH1.

Table 4-10 Additional requirements for concrete subjected to sulphate attack

Class of Exposer	Degree of Exposer	Water soluble Sulphate in soil sample (%)	Cementing Material to be used
S-1	Very Severe	> 2.0	HS or HSb
S-2	Severe	0.20 – 2.0	HS or HSb
S-3	Moderate	0.10 – 0.20	MS, MSb, LH, HS, or HSb

* Information from Table 3 of CSA Standards A23.1-04

5. MISCELLANEOUS

Site work was carried out during the week of August 29th to September 9th, 2015 utilizing a CME 750 all-terrain drill rig operated by DST. Fieldwork was supervised on a full time basis by DST personnel. 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 Selorm Danku, P.Eng and reviewed by Dr. Masud Karim, P.Eng who is the designated principal contact for MTO projects.

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PART 2: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

6. PROJECT DESCRIPTION

DST Consulting Engineers Inc. (DST) has been retained by the prime consultant, Ainley Group, to conduct a foundation investigation and design report for the proposed culvert replacement on Olson Road 75 m North-West of Highway 621. This work was carried out under Agreement No.: 5014-E-0049, Replacement of three local roads board structures in the Northwest Region.

Existing structure at this location is triple cell timber box culvert with overall span of 9 m, width of 4.8 m, minimum clearance of 1.51 m, and 2° skew. Existing structure carries one lane traffic and poor to fair condition with the timber spans sagging and separating. The timber barrel soffit walls are in a poor to fair condition exhibiting checking, splitting, leaning towards the West and timbers are separating. Inlet and outlet of the culvert are in poor to fair condition with Northwest and Northeast wings walls rotated and separated.

The generalized stratigraphy of the existing embankment, based on the conditions encountered in boreholes, consists of sand & gravel fill overlying silty clay that is underlain by sand & gravel or silty sand before encountering the bedrock formation.

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

6.1 Replacement Structure

Three structure replacement options (corrugated steel plate (CSP) culvert, concrete box culvert, pre-cast concrete open footings and bridge with pile foundation) have been discussed for the replacement structure. Concrete box culvert, Pre-cast concrete shallow footing and deep foundation were further discussed as the potential options.

The design of the replacement structure must be in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC, 2010) and all relevant Ministry of Transportation

specifications and guidelines.

6.2 Structure Foundations Types Assessment

6.2.1 Geotechnical Design Parameters

The following parameters in Table 6-1 are adopted as design parameters for each formation of the soils. Based on the in-situ and laboratory tests carried out undrained shear strengths were directly estimated from field vane shear strengths measured on site. Drained internal friction angles for the clay were estimated from the plasticity index (Kenney, 1959). Drained internal friction angles for the granular soils were estimated from standard penetration tests applying empirical correlations developed by Wolff (1989).

Table 6-1 Soil Parameters for Foundation Analyses

Soil type	Unit weight (kN/m ³)	Undrained Shear Strength, (kPa)	Internal drained friction angle (Deg)	Interface friction angle δ , (Deg)	Soil adhesion, (kPa)
Fill-Sand & Gravel	21	-	32	21	-
Silty clay	18	50 – 75 (50)*	23 – 30 (23)	-	38
Silty sand	19	-	29	19	-

*value shown in bracket is recommended value.

6.2.2 Foundation Design (Shallow Footing)

The foundation footing options for box culvert and precast concrete structure were assessed. The shallow footing depths were assumed to be in the silty clay formation. The capacities of shallow foundations were estimated. CSP has no bearing resistance issue therefore bearing resistance is not analysed. The geotechnical resistance for box culvert was estimated assuming a strip footing consisting of a length of 8 m and width equal to the 14 m and a depth of the foundation base equal to 0 m, which is a temporary condition prior to backfill that will be encountered during construction. Bearing capacities for open footings were estimated with varying bearing foundation widths and depth of soil cover. The geotechnical resistance for footings was estimated assuming a strip footing of various width with a length equal to 8 m and founded at a depth of 2.5 m from the existing embankment surface.

Ultimate bearing resistances were estimated accordance with Bridge Design Code (CHBDC), section 6.7.2. The geotechnical resistance was estimated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm under axial load. 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 and shown in Table 6-2.

Table 6-2 Geotechnical resistances and reactions for shallow footing

Footing Width L=8.0 m	Depth of Soil Cover	Ultimate bearing capacity (kPa)	Factored Resistance at ULS (kPa)	Resistance at SLS (kPa)
B = 9.0 m	0	300	150	20
B = 1.0 m	1.0	310	155	60
	1.2	320	160	60
	1.5	330	165	60
B = 1.2 m	1.0	300	150	55
	1.2	310	155	55
	1.5	320	160	55
B = 1.5 m	1.0	290	145	45
	1.2	300	150	45
	1.5	310	155	45
B = 2.2 m	1.0	290	145	40
	1.2	290	145	40
	1.5	300	150	40
B = 3.0 m	1.0	280	140	35
	1.2	290	145	35
	1.5	300	150	35

Where unsuitable or unstable soils (loose or soft soils) are encountered at the foundation preparation depth, 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.

SLS reported here are for final condition. The recommended bearing resistance at SLS is in reference to an allowable net increase over the existing loading condition.

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.

6.2.3 Foundation Design (Deep Foundation)

Deep foundations may be considered if shallow foundations have insufficient capacity. Pile could be installed into dense sand formation or bedrock formation. Pile resistance for various sizes of H piles sitting on the dense sand formation and bedrock were assessed. The soil profile assumed for the pile capacity estimation were 0.2 m of fill underlying 10.5 m of silty clay which underlain again by 1.5 m of sand & gravel before encountering bedrock. Bedrock's unconfined compressive strength results were between 30 and 136 MPa. The following Table 6-3, Table 6-4 and Table 6-5 show design resistance of various sizes of steel piles driven to a specified elevation as friction and bearing pile. The resistance includes both friction and end bearing components. The SLS for pile installed in sand has been based on a settlement not exceeding 25 mm. SLS for pile installed in bedrock is same as ULS as very minimum settlement is expected for pile installed in the bedrock.

Lateral pile load capacities were estimated using Brom's method. Steel grade S240GP was assumed for the pile. Analysis revealed lateral deformation at the surface level under ULS load for the long pile less than 25mm of lateral deformation; therefore, ULS and SLS are the same.

Table 6-3 Design resistance for H steel piles in dense sand formation

Pile Length/ Elevation	H-Pile 305x79		H-Pile 360x84	
	ULS (kN)	SLS (kN)	ULS (kN)	SLS (kN)
L=11 m/ 318.1 m	295 (skin friction – 260, end bearing – 35)	225 (skin friction - 215, end bearing – 10)	345 (skin friction – 310, end bearing 35)	265 (skin friction – 260, end bearing – 5)

Table 6-4 Design resistance for H steel piles at bedrock formation

Pile Length/ Elevation	H-Pile 305x79		H-Pile 360x84	
	ULS (kN)	SLS (kN)	ULS (kN)	SLS (kN)
L=12.2 m/ 316.9 m	320 (skin friction – 270, end bearing – 50)	320 (skin friction - 270, end bearing – 50)	375 (skin friction – 315, end bearing – 60)	375 (skin friction - 315, end bearing – 60)

Table 6-5 Design lateral resistance for H steel piles

H-Pile 305x79		H-Pile 360x84	
ULS (kN)	SLS (kN)	ULS (kN)	SLS (kN)
25	25*	30	30*

*For long pile, lateral load capacity is controlled by yield moment of the pile.

The geotechnical resistance factor for applied for a static analysis in compression is 0.4 in accordance with Section 6 of the CHBDC.

The site is considered not to have consolidation settlement due to the existing embankment. Therefore, the downdrag force on the pile was not considered. Pile driving in the clay can reduce bearing capacity temporarily due to generated high porewater pressure during pile driving. Pile capacity estimated is for ultimate state and did not consider for the temporary reduced state of bearing capacity. It will be further discussed if pile foundation option is selected for the site.

In pile groups, piles should be spaced no closer than 2.5 times the pile larger dimension, measured centre to centre. All piles in a bent should be driven to approximately the same tip elevation. For the friction piles, their final capacity will be achieved sometime after driving, and a 30 day period is recommended for this.

Driving records should be kept for each pile. Information to be recorded should include but not necessarily be limited to: pile dimensions, hammer type, rated energy, ram weight, cap block weight and type, anvil weight, number of blows for each 0.3 m of penetration and final set. All pile driving equipment must be in good working order. The behaviour of the piles should be monitored during driving for any signs indicative of pile damage.

6.3 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. 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. Placement of bedding material for the culvert replacement shall be carried out in dry condition with using suitable surface water control method at the site such as diverting water flow and suitable dewatering methods.

The bedding for the structure should be designed in accordance with Section 7.8 of the CHBDC. The bedding shall be a minimum of 0.5 m thick and extend to a minimum width (half of the

width of culvert) beyond all sides of the culvert. The bedding material should consist of “Granular A” as per Soil Group I in accordance with Table 7.4 of the Canadian Highway Bridge Design Code. The “Granular A” shall be in accordance to OPSS.PROV 1010. The “Granular A” should be placed in layers not exceeding 200 mm in thickness, loose measurement, and each layer compacted to a minimum of 95 % of standard Proctor maximum dry density. 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.

6.4 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 on 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”.

6.5 Lateral and Sliding Resistances

The analysis of horizontal and vertical effects of earth loads on the structure can be performed considering soil parameters given in Table 6-1 and as described in Section 7.6.3.1 in Canadian Highway Bridge Design Code. Temporary bracing and shoring may be designed using the typical soil parameters given in Table 6-6, however 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 6-6 Lateral Earth Pressure Coefficients

Soil type	Active Earth Pressure (K_a)	Passive Earth Pressure, (K_p)	Earth Pressure at Rest, (K_0)
Equation *	$\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$	$\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)$	$(1 - \sin\phi)$
Fill-Sand & Gravel	0.30	3.25	0.47
Silty clay	0.43	2.28	0.60
Silty sand	0.34	2.88	0.51

* ϕ is an angle of internal friction

**The earth pressure coefficients provided here are for the normally consolidated soils condition considering fully mobilized condition

For overconsolidated (OC) soil, 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

Alternatively, earth pressure due to compaction induced stresses can be determined by the following procedure.

The shape of the lateral pressure distribution will depend on the degree of compaction achieved in the soil backfill against the wall. Where the backfill adjacent to the wall will be compacted to 95% of the SPMDD or greater, the design earth pressure should adopt a combined trapezoidal/triangular distribution as per Figure 24.9 of CFEM. The typical relationships to be used in calculating the lateral pressures for structural design are provided in Figure 24.9 of CFEM and the load of typical compactors are provided in Table 24.5 of CFEM.

6.6 Approach Embankment

Stability and settlement of the approach embankments were analysed. Presence of silty clay may cause settlement if additional loading is applied. If grade raise to existing embankment is required, settlement for additional load shall be assessed. However, existing embankment is in a stable condition and no excessive settlement is expected.

The approach embankments should have side slopes no steeper than 2h: 1v. For approaches embankments, earth fill should be granular (Select Subgrade Material below subgrade level) and should be compacted in accordance with OPS 501.

Proposed road embankment side slopes should be vegetated or place the rip rap for long term erosion control from surface runoff.

It was understood that 1 m grade raise on existing embankment is required for the new culvert structure. Settlement by additional load is estimated approximately 130 mm.

6.7 Stability of Embankment Slopes and Abutments Forward Slope

The material used for 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 on 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 and 100 µm, specified in OPSS 1860 "Material Specifications for Geotextiles".

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.

6.8 Seismic Design Loading

Seismic loading analysis was considered in the assessment in accordance to Section 4.4.6 of the CHDBC 2010. The Peak Horizontal Acceleration (PHA) was obtained in accordance with the CHDBC 2010 by interpolating National Building Code of Canada (2010) data. The value of PHA for

10 % probability of exceedance in 50 years in the area is 0.011 g. The seismic performance zone classification for the site defined for emergency route and other bridges is Zone 1 (Table 4.1 Seismic Performance Zones, Section 4.4.4 of CHBDC, 2010).

Based on the sub-surface formation found in the boreholes, the soil profile type is classified as Type III and the site coefficient (S) for the project site is classified as 1.5 (Table 4.4, Site Coefficient, Section 4.4.6.1, of CHBDC, 2010).

6.9 Liquefaction Potential

Liquefaction potential for the site was analysed using the method proposed by Seed and Idriss (1971). Liquefaction potential depends primarily on six factors, namely;

- Relative density of the soil deposit, R_D
- Confining pressure over the soil deposits, σ_3
- Peak pulsating stress from the seismic source, σ_d
- Number of cycles of pulsating stress application,
- Overconsolidation ratio of the soil deposit, and
- Fines content.

The design earthquake of 7.5 was applied for the liquefaction potential analyses. The maximum amplitude of acceleration was calculated applying the following equation proposed by Boore et. al. 1993.

The cyclic stress ratio induced in the soil by the design earthquake was estimated applying the following simplified formula proposed by Seed and Idriss (1971)

$$(r_{cyc}/\sigma'_{z0})_{eqk} = 0.65 (a_{max}/g)(\sigma_{z0}/\sigma'_{z0}) \gamma_d$$

Where

a_{max}/g = peak horizontal ground acceleration divided by acceleration of gravity

σ_{z0} = initial vertical total stress

σ'_{z0} = initial vertical effective stress

γ_d = stress reduction factor

The Standard Penetration Test (SPT) results were considered an assessment which incorporates the relative density and overconsolidation ratio of the soil deposits. SPT values obtained from field investigation were corrected to N_{60} values. The cyclic stress ratios to cause liquefaction due to an earthquake magnitude of $M = 7.5$ were obtained based on N_{60} values and fines content. These cyclic stress ratios were converted to cyclic stress ratio for the design earthquake magnitude using following equation.

$$(r_{cy}/\sigma'_{z0})_M = (r_{cy}/\sigma'_{z0})_{M=7.5}$$

Where M is the magnitude scaling factor.

The factor of safety against liquefaction (FOS) was then calculated applying the following equation;

$$F = (r_{cy}/\sigma'_{z0})_M / (r_{cy}/\sigma'_{z0})_{eqk}$$

The minimum acceptable FOS is considered as 1.25. The following Table 6-7 shows FOSs calculated for depths of various soil formations where SPT data is available. The value of PHA used for analyses was 0.011 g. Earthquake magnitude scaling factor used for the analyses was 1.4.

Table 6-7 Factor of Safety against Liquefaction Potential of Overburden Soils

Borehole No.	Depth (m)	$(N_1)_{60}$	FOS
BH1	9.1 – 13.1	6 – 50	4.5 – 44.1
BH2	6.1 – 12.5	7 – 70	6.3 – 44.1
BH5	10.7 – 12.2	18 – 50	17.8 – 44.1
BH6	10.5	50	44.1

Plasticity index of tested fine grained soils samples were between 26 and 57% which can be considered as a non-liquefiable soil for design purpose. High factor of safety was resulted for liquefaction analyses on coarse grained soils with the designed earthquake loading, therefore liquefaction is not imminent at the site.

6.10 Frost Protection

In accordance with OPSD 3090.100 "Foundation Frost Depths for Northern Ontario", the frost penetration at this location is about 2.4 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 2.4 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 803.030 and 803.031 "Frost Treatment - Pipe Culverts, Frost Penetration Line below Bedding Grade" and "Frost Treatment - Pipe Culverts, Frost Penetration Line between Top of the 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 127 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 face of the buried structure.

6.11 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. Either concrete cutoff wall or geotextile filter or graded filter shall be applied as undermining prevention measure. Cutoff walls shall be extended the whole width and depth of the bedding layer.

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".

6.12 Staged/Detour Construction

Based on site condition, detour may be used during the structure replacement construction period. Therefore, staged construction method is not discussed.

7. CLOSURE

Table 7.1 below summarizes the advantages and disadvantages of the using different structure replacement options.

Table 7-1 Advantages and Disadvantages of the Proposed Bridge/Culvert Options

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 1 CSP Culvert	Feasible	No Bearing capacity issue. Reduce construction time.	High maintenance. Requires road closure.	Low to Moderate	Low risk
Option 2- Deep foundation for the structure	Feasible	No bearing capacity concerns	Use of this option could increase construction time.	Moderate to high cost.	In general terms low risk option.
Option 3- Open Footing	Not Feasible	Use of pre-cast members could reduce construction time. Low maintenance cost.	Use of this option could increase construction time. Requires foundation excavation and preparation. Requires road closure.	Moderate cost.	In general terms low risk option (except for shoring if require).
Option 4- Concrete Box	Not Feasible	Use of pre-cast members could reduce construction time. Low maintenance cost.	Requires road closure.	Low to Moderate cost.	In general terms low risk option. (except for shoring if require)

8. REFERENCES

Canadian Foundation Engineering Manual. 2006. Fourth Edition, Canadian Geotechnical Society.
Canadian Highway Bridge Design Code. 2010, 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.

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:



Selorm Danku P. Eng
Geotechnical Engineer

Reviewed by:



Dr. ASM Masud Karim, P.Eng.
Senior Associate – Regional Manager
Infrastructure Client Group

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

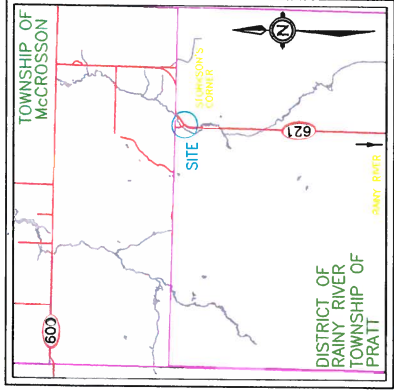
DRAWINGS

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

AG No 5014-E-0049
WP No 6065-13-00
SITE No 45-282C
GEORES No 52D-22

REPLACEMENT OF
LITTLE GRASSY TIMBER BRIDGE
SITE PLAN
STA 10+060 TO STA 10+120
Survey _____ Revised _____

SHEET
1

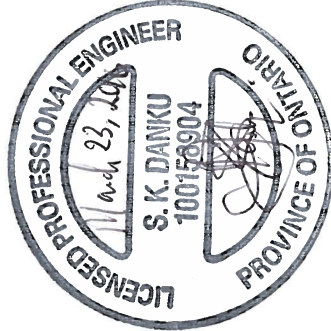


KEY PLAN

2.5 km 0 2.5 km

LEGEND

Borehole



TIMBER BRIDGE 7m SPAN
SPRUCE

10+080

BM 328.565
N&W IN ROOT OF
201491.8
E 5420262.7

W/L 326.95
OCT. 28/2014

W/L 326.89
OCT. 28/2014

WOOD TIE RETAINING WALL

CREEK

EC

P&W FENCE

PLAN



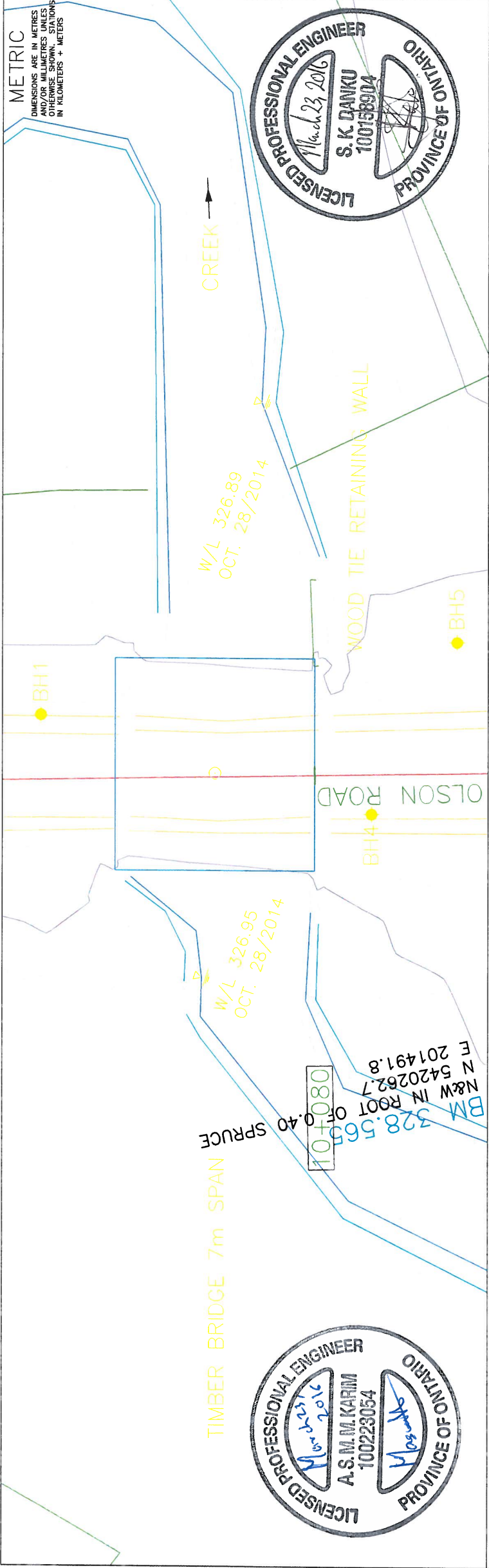
No.	Elev. (m)	MTM Zone 16		Survey	
		North (m)	East (m)	Station	Offset
BH1	328.6	5420290	201494	10+089	2.26 m RT
BH2	328.6	5420290	201491	10+092	0.92 m LT
BH3	328.4	5420299	201490	10+102	1.49 m RT
BH4	328.6	5420276	201495	10+078	1.38 m LT
BH5	329.1	5420265	201501	10+075	4.59 m RT
BH6	328.7	5420265	201501	10+065	1.62 m LT

REV	DATE	ISSUE	APPROVAL
1	11/03/16	FINAL	MK

NOTE:
The boundaries between soil areas have been established only at borehole
locations. All other boundaries are assumed by interpolation
and may not represent actual conditions.

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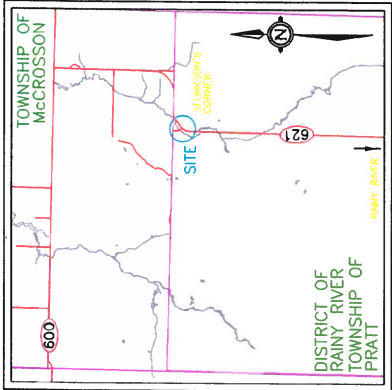
DRAWING 1



AG No 5014-E-0049
WP No 6065-13-00
SITE No 45-282C
GEOCRES No 52D-22

REPLACEMENT OF
LITTLE GRASSY TIMBER BRIDGE
STRATIGRAPHIC PROFILE AND
CROSS SECTION
STA 10+060 TO STA 10+120
Survey _____ Revised _____

SHEET
2

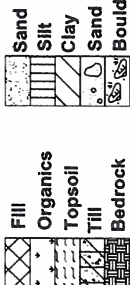


KEY PLAN
2.5 km
0 2.5 km

LEGEND

Borehole

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



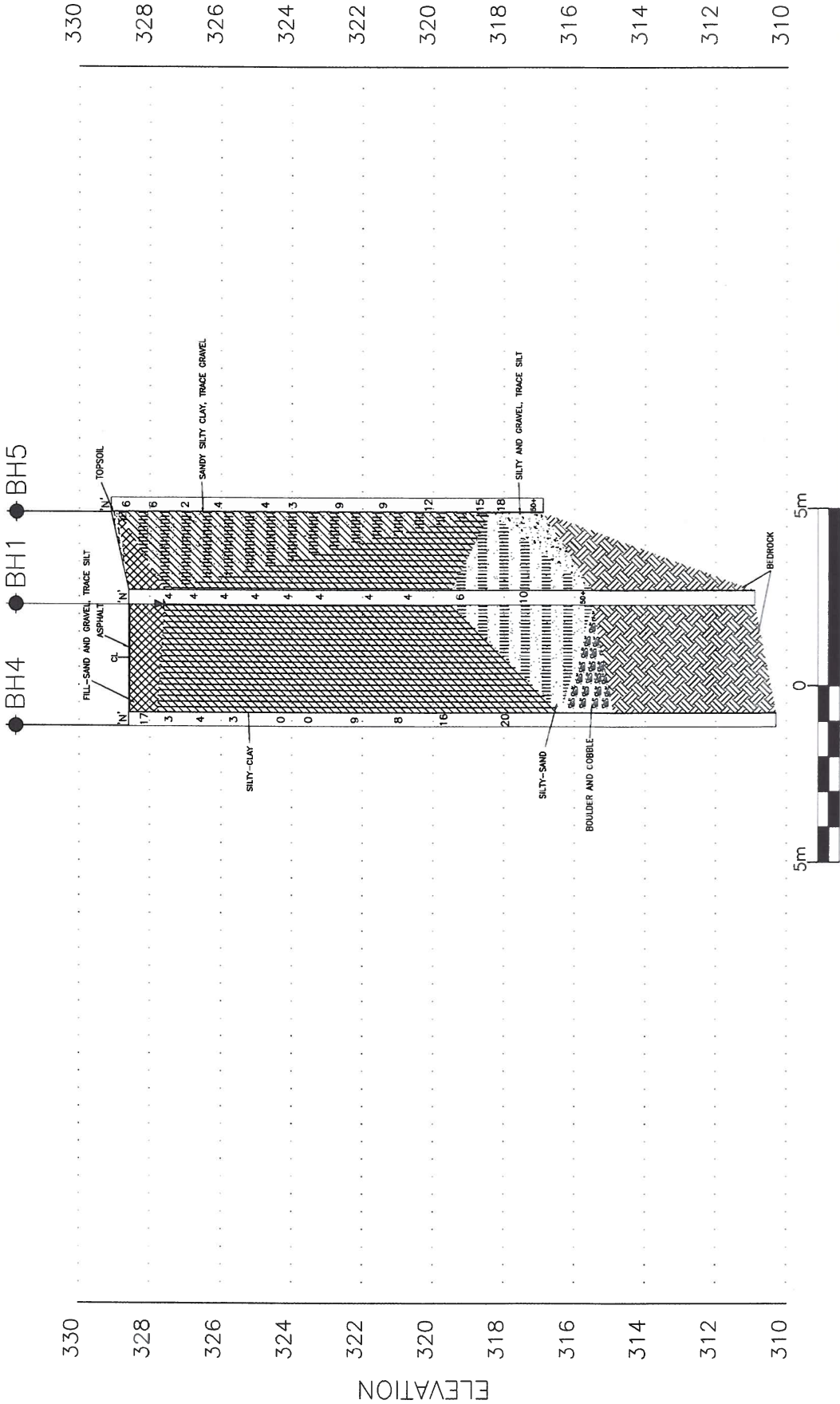
No.	Elev. (m)	MTM Zone16			Survey		
		North (m)	East (m)	Station	Station	Offset	
BH1	328.6	5420290	201494	10+089	2.26 m	RT	
BH2	328.6	5420290	201491	10+092	0.92 m	LT	
BH3	328.4	5420299	201490	10+102	1.49 m	RT	
BH4	328.6	5420276	201495	10+078	1.38 m	LT	
BH5	329.1	5420265	201501	10+075	4.59 m	RT	
BH6	328.7	5420265	201501	10+065	1.62 m	LT	

REV	DATE	ISSUE	APPROVAL
1	11/03/16	FINAL	HK

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

DST
DST Consulting Engineers Inc.
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Thunder Bay, ON P7B 5V5
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Email: thunderbay@dstgroup.com

DRAWING 2



Appendix D
ENCLOSURES

RECORD OF BOREHOLE No BH1

1 OF 2

METRIC

W.P. G.W.P. 6065-13-00 LOCATION LITTLE GRASSY RIVER TRIB. CULVERT ORIGINATED BY RW
DIST MTO HWY OLSON ROAD BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID COMPILED BY SA
DATUM Geodetic DATE 2015 09 09 CHECKED BY BV

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 □ QUICK TRIAXIAL	+ FIELD VANE × LAB VANE							
328.6	GROUND SURFACE							20 40 60 80 100								
	FILL - SAND AND GRAVEL, TRACE SILT VERY LOOSE BROWN		AS1	AS		▽	328									
327.6	SILTY CLAY		SS2	SS	4		327									
1.0			SS3	SS	2		326									
	VERY SOFT TO FIRM GREY		SS4	SS	4		325									
			SS5	SS	4		324									
			SS6	SS	2		323									
			SS7	SS	1		322									
							321									
			SS8	SS	7		320									
							319									
			SS9	SS	7		318									
							317									
319.5	SILTY AND SAND TRACE GRAVEL LOOSE TO VERY DENSE GREY		SS10	SS	6	316										
9.1																
			SS11	SS	10											
			SS12	SS	100+											

Continued Next Page

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

ENCLOSURE 1

RECORD OF BOREHOLE No BH1

2 OF 2

METRIC

W.P. G.W.P. 6065-13-00 LOCATION LITTLE GRASSY RIVER TRIB. CULVERT ORIGINATED BY RW
 DIST MTO HWY OLSON ROAD BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID COMPILED BY SA
 DATUM Geodetic DATE 2015 09 09 CHECKED BY BV

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	W _p	W	W _L			
315.5 13.1	GROUND SURFACE																
	BEDROCK GNEISS DIORITE																
	RQD = 41%																
	RQD = 47%																
	RQD = 73%																
310.9 17.7	END OF BOREHOLE AT 17.7 m																

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

RECORD OF BOREHOLE No BH2

1 OF 1

METRIC

W.P. G.W.P. 6065-13-00 LOCATION LITTLE GRASSY RIVER TRIB. CULVERT ORIGINATED BY RW
DIST MTO HWY OLSON ROAD BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID COMPILED BY SA
DATUM Geodetic DATE 2015 09 01 CHECKED BY BV

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
328.6	GROUND SURFACE							20	40	60	80	100					
	FILL - SAND AND GRAVEL, TRACE SILT COMPACT BROWN			SS1	SS	25											
327.8	SILTY CLAY VERY SOFT TO STIFF GREY			SS2	SS	6											
0.8				SS3	SS	5											
				SS4	SS	4											
				SS5	SS	2											
				SS6	SS	2											
				SS7	SS	1											
322.5	SAND WITH GRAVEL, SOME SILT			SS8	SS	9											
6.1																	
				SS9	SS	8											
				SS10	SS	7											
				SS11	SS	12											

ON_MOT-HIGH VANES LITTLE GRASSY CREEK.GPJ DST_MIN.GDT 18/3/16

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ENCLOSURE 3

RECORD OF BOREHOLE No BH3

1 OF 1

METRIC

W.P. G.W.P. 6065-13-00 LOCATION LITTLE GRASSY RIVER TRIB. CULVERT ORIGINATED BY RW
 DIST MTO HWY OLSON ROAD BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID COMPILED BY SA
 DATUM Geodetic DATE 2015 08 31 CHECKED BY BV

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			20	40	60	80	100		
328.4	GROUND SURFACE													
	FILL - SAND AND GRAVEL		SS1	SS	22		328							
327.6	COMPACT BROWN													
0.8	SILTY CLAY VERY SOFT TO STIFF BROWN TO GREY		SS2	SS	3		327							
			SS3	SS	2									
			SS4	SS	3		326							
			SS5	SS	5		325							
			SS6	SS	2		324							
			SS7	SS	1		323							
			SS8	SS	7		322							
			SS9	SS	7		321							
			SS10	SS	10		320							
			SS11	SS	2		318							
316.5	END OF BOREHOLE AT 11.9 m						317							
11.9														

ON MOT-HIGH VANES LITTLE GRASSY CREEK.GPJ DST_MIN.GDT 18/3/16

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ENCLOSURE 4

RECORD OF BOREHOLE No BH4

1 OF 2

METRIC

W.P. G.W.P. 6065-13-00 LOCATION LITTLE GRASSY RIVER TRIB. CULVERT ORIGINATED BY RW
 DIST MTO HWY OLSON ROAD BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID COMPILED BY SA
 DATUM Geodetic DATE 2015 08 29 CHECKED BY BV

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			20	40	60	80	100		
328.6	GROUND SURFACE													
327.8	FILL - SAND, WITH SILT, SOME GRAVEL COMPACT GREY		SS1	SS	17		328							13 54 (33)
0.8	SILTY CLAY SOFT TO STIFF GREY		SS2	SS	3		327							
			SS3	SS	4									
			SS4	SS	3		326							
			ST1	SH			325							
			SS5	SS	0		324							
			SS6	SS	0		323							
			SS7	SS	9		322							
			SS8	SS	8		321							
			SS9	SS	16		320							
318.9	CLAYEY SAND, SOME GRAVEL						319							
9.7	SANDY CLAY, SOME GRAVEL		SS10	SS	20		318							
316.4	BOULDER AND COBBLE						317							
12.2							316							

Continued Next Page

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

ENCLOSURE 5

RECORD OF BOREHOLE No BH4

2 OF 2

METRIC

W.P. G.W.P. 6065-13-00 LOCATION LITTLE GRASSY RIVER TRIB. CULVERT ORIGINATED BY RW
 DIST MTO HWY OLSON ROAD BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID COMPILED BY SA
 DATUM Geodetic DATE 2015 08 29 CHECKED BY BV

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			20	40	60	80	100		
	GROUND SURFACE							SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE □ QUICK TRIAXIAL × LAB VANE						
								WATER CONTENT (%)						
								50	100	150	200	250		
314.9							315							
13.7	BEDROCK GNEISS DIORITE													
	RQD = 0%						314							
	RQD = 66%						313							
	RQD = 55%						312							
							311							
310.3	END OF BOREHOLE AT 18.3 m													
18.3														

ONL_MOT-HIGH VANES LITTLE GRASSY CREEK.GPJ DST_MIN.GDT 18/3/16

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

RECORD OF BOREHOLE No BH5

1 OF 1

METRIC

W.P. G.W.P. 6065-13-00 LOCATION LITTLE GRASSY RIVER TRIB. CULVERT ORIGINATED BY RW
 DIST MTO HWY OLSON ROAD BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID COMPILED BY SA
 DATUM Geodetic DATE 2015 08 31 CHECKED BY BV

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		20	40	60	80	100		
329.1	GROUND SURFACE												
328.9	TOPSOIL												
0.2	SANDY SILTY CLAY, TRACE GRAVEL SOFT TO FIRM BROWN TO GREY		SS1	SS	6								8 38 (54)
			SS2	SS	6								11 41 27 21
			SS3	SS	2								
			SS4	SS	4								
			SS5	SS	4								
			SS6	SS	3								
			SS7	SS	9								Saturated
			SS8	SS	9								
			SS9	SS	12								
			SS10	SS	15								
318.4	SAND SOME SILT		SS11	SS	18								0 90 (10)
10.7	COMPACT TO VERY DENSE GREY		SS12	SS	100+								
316.9	END OF BOREHOLE AT 12.2 m												
12.2													

ON MOT-HIGH VANES LITTLE GRASSY CREEK.GPJ DST_MIN.GDT 18/3/16

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ENCLOSURE 7

RECORD OF BOREHOLE No BH6

1 OF 1

METRIC

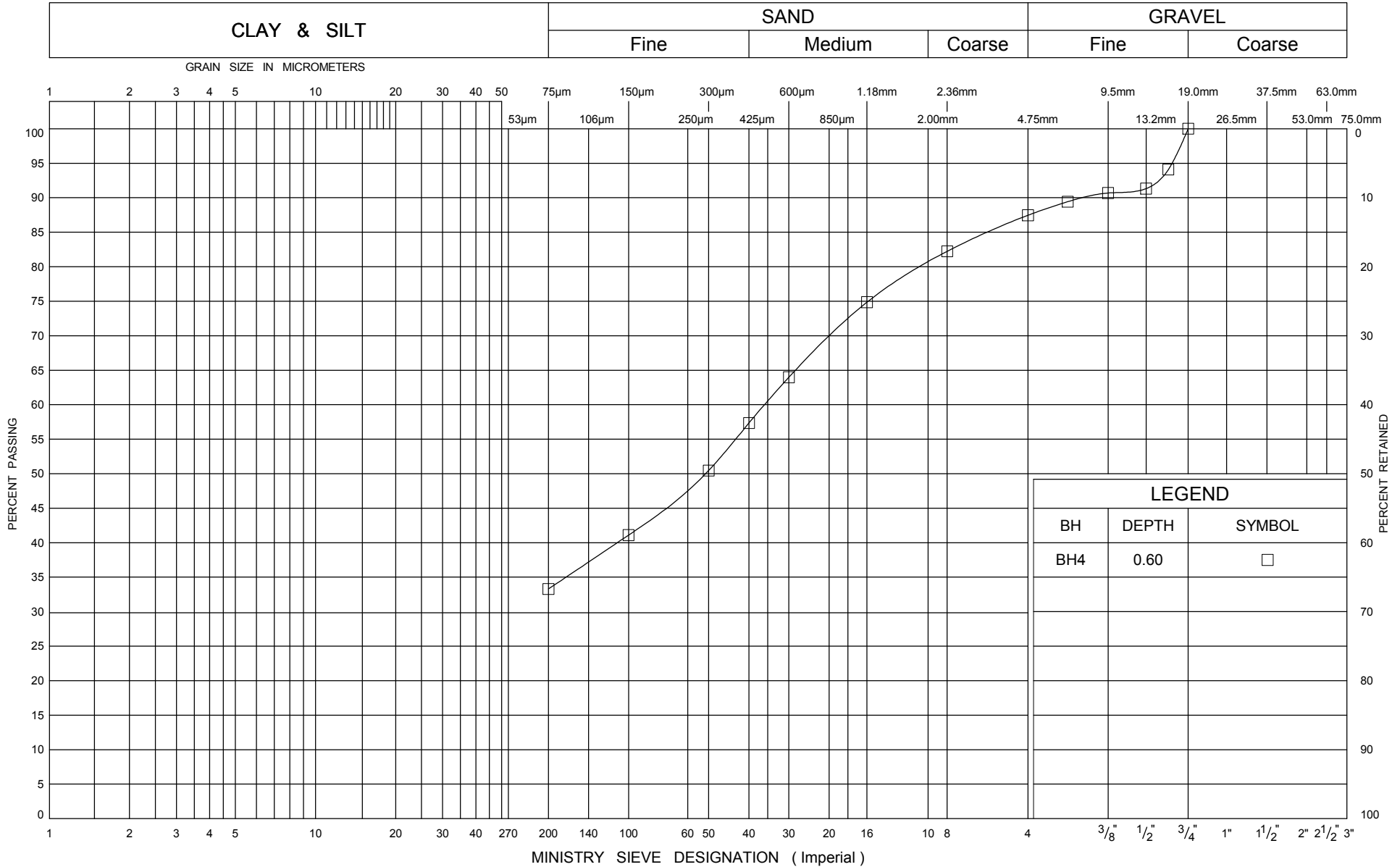
W.P. G.W.P. 6065-13-00 LOCATION LITTLE GRASSY RIVER TRIB. CULVERT ORIGINATED BY RW
 DIST MTO HWY OLSON ROAD BOREHOLE TYPE HOLLOW STEM AUGER - 80 mm ID COMPILED BY SA
 DATUM Geodetic DATE 2015 08 30 CHECKED BY BV

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	□ QUICK TRIAXIAL	+	×						FIELD VANE	LAB VANE	
328.7	GROUND SURFACE							20	40	60	80	100							
328.1	FILL - SAND AND GRAVEL		SS1	SS	10	▽													
0.6	COMPACT BROWN SILTY CLAY																		
	VERY SOFT TO VERY STIFF BROWN TO GREY		SS2	SS	6														
			SS3	SS	10														
			SS4	SS	7														
			SS5	SS	6														
			SS6	SS	1														
			SS7	SS	8														
			SS8	SS	7														
		SS9	SS	10															
		SS10	SS	11															
318.9	SILTY SAND																		
9.8	VERY DENSE GREY																		
318.0	END OF BOREHOLE AT 10.7 m		SS11	SS	100+														
10.7																			

ON MOT-HIGH VANES LITTLE GRASSY CREEK GPJ DST_MIN.GDT 18/3/16

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

UNIFIED SOIL CLASSIFICATION SYSTEM



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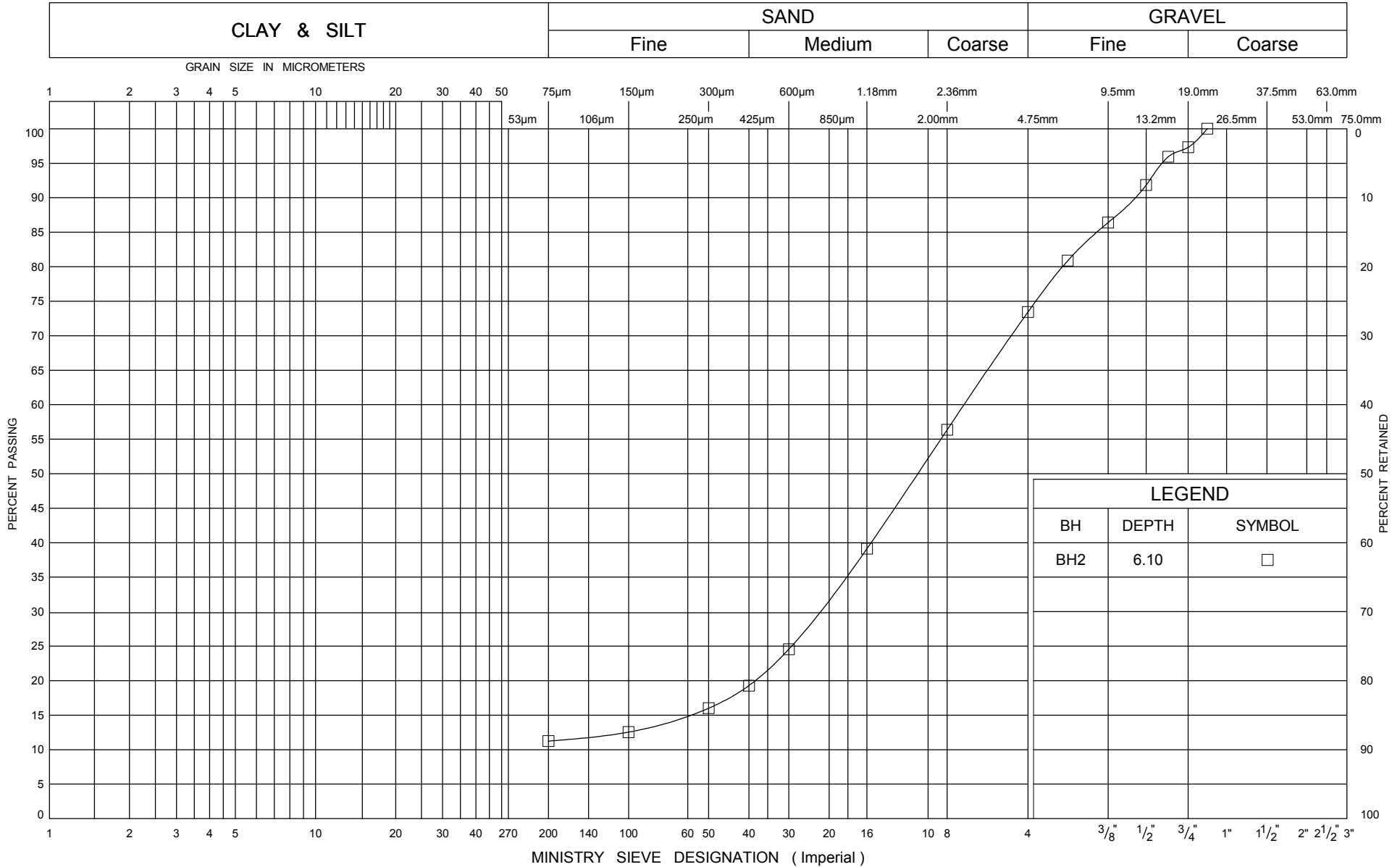
GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
FILL - SILTY SAND

ENCLOSURE 9

DST REF. # GS-TB-021102

LITTLE GRASSY TRIB. CULVERT

UNIFIED SOIL CLASSIFICATION SYSTEM



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GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
SAND

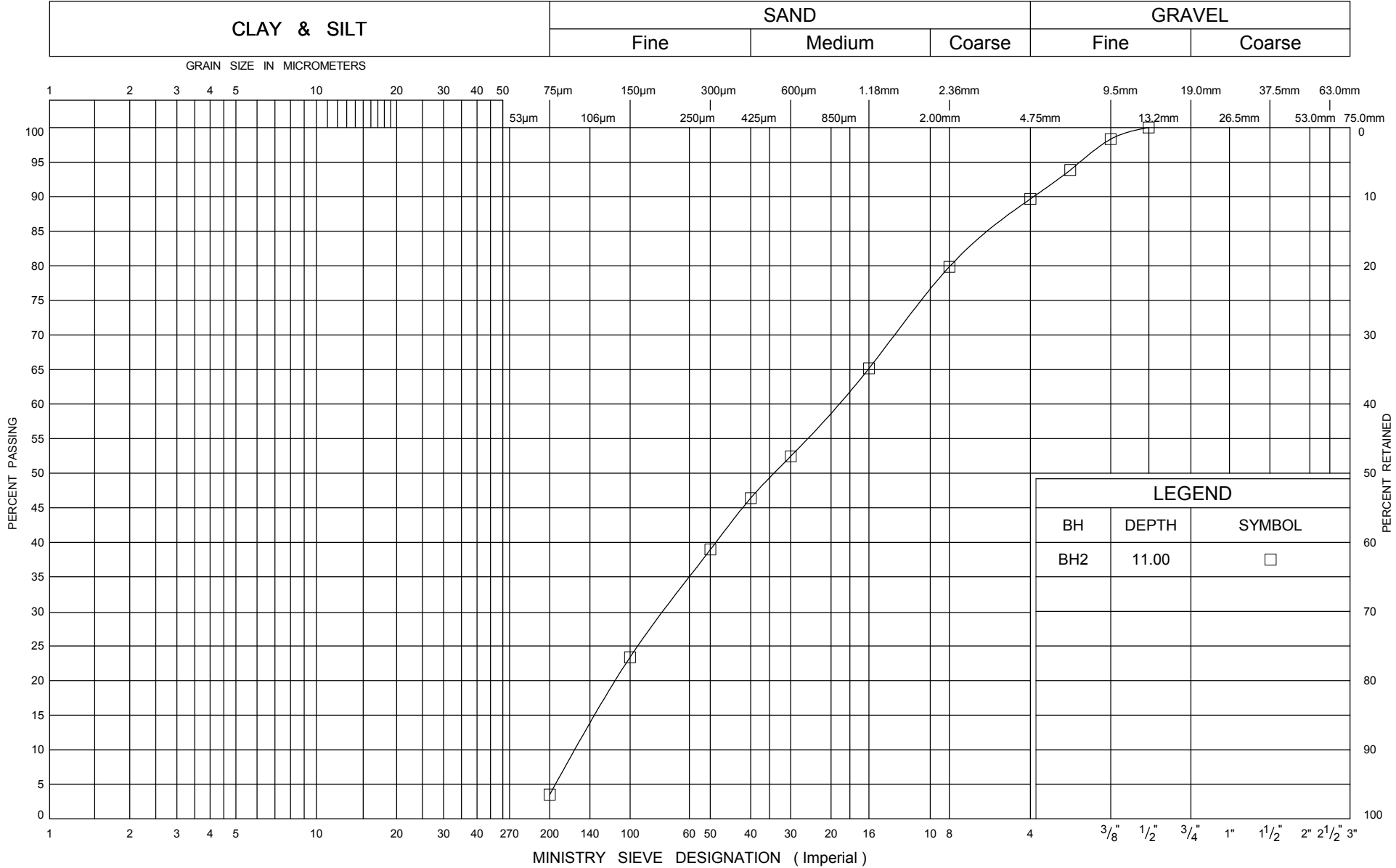
ENCLOSURE 10

DST REF. # GS-TB-021102

LITTLE GRASSY TRIB. CULVERT

ONTARIO MOT GRAIN SIZE LITTLE GRASSY CREEK.GPJ DST_MIN.GDT 19/11/15

UNIFIED SOIL CLASSIFICATION SYSTEM



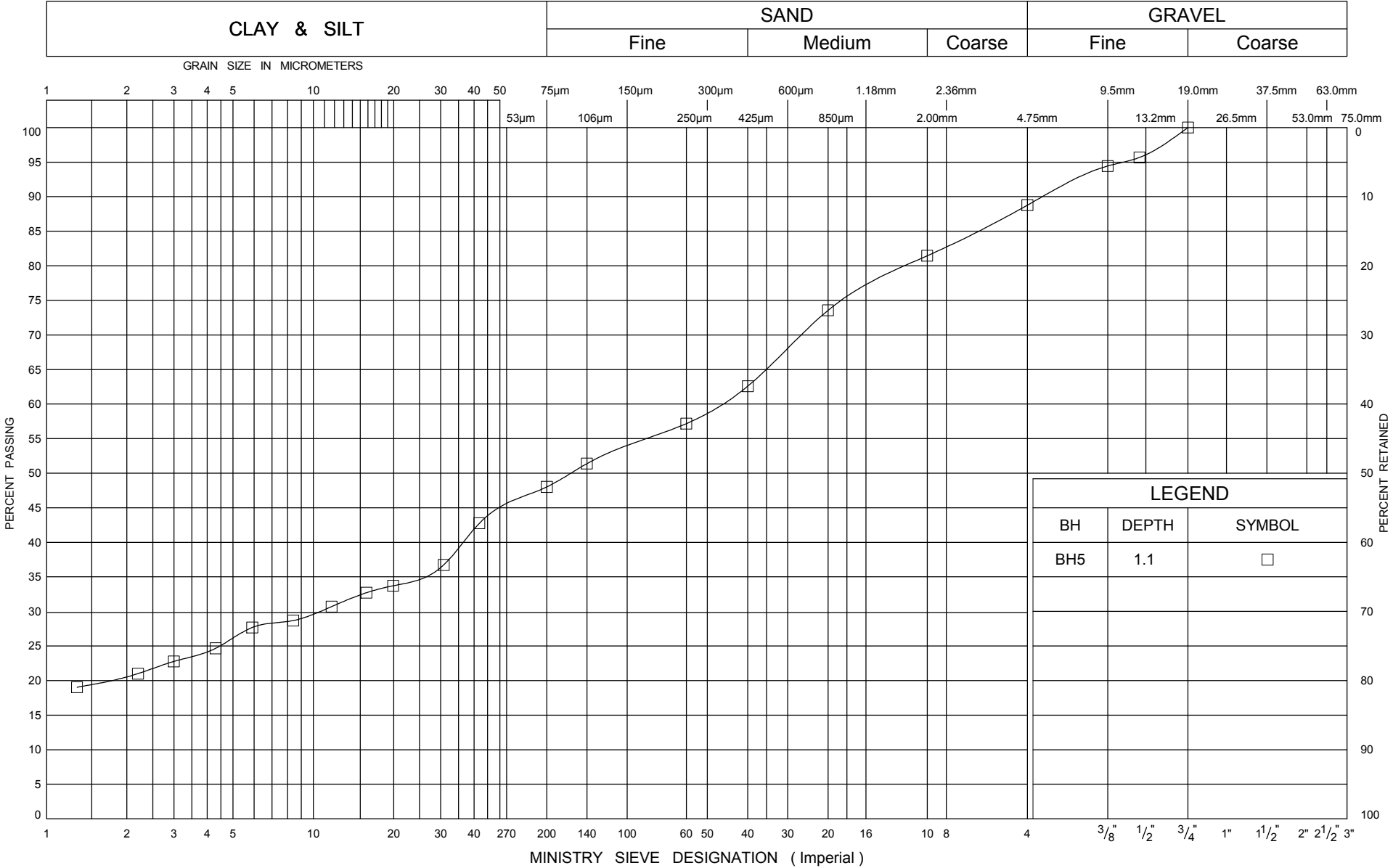
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GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
SAND

ENCLOSURE 11
DST REF. # GS-TB-021102
LITTLE GRASSY TRIB. CULVERT

ONTARIO MOT GRAIN SIZE LITTLE GRASSY CREEK.GPJ DST_MIN.GDT 20/11/15

UNIFIED SOIL CLASSIFICATION SYSTEM

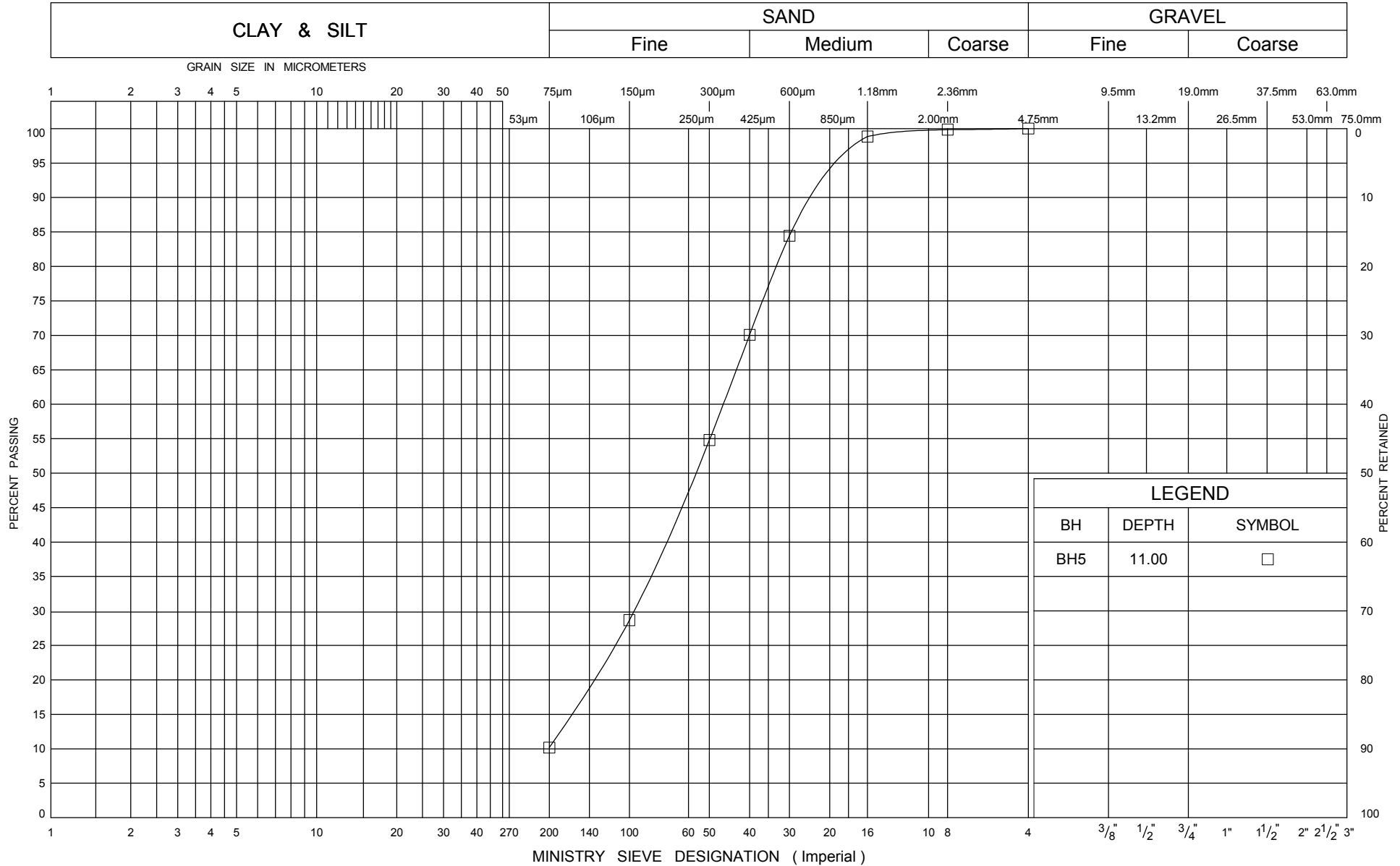


GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
SAND

ENCLOSURE 12
DST REF. # GS-TB-021102
LITTLE GRASSY TRIB. CULVERT



UNIFIED SOIL CLASSIFICATION SYSTEM



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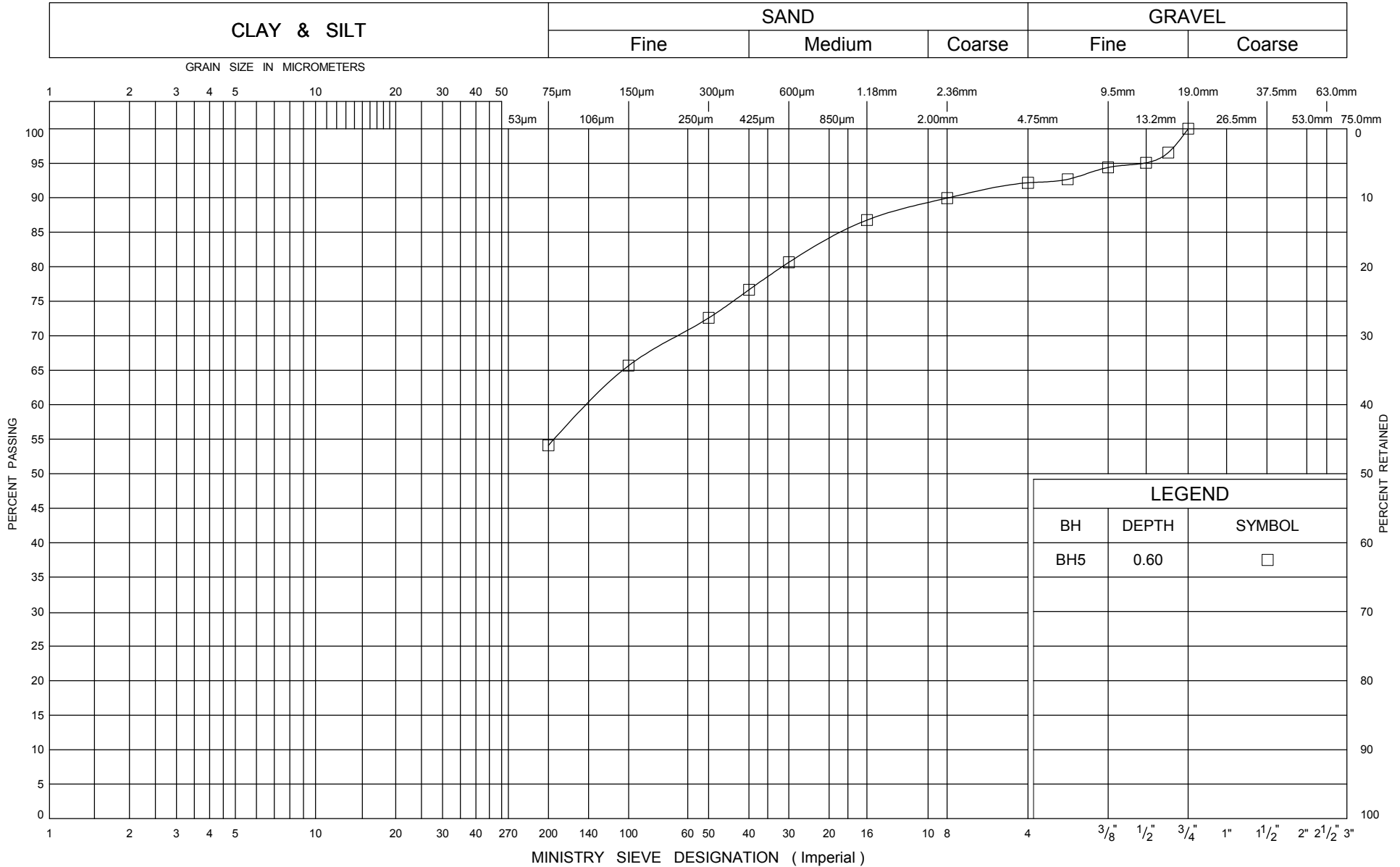
GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
SAND

ENCLOSURE 13

DST REF. # GS-TB-021102

LITTLE GRASSY TRIB. CULVERT

UNIFIED SOIL CLASSIFICATION SYSTEM



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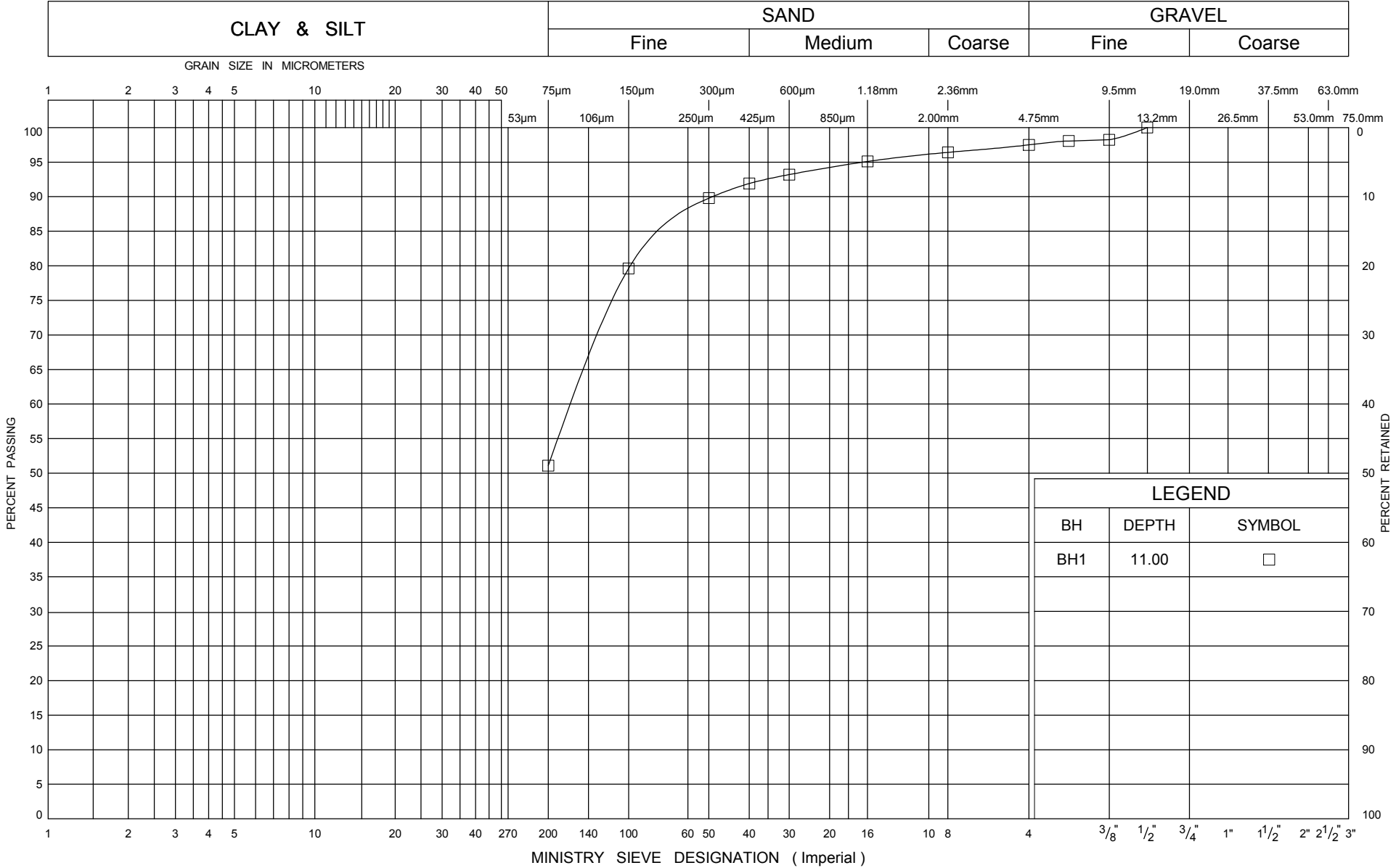
GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
SANDY SILTY CLAY

ENCLOSURE 14

DST REF. # GS-TB-021102

LITTLE GRASSY TRIB. CULVERT

UNIFIED SOIL CLASSIFICATION SYSTEM

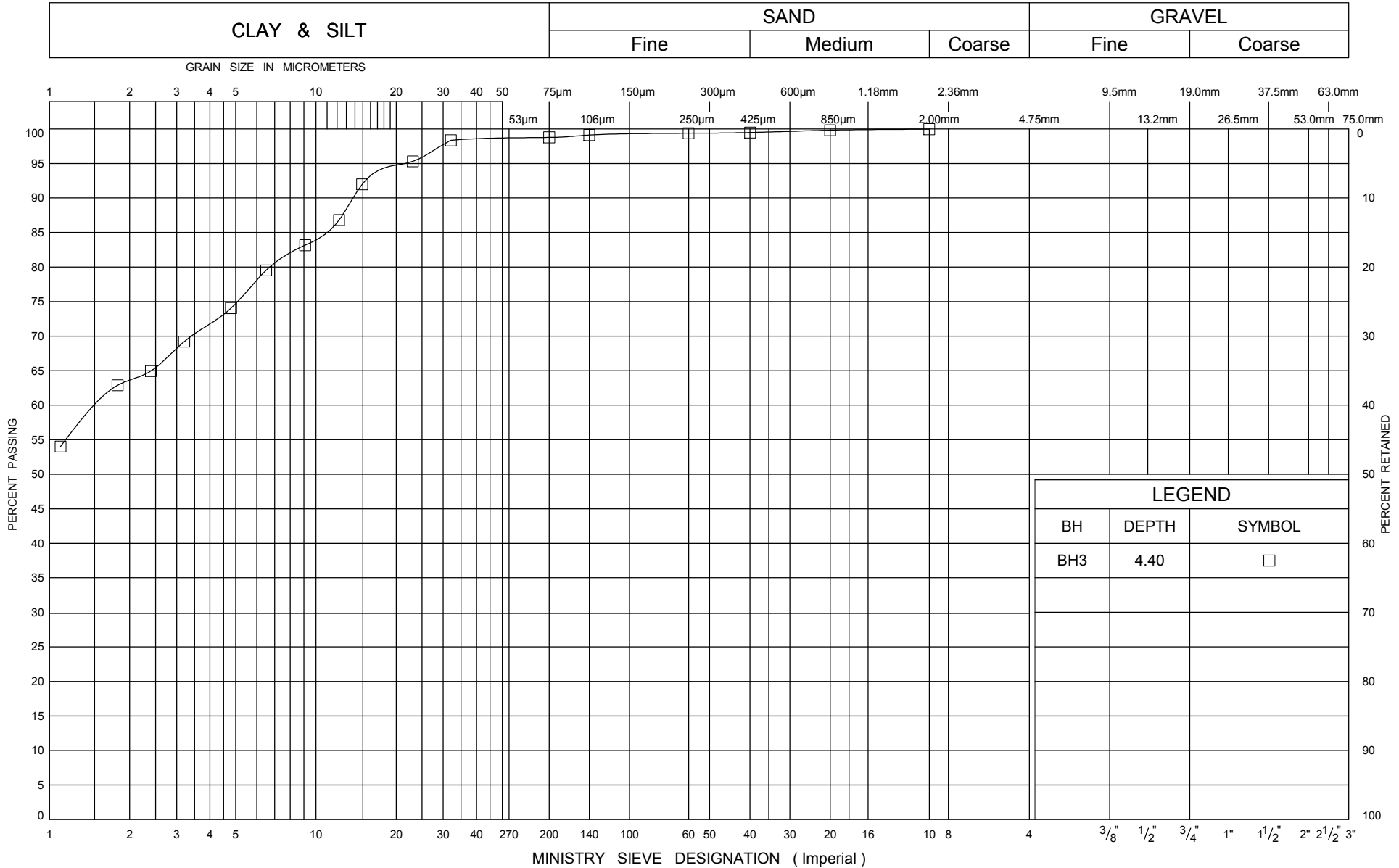


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GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
SILT

ENCLOSURE 15
DST REF. # GS-TB-021102
LITTLE GRASSY TRIB. CULVERT

UNIFIED SOIL CLASSIFICATION SYSTEM



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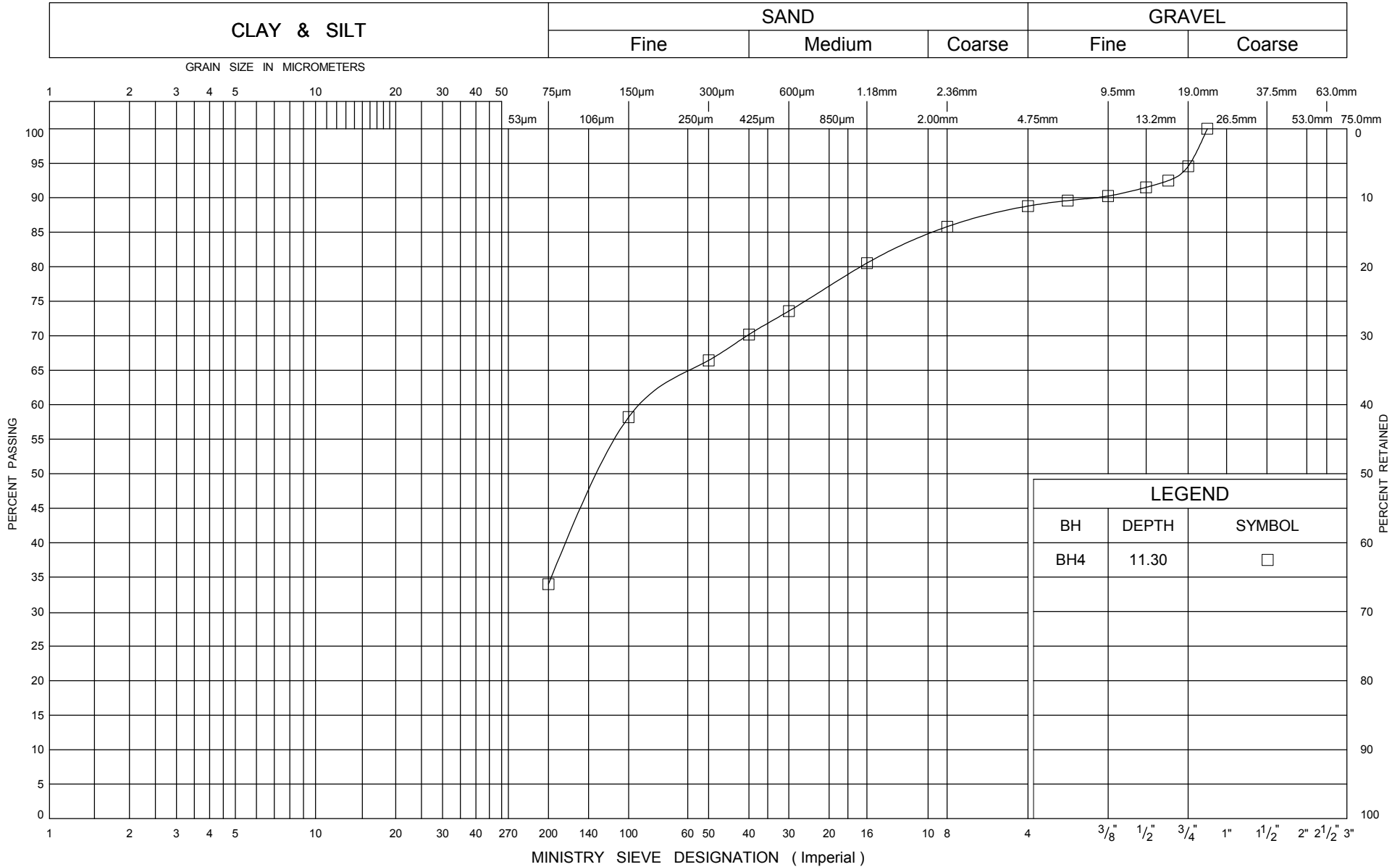
GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
SILTY CLAY

ENCLOSURE 16

DST REF. # GS-TB-021102

LITTLE GRASSY TRIB. CULVERT

UNIFIED SOIL CLASSIFICATION SYSTEM



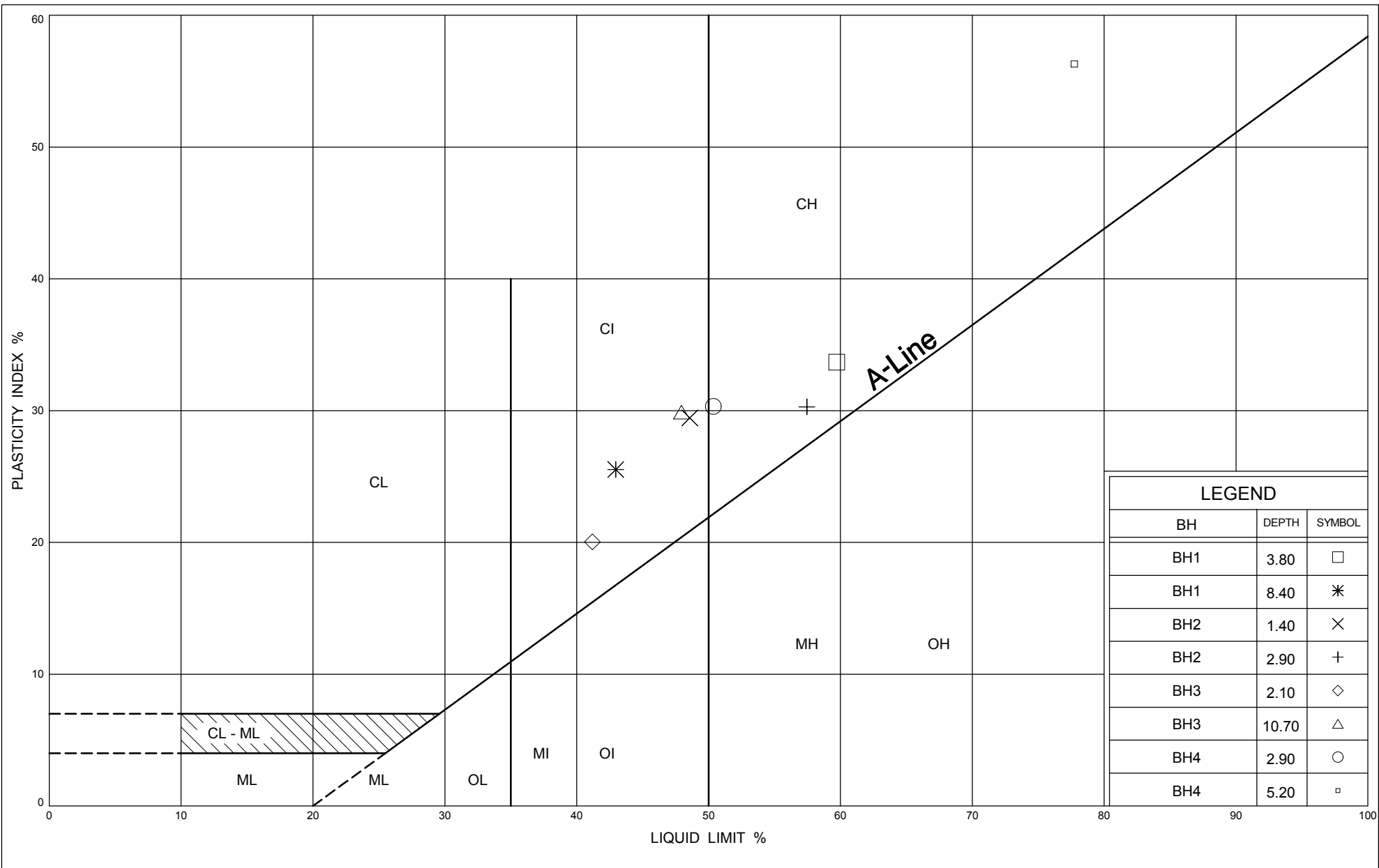
Ministry of
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GRAIN SIZE DISTRIBUTION
SOIL DESCRIPTION
CLAYEY SAND

ENCLOSURE 17

DST REF. # GS-TB-021102

LITTLE GRASSY TRIB. CULVERT



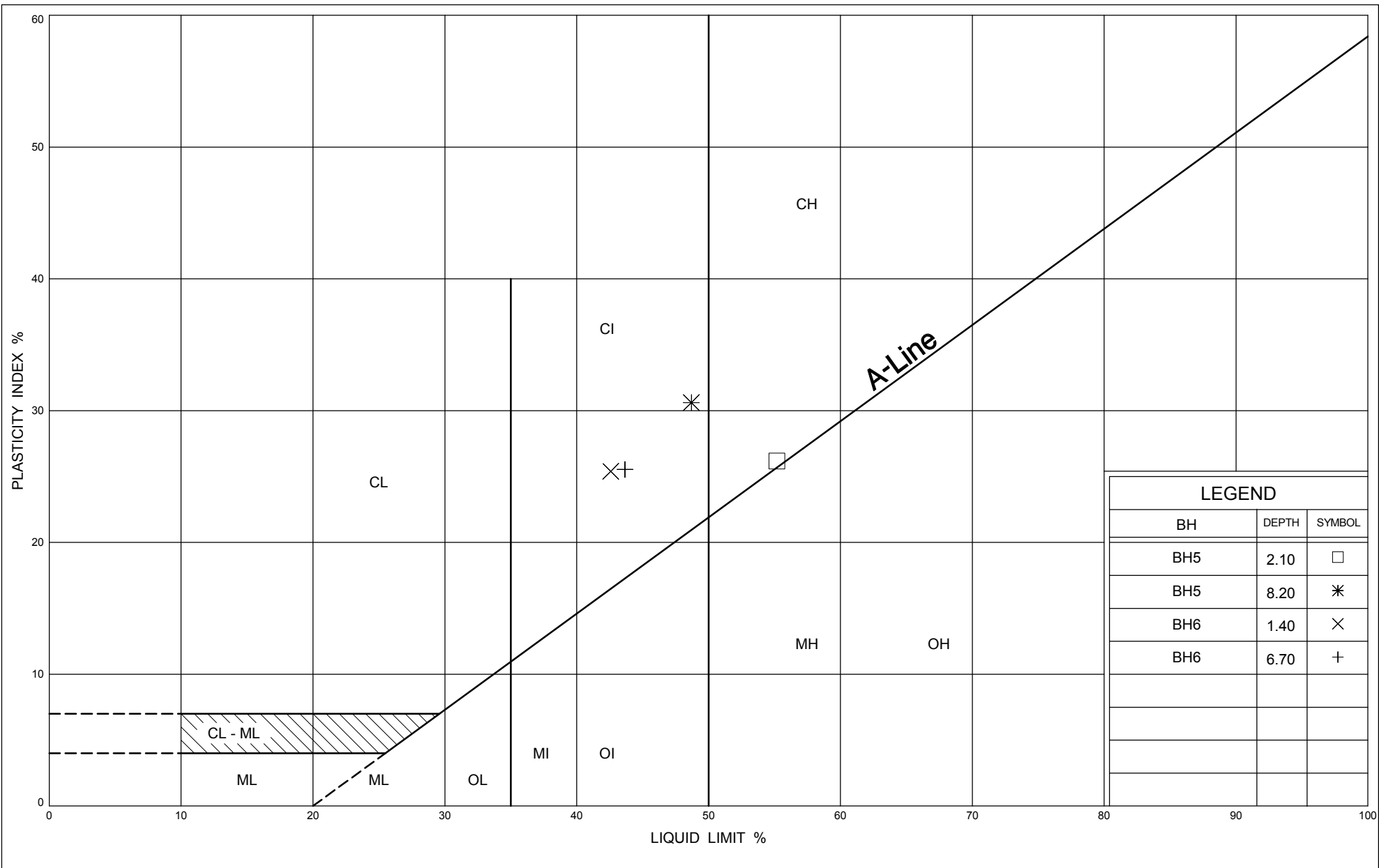
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PLASTICITY CHART INTERMEDIATE TO HIGH PLASTIC CLAY

ENCLOSURE 18

DST REF. # GS-TB-021102

LITTLE GRASSY TRIB. CULVERT



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PLASTICITY CHART INTERMEDIATE TO HIGH PLASTIC CLAY

ENCLOSURE 19

DST REF. # GS-TB-021102

LITTLE GRASSY TRIB. CULVERT

POINT LOAD TEST RESULTS (diametric and axial)

PROJECT: 5014-E-0049, Replacement of Little Grassy Trib. Culvert **JOB NO.:** GS-TB-021102

This spreadsheet is based on information from 'Suggested Method for Determining Point Load Strength', International Society for Rock Mechanics Commission on Testing Methods, 1985.

* Valid or Invalid based on description of break according to Fig 4 from ' Suggested Method for Determining Point Load Strength '

I_s = uncorrected point load strength	$D_e^2 = D^2$ for diametral tests	F = size correction factor
P = load	$D_e^2 = 4A/\pi$ for axial tests	$F = (D_e/50)^{0.45}$ or Fig. 7 from ' <i>Suggested Method for Determining Point Load Strength</i> '
D_e = equivalent core diameter	where $A = WD$	$F = \text{SQRT}(D_e/50)$ for tests near the standard (50 mm) size
	$I_s = P/D_e^2$	Size Correction $I_{s(50)} = F \times I_s$
Uniaxial Compressive Strength = $C_o = 21 \times I_s$ (50)		

21 is from: "*Using the Point Load Test to Determine the Uniaxial Compressive Strength of Coal Measure Rock*", Peng SS, Mark C, eds. Proceedings of the 19th International Conference on Ground Control in Mining. Morgantown, WV: West Virginia University.

[illegible]



DST Thunder Bay
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DST Consulting Engineers Inc.
605 Hewitson Street
Thunder Bay ON P7B 5V5

Date Received: 16-NOV-15
Report Date: 18-NOV-15 09:55 (MT)
Version: FINAL

Client Phone: 807-626-1310

Certificate of Analysis

Lab Work Order #: L1702558
Project P.O. #: NOT SUBMITTED
Job Reference:
C of C Numbers:
Legal Site Desc:

Rikki Thomson
Account Manager

[This report shall not be reproduced except in full without the written authority of the Laboratory.]

ADDRESS: 1081 Barton Street, Thunder Bay, ON P7B 5N3 Canada | Phone: +1 807 623 6463 | Fax: +1 807 623 7598
ALS CANADA LTD Part of the ALS Group A Campbell Brothers Limited Company

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters		Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L1702558-1 STRAWBERRY CREEK BRIDGE Sampled By: CLIENT on 16-NOV-15 @ 00:01 Matrix: SOIL								
Physical Tests								
Conductivity		88.9		4.0	umhos/cm		17-NOV-15	R3312712
% Moisture		2.69		0.10	%	17-NOV-15	18-NOV-15	R3312766
pH		7.31		0.10	pH units		17-NOV-15	R3312709
Resistivity		8330		100	ohm cm	17-NOV-15	17-NOV-15	R3312722
Leachable Anions & Nutrients								
Chloride		<20		20	mg/kg	17-NOV-15	17-NOV-15	R3313157
Anions and Nutrients								
Sulphate		74		20	mg/kg	17-NOV-15	17-NOV-15	R3313157
L1702558-2 KASH KAKOESIS RIVER BRIDGE Sampled By: CLIENT on 16-NOV-15 @ 00:01 Matrix: SOIL								
Physical Tests								
Conductivity		254		4.0	umhos/cm		17-NOV-15	R3312712
% Moisture		43.7		0.10	%	17-NOV-15	18-NOV-15	R3312766
pH		7.64		0.10	pH units		17-NOV-15	R3312709
Resistivity		4030		100	ohm cm	17-NOV-15	17-NOV-15	R3312722
Leachable Anions & Nutrients								
Chloride		<20		20	mg/kg	17-NOV-15	17-NOV-15	R3313157
Anions and Nutrients								
Sulphate		258		20	mg/kg	17-NOV-15	17-NOV-15	R3313157
L1702558-3 LITTLE GRASSY TIMBER BRIDGE Sampled By: CLIENT on 16-NOV-15 @ 00:01 Matrix: SOIL								
Physical Tests								
Conductivity		215		4.0	umhos/cm		17-NOV-15	R3312712
% Moisture		27.3		0.10	%	17-NOV-15	18-NOV-15	R3312766
pH		7.19		0.10	pH units		17-NOV-15	R3312709
Resistivity		4200		100	ohm cm	17-NOV-15	17-NOV-15	R3312722
Leachable Anions & Nutrients								
Chloride		<20		20	mg/kg	17-NOV-15	17-NOV-15	R3313157
Anions and Nutrients								
Sulphate		84		20	mg/kg	17-NOV-15	17-NOV-15	R3313157

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
CL-WT	Soil	Chloride in Soil	EPA 300.0
EC-WT	Soil	Conductivity (EC)	EPA 9050A
A representative subsample is tumbled with de-ionized (DI) water. The ratio of water to soil is 2:1 v/w. After tumbling the sample is then analyzed by a conductivity meter.			
MOISTURE-WT	Soil	% Moisture	Gravimetric: Oven Dried
PH-WT	Soil	pH	MOEE E3137A
Soil samples are mixed in the deionized water and the supernatant is analyzed directly by the pH meter.			
RESISTIVITY-WT	Soil	Resistivity	MOECC E3138
Resistivity on a soil is a 2:1 extraction of DI water to soil. Sample is tumbled for 30 min. Conductivity of the extraction is taken and the inverse is calculated for resistivity.			
SO4-WT	Soil	Sulphate	EPA 300.0

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
WT	ALS ENVIRONMENTAL - WATERLOO, ONTARIO, CANADA

Chain of Custody Numbers:

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.

mg/kg - milligrams per kilogram based on dry weight of sample

mg/kg wwwt - milligrams per kilogram based on wet weight of sample

mg/kg lwt - milligrams per kilogram based on lipid weight of sample

mg/L - unit of concentration based on volume, parts per million.

< - Less than.

D.L. - The reporting limit.

N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.

UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.

Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Quality Control Report

Workorder: L1702558

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Client: DST Thunder Bay
DST Consulting Engineers Inc. 605 Hewitson Street
Thunder Bay ON P7B 5V5

Contact: Selorm Danku

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
CL-WT Soil								
Batch	R3313157							
WG2215497-4 CRM		AN-CRM-WT						
Chloride			99.8		%		70-130	17-NOV-15
WG2215497-3 DUP		L1702558-1						
Chloride		<20	<20	RPD-NA	mg/kg	N/A	30	17-NOV-15
WG2215497-2 LCS								
Chloride			97.5		%		70-130	17-NOV-15
WG2215497-1 MB								
Chloride			<20		mg/kg		20	17-NOV-15
EC-WT Soil								
Batch	R3312712							
WG2215454-1 DUP		L1702558-3						
Conductivity		215	197		umhos/cm	8.7	20	17-NOV-15
WG2215743-1 MB								
Conductivity			<4.0		umhos/cm		4	17-NOV-15
MOISTURE-WT Soil								
Batch	R3312766							
WG2215461-2 LCS								
% Moisture			98.6		%		90-110	18-NOV-15
WG2215461-1 MB								
% Moisture			<0.10		%		0.1	18-NOV-15
PH-WT Soil								
Batch	R3312709							
WG2215439-1 DUP		L1702558-2						
pH		7.64	7.48	J	pH units	0.16	0.3	17-NOV-15
WG2215740-1 LCS								
pH			6.96		pH units		6.7-7.3	17-NOV-15
RESISTIVITY-WT Soil								
Batch	R3312722							
WG2215421-1 DUP		L1702558-1						
Resistivity		8330	8260		ohm cm	0.8	25	17-NOV-15
SO4-WT Soil								
Batch	R3313157							
WG2215497-4 CRM		AN-CRM-WT						
Sulphate			111.9		%		70-130	17-NOV-15
WG2215497-3 DUP		L1702558-1						
Sulphate		74	97		mg/kg	27	30	17-NOV-15

Quality Control Report

Workorder: L1702558

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Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
SO4-WT	Soil							
Batch	R3313157							
WG2215497-2	LCS							
Sulphate			97.2		%		70-130	17-NOV-15
WG2215497-1	MB							
Sulphate			<20		mg/kg		20	17-NOV-15

Quality Control Report

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Legend:

Limit	ALS Control Limit (Data Quality Objectives)
DUP	Duplicate
RPD	Relative Percent Difference
N/A	Not Available
LCS	Laboratory Control Sample
SRM	Standard Reference Material
MS	Matrix Spike
MSD	Matrix Spike Duplicate
ADE	Average Desorption Efficiency
MB	Method Blank
IRM	Internal Reference Material
CRM	Certified Reference Material
CCV	Continuing Calibration Verification
CVS	Calibration Verification Standard
LCSD	Laboratory Control Sample Duplicate

Sample Parameter Qualifier Definitions:

Qualifier	Description
J	Duplicate results and limits are expressed in terms of absolute difference.
RPD-NA	Relative Percent Difference Not Available due to result(s) being less than detection limit.

Hold Time Exceedances:

All test results reported with this submission were conducted within ALS recommended hold times.

ALS recommended hold times may vary by province. They are assigned to meet known provincial and/or federal government requirements. In the absence of regulatory hold times, ALS establishes recommendations based on guidelines published by the US EPA, APHA Standard Methods, or Environment Canada (where available). For more information, please contact ALS.

The ALS Quality Control Report is provided to ALS clients upon request. ALS includes comprehensive QC checks with every analysis to ensure our high standards of quality are met. Each QC result has a known or expected target value, which is compared against pre-determined data quality objectives to provide confidence in the accuracy of associated test results.

Please note that this report may contain QC results from anonymous Sample Duplicates and Matrix Spikes that do not originate from this Work Order.

2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: , DST Consulting Engineers Inc.

September 16, 2015

Site Coordinates: 48.5451 North 89.5664 West

User File Reference: 48.545187,-89.566437

National Building Code ground motions:

2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.095	0.057	0.026	0.008	0.036

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. *These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.011	0.035	0.055
Sa(0.5)	0.007	0.022	0.034
Sa(1.0)	0.003	0.011	0.016
Sa(2.0)	0.001	0.003	0.005
PGA	0.003	0.011	0.019

References

National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3

Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Aussi disponible en français

