

FINAL REPORT

**Foundation Investigation and Design Report
Algoma Central Rail CR Overhead (Site No. 38C-006)
Hwy 17, Wawa Area
WO 5009-E-0060
MTO GEOCRES No. 41N-18**

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March 31, 2011

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1. Part I: FOUNDATION INVESTIGATION

1.1 Introduction

This report presents the results of a geotechnical investigation completed by Trow Associates Inc. (Trow) for the Algoma Central Rail Overhead Rehabilitation (Site No. 38C-006), Lendrum Township, Wawa Area. The existing structure consists of a single span bridge with free span of 12 m and an overall length of 13.4 m.

The work was undertaken under Agreement # 5006-E-0060. The terms of reference were as presented in MTO letter dated January 24, 2011.

The purpose of the investigation is to examine the existing soil conditions within the area of the existing bridge and foundation supports. The site specific geotechnical investigation consisted of test borings, borehole logging, and field and laboratory testing. This foundation investigation report has been prepared specifically and solely for the project described herein. It contains the factual results of the investigation and the laboratory testing.

1.2 Site Description and Geological Setting

1.2.1 Site Description

The site is located on Hwy 17, approximately 3.3 km north of Hwy 101 in the Township of Lendrum, approximately 3.3 km north of Wawa, in the District of Algoma. There is an existing bridge at the site that crosses the historic railway line. The bridge consists of a single span concrete structure set on concrete abutments. The roadway is paved and guiderail is present of the roadway edge on each side. Rock outcrops are visible just north of the bridge structure and the vegetation in the immediate area consists of small to medium sized coniferous and deciduous trees with small shrubs.

The site plan is as shown on the drawings in Appendix B.

1.2.2 Geological Setting

According to the Ontario Ministry of Natural Resources (MNR), Map 5010, the regional surficial geology in the area consists of sand and gravel glaciofluvial outwash plain (valley train), with a moderate local relief (terraced) and dry surface conditions

According to Bedrock Geology of Ontario Map 2543 (Ministry of Northern Development and Mines, Ontario), the bedrock underlying the site is from the Archean Age and Neo-to Mesoarchean geologic era (approximately 2.5 to 3.4 billion years old) and typically consists of felsic to intermediate metavolcanic rock.

1.3 Investigation Procedures

1.3.1 General

The field work for this investigation was performed between March 15 and March 20, 2011 and consisted of drilling four (4) sampled boreholes (BH-1, BH-2, BH-3, and BH-4). The four (4) boreholes were strategically located adjacent to the existing abutments to permit geotechnical investigation of the foundations. BH-1 was advanced from the road surface and was located on the south-east side of the bridge. BH-2 was advanced from the old CP rail bed surface and was located on the northeast side of the bridge. BH-3 and BH-4 were also advanced from the old CP rail bed surface and were located on the southwest and northwest side, respectively, of the bridge. Site photographs of the borehole locations are provided in Appendix A. Drawing No. 1 in Appendix B shows the locations of the four (4) boreholes.

All boreholes were advanced using a Morooka track mounted CME 55 drill rig, equipped with continuous flight hollow stem augers. All borehole drilling/sampling were operated by a specialist drilling contractor, Abraflex Drilling, also an MOE Licensed Well Drilling Contractor.

During the drilling, soil samples were obtained using a 51 outside diameter (O.D.) split-spoon sampler with automatic trip hammer, in accordance with Standard Penetration Test (SPT) procedures (ASTM D 1586), at intervals shown on the attached borehole logs (Appendix C). The SPT “N” values were recorded and used to provide an assessment of in-situ consistency or compactness of the non-cohesive soils.

After completion, boreholes were backfilled by the drilling contractor, Abraflex, with the native soils removed by the augers. This is an acceptable procedure if groundwater is not encountered (as was the case here), in accordance with O.Reg. 903.

The fieldwork was supervised by a member of Trow’s engineering staff who directed the drilling and sampling operation, logged borehole data in accordance with MTO Soils Classification System for foundation reports, and retrieved soil samples for subsequent laboratory testing and identification. All of the recovered soil samples were placed in moisture-proof bags and returned to Trow’s Thunder Bay laboratory for additional visual, textual and olfactory examination.

Details of the soil strata encountered in the boreholes are included in attached borehole log sheets in Appendix C, and plotted on the profiles in Appendix B.

The borehole locations were surveyed by Trow personnel, with reference to the benchmark located adjacent to the site (GBM 0011969U303), with an elevation of 290.329 m.

1.3.2 Laboratory Testing

All samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included natural water content (LS-701) and grain size distribution tests (LS703/704) on approximately 25% of the collected soil samples. Hydrometer analyses and Atterberg limit testing was not performed since the soils were entirely non-plastic.

The laboratory test results are provided on the attached borehole log sheets in Appendix C. The results of the grain size analyses are presented geographically in Appendix D.

1.4 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the borehole log sheets in Appendix C. Laboratory test results are provided in Appendix D. The “Explanation of Terms Used in Report” preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report.

A borehole location plan and cross section soil profiles are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole log and cross section soil profiles are inferred from non-continuous sampling, observations of drilling progress and results of Standard Penetration Tests. These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Further, subsurface conditions may vary between and beyond the borehole locations.

In general, the stratigraphic sequence at the site typically consists of sand and gravel fill material that is underlain by sand to sand and silt.

A summary of the soil and groundwater conditions encountered in the boreholes is provided below.

1.4.1 Asphalt

Asphaltic concrete was encountered at ground surface in BH-1. The thickness of the asphaltic concrete layer was 0.1 m, and the top elevation of this layer is about 290.3 m.

1.4.2 Sand and Gravel (Fill)

Sand and gravel fill was encountered in all boreholes completed during the site investigation. At BH-1 the sand and gravel was encountered underlying the asphalt and in BH-2, BH-3 and BH-4, that were completed from the historic rail bed surface, the sand and gravel was encountered at the ground surface. The thickness of the sand and gravel fill was

approximately 11.3 m in BH-1 and extended from elevation 290.2 m to 278.9 m. The sand and gravel fill encountered in BH-2, BH-3 and BH-4, advanced from the historic rail bed surface, ranged in thickness from 0.2 m to 1.7 m and extends from elevation of about 281.1 m to 279.4 m.

The composition of this layer is sand and gravel to gravel and sand, trace to some silt, with occasional cobbles. The sand and gravel fill is brown to grey in colour, and frozen to damp and contained some to trace of organics. Uncorrected STP “N” value ranges from 21 to 100 blows per 300 mm in BH-1, completed from the road surface, classifying the material as compact to very dense in compactness condition. The sand and gravel layer encountered at the surface of the historic rail bed was frozen resulting in collection of one (1) uncorrected SPT “N” value during the site investigation. The resultant SPT “N” value was eight (8) that classifies the sand and gravel fill in this area as loose.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 4% to 9%

Grain Size Distribution:

- 39% to 52% gravel;
- 40% to 51% sand; and
- 8% to 11% silt and clay size

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheet in Appendix C. The results of the grain size distribution tests are also provided on Figure 1 in Appendix D.

1.4.3 Sand to Silt

Sand was encountered in all boreholes advanced during the site investigation. Layers of finer material, consisting of varying degrees of silt content, were also encountered within the sand layer.

The sand was encountered underlying the sand and gravel fill and had a thickness ranging from about 12.1 m to 15.7 m. It extends to depths between 14.5 m and 23.5 m from the ground surface, with approximate elevations ranging from 265.1 m to 266.8 m. Advancement was terminated at depth of 23.5 m in BH-1 and at 15.9 m in BH-2 and BH-3. Advancement in BH-4 was terminated at 14.7 m depth due to refusal to the augers.

Layers or zones of sand with increased silt content were encountered in BH-1 at 11.4 m depth to 12.7 m depth and also at 13.7 m depth to 15.3 m depth. Sand with increased silt

content was also encountered in BH-2 at depths of 3.1 m to 4.6 m, in BH-3 at 4.6 m to 9.2 m and in BH-4 at depths of 1.7 m to 9.2 m.

The sand deposit consists of sand, some silt to sand and silt, trace gravel and was fine to medium grained. A 120 mm layer of wet sand and silt was encountered in BH-3 at approximately 6.1 m deep and a 100 mm layer of wet sand and silt was encountered in BH-4 at approximately 3.0 m deep. Interbedded wet sand and silt layers were also encountered in BH-4 at approximately 3.8 m deep. The sand is brown in color, and was frozen to moist to damp. Uncorrected SPT “N” values range from 5 to 37 blows per 300 mm, classifying the sand as loose to dense in compactness condition.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 3% to 17%

Grain Size Distribution:

- 0% gravel;
- 51% to 89% sand; and
- 11% to 49% silt and clay size

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheet in Appendix C. The results of the grain size distribution tests are also provided on Figure 2 in Appendix D.

As mentioned above, the sand layer contained layers or zones with higher silt contents. These layers consisted of silt, some sand to silt and sand. Occasional clay seams, 5 mm to 10 mm thick, with trace organics and trace oxidation were noted in BH-1 at approximately 11.4 m deep. The silt and sand was brown in color, moist and was fine grained. Uncorrected SPT “N” values range from 8 to 15 blows per 300 mm, classifying the silt and sand layers as loose to compact in compactness condition.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 10% to 31%

Grain Size Distribution:

- 0% gravel;
- 14% to 47% sand; and

- 53% to 87% silt and clay size

The results of the moisture content and grain size distribution tests are provided on the record of borehole sheet in Appendix C. The results of the grain size distribution tests are also provided on Figure 3 in Appendix D.

1.5 Groundwater Conditions

Groundwater or standing water was not encountered in the boreholes advanced during the site investigation. Increased moisture contents were observed in finer grained soils sampled during the site investigation. It should be noted that the groundwater levels are subject to seasonal fluctuations and could become established within the soil subsurface after extended periods of precipitation or potentially after the spring melt and freshet has occurred.

2. Part II: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

2.1 Introduction

The purpose of the following subsections is to provide recommendations for the design and construction of the foundations to support the proposed Highway 17 replacement bridge structure over the former Algoma Central Railway (ACR) about 3 km north of Wawa, Ontario . We understand that the foundations for the replacement bridge structure may be replaced entirely, or the existing foundations modified to accommodate the proposed loads.

The present bridge abutment foundations are concrete spread footings about 5.8 m wide by 12.8 m long by 0.9 m to 1.2 m thick, at depths of between about 2 m and 3 m below the rail bed grade.

This report addresses the geotechnical design of the foundation for the bridge by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS) as well as other geotechnical parameters that may be required in accordance with the *Canadian Highway Bridge Design Code (CHBDC)* (November 2006), the *Canadian Foundation Engineering Manual (CFEM)* (2006), and good practice.

2.2 Geotechnical Design Considerations for Foundations

2.2.1 General

The geotechnical investigation and its findings pertaining to the subsurface soil characteristics have been covered in **Part I - Foundation Investigation Report** which contains details of the field and laboratory aspects of the investigation. In general, the natural stratigraphic sequence at the site typically consists of a deposit of sand and silt with thickness in excess of 15 m. The approach fill typically consists of about 11 m of sand and gravel fill.

In the context of the *Canadian Highway Bridge Design Code (CHBDC)*, a satisfactory foundation design would require, in terms of Limit States Design, the factored geotechnical resistance of its foundation to withstand and not exceed the imposed Ultimate Limit State loads - (ULS) Design Approach, and its ability to deform acceptably under the Service Limit State loads - (SLS) Design Approach. These associated loads are typically known as unfactored and factored loads, respectively.

The foundation recommendations for the proposed construction in this project were developed based on:

- Soil conditions encountered in the geotechnical soil borings performed for this study,
- Review of the original foundation investigation report by Racey, MacCallum and Associates, 12 November, 1956,
- Review of foundation investigation report by MTO, August 22, 1975.

2.2.2 Geotechnical Resistance at Ultimate Limit States

The Factored Ultimate Limit States (ULS) bearing resistances for footings constructed on the undisturbed native soil, with a width ranging between 3 m and 6 m and founded at a depth ranging between 2.4 m and 4.7 m are shown in Table 1, below. The presumed footing length is between 12 m and 13 m, similar to that which reportedly exists.

TABLE 1: Factored ULS Bearing Resistance (in kPa)				
Footing Width (m)	Footing Depth (m)			
	2.4	3.2	3.7	4.7
3	880	1110	1270	1595
4	935	1205	1385	1670
5	995	1260	1435	1805
6	1055	1320	1495	1860
Resistance factor of 0.5 included in above values				

The existing footings are reportedly 5.8 m wide by 12.8 m long. Based on the McIntosh Perry General Arrangement drawing provided, the undersides of the north and south abutment footings have been measured to be approximately 3.2 m and 4.7 m below grade, respectively. **Based on the results in Table 1, above, the factored ULS resistances for the north and south footings are 1320 kPa and 1860 kPa, respectively.**

2.2.3 Geotechnical Reaction at Serviceability Limit State

Serviceability Limit States (SLS) generally consider the unfactored permanent and transitory loads being used to determine total and differential settlements of the structure with the magnitude of unfactored loads and tolerable total and differential settlement limits being established by the Structural or Design Engineer.

In determining the settlement characteristics of the proposed building, the unfactored loads are required to be provided by the Structural or Design Engineer. The geotechnical reaction at the Serviceability Limit States can be determined by several methods outlined in the various geotechnical literature, including the *Canadian Highway Bridge Design Code* and the *Canadian Foundation Engineering Manual*.

Based on the method of Burland and Burbidge, outlined in the CFEM (4th Ed.), and the corrected SPT values from the current geotechnical investigation, averaging between about 14 and 15 within the potential depth of influence of footings, the SLS values on native soil presented in Table 2, below, have been calculated for deflections of 25 mm and 35 mm.

TABLE 2: SLS Bearing Reaction on Native Soil (in kPa)		
Footing Width (m)	For 25 mm Deflection	For 35 mm Deflection
3	290	400
4	235	325
5	195	275
6	170	240
For footing depths between 1.8 m and 5 m.		

We have also considered the fact that the soil beneath the existing footings has been preloaded with the working stress pressures of between 150 kPa and 190 kPa, more or less equivalent to SLS, assuming that the recommendations of the previous 1956 and 1975 foundation investigation reports were followed. These SLS reactions are shown in Table 3, below, and are based on a modified Burland and Burbidge approach, considering the preconsolidation stress due to the existing footing pressure, assumed to be 150 kPa.

TABLE 3: SLS Bearing Reaction on Soil Beneath Existing Foundations (in kPa)		
Footing Width (m)	For 25 mm Deflection	For 35 mm Deflection
3	390	500
4	335	425
5	295	375
6	270	340
For footing depths between 1.8 m and 5 m.		

Based on the results shown in Table 3, the SLS bearing reaction for the existing abutment footings is 270 kPa.

If the existing foundations are refurbished or replacement footings are constructed in the locations of the existing foundations, Table 3 SLS reactions may be used.

Since the subsoils are cohesionless and groundwater appears to be well below the depth of influence of foundation stresses, any settlements should be largely complete shortly after the permanent dead loads are applied. Subsequent loadings due to transitory loads will cause very minor additional settlement.

Differential settlements should be within 20 mm.

2.2.4 Frost Protection

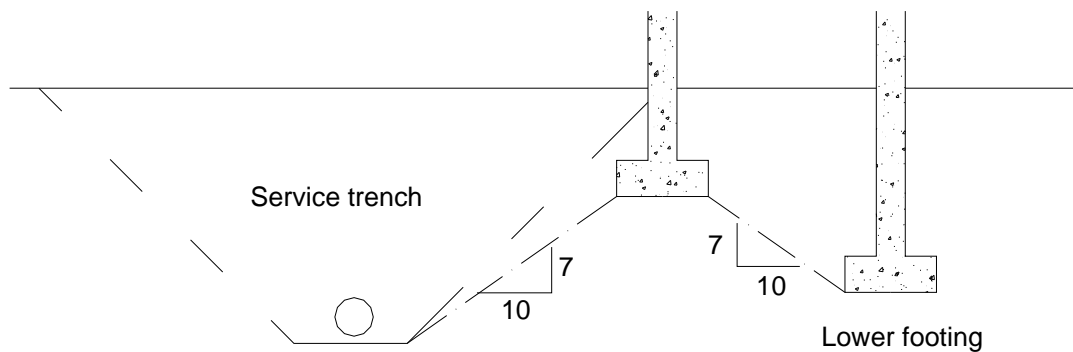
The potential frost depth at this site is about 2.4 m. Consequently, all footings exposed to seasonal freezing conditions should be protected from frost action by at least 2.4 m of soil cover or equivalent insulation. While the native soils are considered frost susceptible, there was no groundwater encountered within a depth of 15 m to cause frost heave. However, it is considered prudent to provide this cover to account for infiltrating water during freeze-thaw, which may cause some amount of frost heave.

2.2.5 Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of mass concrete and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The unfactored coefficient of friction for concrete cast directly on the undisturbed or recompacted native soils can be taken as 0.4 for use in design.

2.2.6 Foundation Elevation

If footings are to be placed at different elevations they should be located such that the higher footings are set below a line drawn up at 10 horizontal to 7 vertical from the near edge of the lower footing, as indicated on the following sketch.



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

This concept should also be applied to excavations for new foundations in relation to existing footings or underground services. Lower footings should be placed prior to upper foundations to prevent undermining conditions

Where footings are stepped down, a maximum level difference of 600 mm should be maintained.

2.2.7 Backfill and Lateral Earth Pressures for Design

Backfill against the abutments and wing walls should be a clean, free draining, non-frost susceptible granular material, such as OPSS Granular B, to reduce the effects of adfreeze, and in accordance with OPSD 3101.150. The existing fill, based on the results from BH-1, is non-frost susceptible and may be re-used. The existing native sand and silt, however, is too fine based on the grain size analysis and should not be used as backfill. Based on the grain size analysis, this soil is considered frost susceptible. Select fill as currently exists behind the abutments may be re-used.

The fill should be placed in lifts not greater than 300 mm in thickness and compacted to not less than 95% SPMDD. Smaller lifts and lighter compaction equipment should be used immediately adjacent to concrete walls to prevent overstressing and damage. Care should be taken to place and compact fill simultaneously on both sides of any subwalls.

For design, the soil parameters provided in Table 3, below, may be used.

Table 3. Earth Pressure Parameters		
Parameter	Granular B Type I	Granular B Type II and Granular A
Friction Angle, ϕ'	32	35
Active EP Coeff., K_a	0.31	0.27
Passive EP Coeff., K_p	3.3	3.7
At Rest EP Coeff., K_0	0.47	0.43
Unit Weight, γ , kN/m ³	19	21
Friction Factor	0.40	0.45

2.2.8 Liquefaction Considerations

The subsoil at the site mainly consists of granular materials (i.e., sand and silt) with an average N-value of about 18, to the maximum depths of about 15 m, investigated. The water level is at a depth greater than 15 m. Accordingly, this subsoil is not subject to liquefaction.

2.2.9 Earthquake Considerations

Based on soil conditions observed during the geotechnical investigation, and published information for the area, the governing subgrade soils within the upper 30 m are, in accordance with the NBC 2005, generally characterized as stiff soils ($15 < \tilde{N}_{60} < 50$ or $50 < S_u < 100$ kPa). In this regard, the Site Classification for Seismic Site Response is Site Class D, as shown on Table 4.1.8.4.A in the 2005 NBC.

This is considered to equate to a Soil Profile Type I as outlined in Section 4.4.6.2 of the CHBDC. However, according to 4.4.5.2.1 of the CHBDC, seismic analysis is not required for most single span bridges. The structural engineer should ensure that the structural design incorporates the necessary requirements.

2.3 Excavation and Groundwater Control

For the construction of the proposed foundations, excavations at least about 2 m depth will be required. The excavations are expected to encounter mostly sand and silt, although coarser (sand and gravel and possibly cobbles) will be encountered when excavating in the embankment fill.

All excavations should be carried out in accordance with the latest version of the Occupational Health and Safety Act. For the purpose of the Act, the existing materials are considered as Type 3 soils.

No unusual construction conditions are expected for the excavations in the native soils. Heavy duty equipment will be required to excavate the sand and gravel and progress could be slow. A Non-Standard Special Provision should be included in the contract documents to alert the Contractor of the possible presence of cobbles that may interfere with or slow the progress of excavation at some areas.

Excavations are not expected to encounter any significant groundwater, as no static groundwater level was encountered in the investigation. Accordingly, no special groundwater control measures would be required.

A representative of Trow should be on-site during the foundation installation and for any fill material placement, to verify the design assumptions, and to verify the design recommendations.

2.4 Closure

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. could be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the works, should, in this light, decide on their own investigations as well as their own interpretations of the factual borehole results so that they may draw their own conclusions as to how the subsurface conditions may affect them.

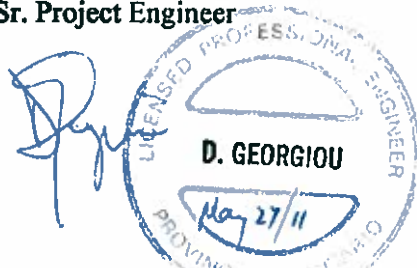
This Foundation Investigation and Design Report has been prepared by Ben Plumridge, P.Eng. and Demetri N. Georgiou, P.Eng., and reviewed by S.E. Gonsalves, P.Eng., Designated MTO Foundations Contact.

We trust that these comments provide you with sufficient information to proceed with design. Should you have any questions, please do not hesitate to contact this office.

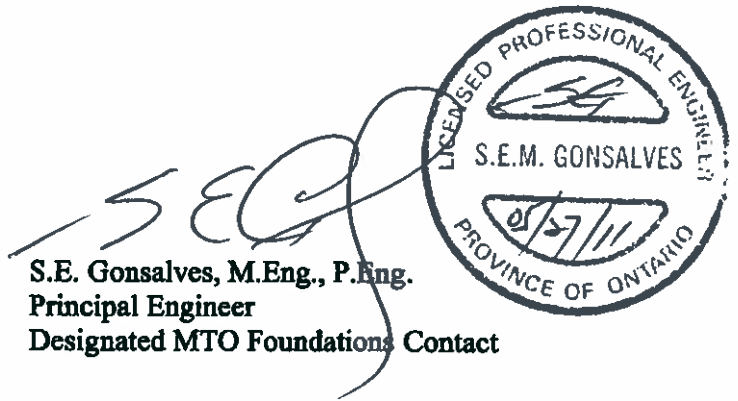
Yours truly,

Trow Associates Inc.

**Ben Plumridge, P.Eng.
Sr. Project Engineer**



**Demetri N. Georgiou, M.A.Sc., P.Eng.
Principal Engineer
Thunder Bay Branch Manager**



**S.E. Gonsalves, M.Eng., P.Eng.
Principal Engineer
Designated MTO Foundations Contact**

Encl.

APPENDIX A: PHOTOGRAPHS



Photo No. 1: Set-up at BH-1 located on south-east side of bridge. Borehole advanced from Hwy 17 shoulder. View looking north.



Photo No. 2: Auger cuttings from BH-1 consisting of sand and gravel fill with cobbles, that is underlain asphalt.



Photo No. 3: Local GBM at the site used for survey elevations of boreholes.



Photo No. 4: Set-up at BH-2 located on north-east side of bridge. Borehole advanced from historic railway surface. View looking north-east.



Photo No. 5: Set-up at BH-3 located on south-west side of bridge. Borehole advanced from historic railway surface. View looking south-west.

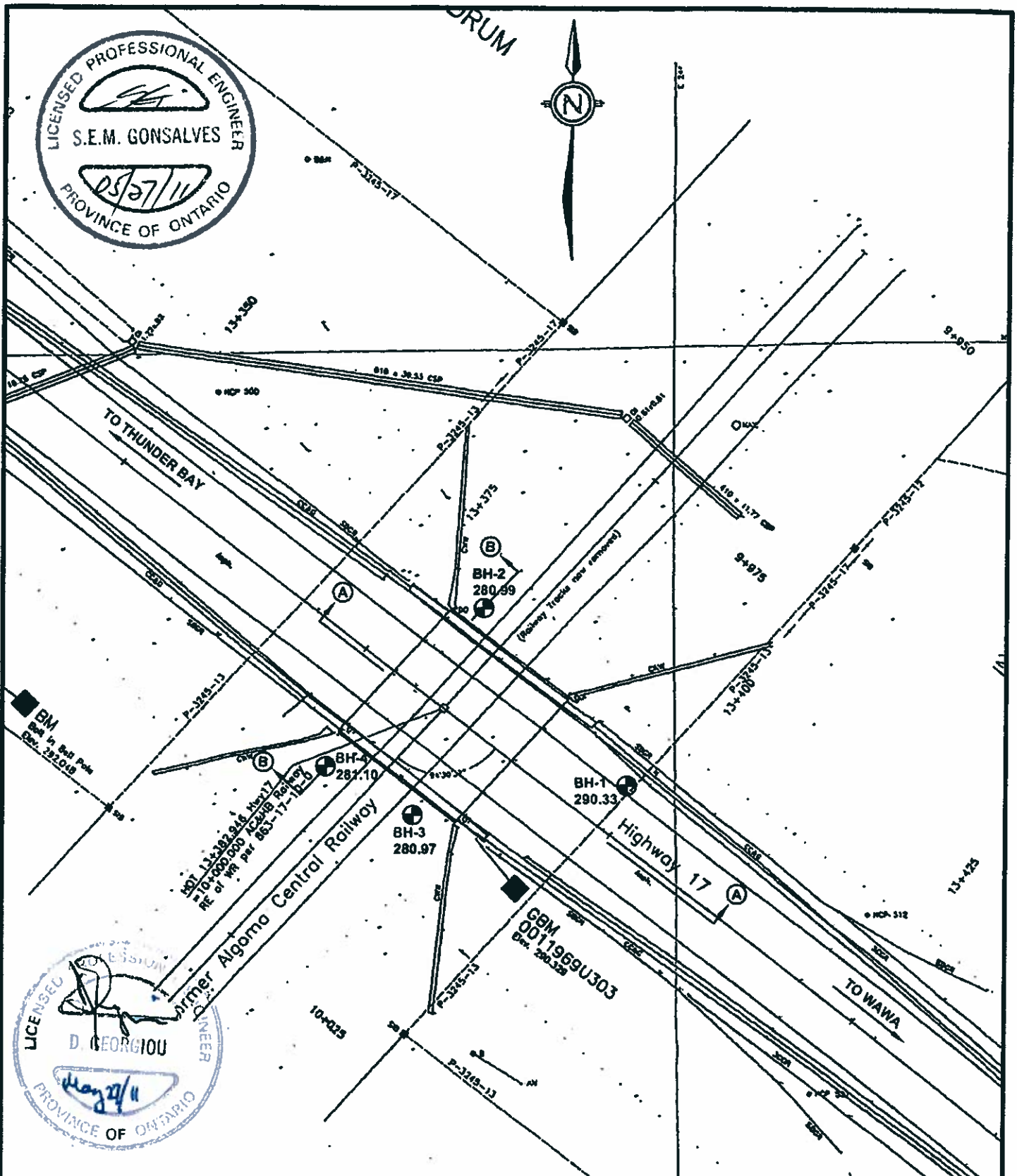


Photo No. 6: Set-up at BH-4 located on north-west side of bridge. Borehole advanced from historic railway surface. View looking north-west.

APPENDIX B: DRAWINGS



 Trow Associates Inc. Thunder Bay, Ontario		FIGURE 1
SITE LOCATION PLAN Foundation Investigation & Design Algoma Central Rail-CR Overhead (Site No. 38C-006) GeoCres No. 41N-18 Wawa, ON		PROJECT NO.: ADM-00011658-A0
		SCALE: 1:25000
		DRAWN BY: DT
		CHECKED BY: BP
		DATE: MARCH 31, 2011



NOTES:

1) REFERENCE: BASE PLAN PROVIDED BY CLIENT

LEGEND:

● BH1 BOREHOLE LOCATION
290.33 ELEVATION IN METRES



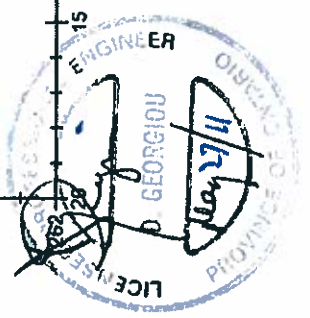
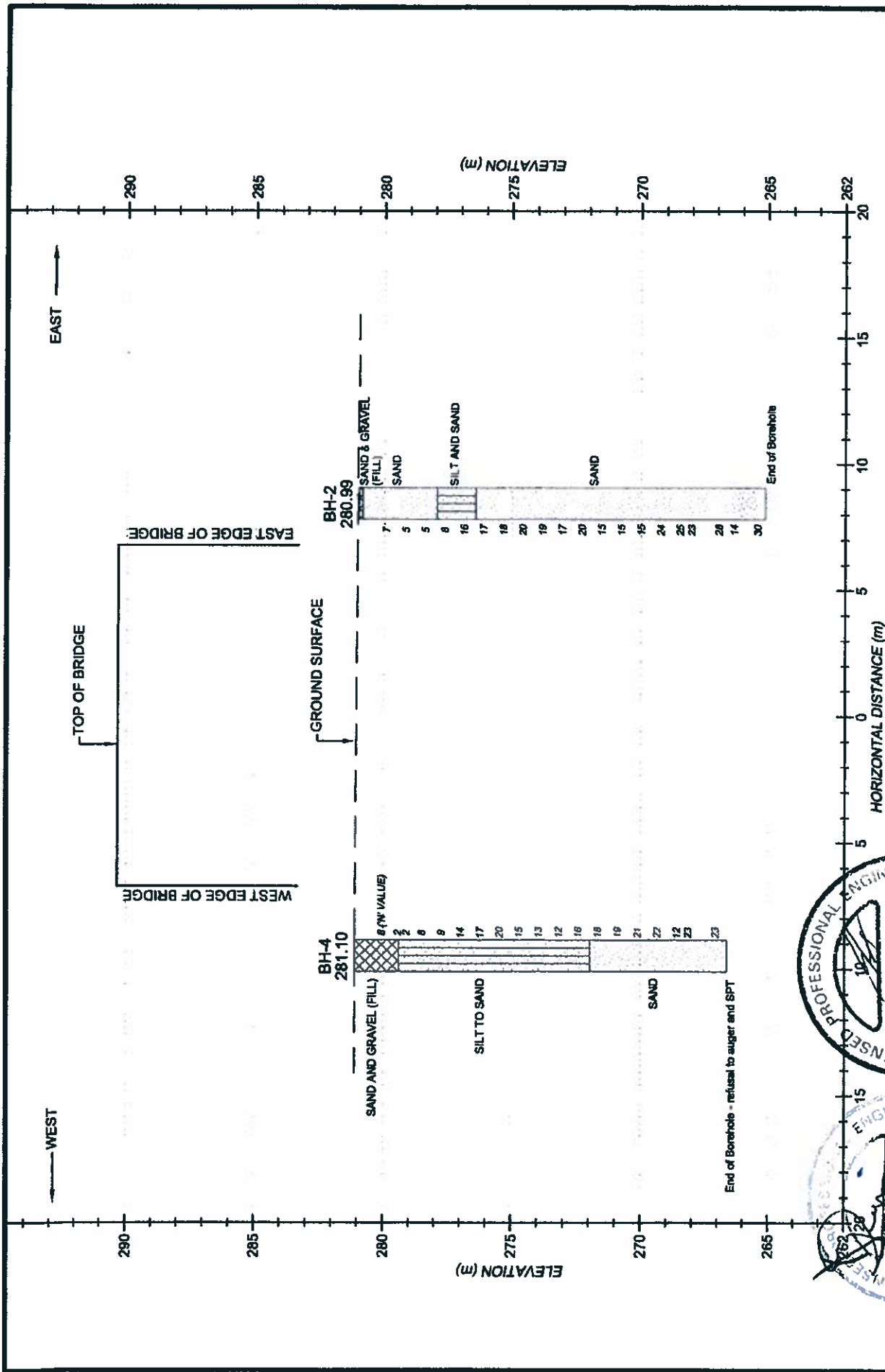
Trow Associates Inc.
Thunder Bay, Ontario

FIGURE
2

BOREHOLE LOCATION PLAN

Foundation Investigation & Design
Algoma Central Rail-CR Overhead
(Site No. 38C-008)
GeoCres No. 41N-18
Wawa, ON

PROJECT NO.: ADM-00011858-A0
SCALE: 1:400
DRAWN BY: DT
CHECKED BY: BP
DATE: MARCH 31, 2011



APPENDIX C: BOREHOLE LOGS

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

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

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

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

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

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}}$
P_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ'	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_p) / I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(w_L - w) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No BH-1

1 OF 1

METRIC

W.P. **5142-06-01** LOCATION **Algoma Central Rail CR Overhead (Site No. 38C-006)** ORIGINATED BY **EF**
 DIST **Algoma** HWY **17** BOREHOLE TYPE **CME 55 Trackout / HSA** COMPILED BY **AM**
 DATUM **Geodetic** DATE **3.15.11 - 3.17.11** CHECKED BY **BP**

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	"N" VALUE		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			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
290.3	Asphalt						290							
290.3	ASPHALT - about 100 mm		S1	AUGER			290							
	SAND AND GRAVEL (FILL) -		S2	AUGER										
	frozen, brown to grey, trace to some silt		S3	AUGER										
	- becoming brown, no SPT penetration due to frost at about 0.8 m depth		S4	SS	100									0 0 0
	- very dense, brown, damp at about 1.5 m depth		S5	SS	66		288							0 0 0
	- occasional cobbles at about 1.6 m depth		S6	SS	37									45 44 11
	- becoming dense at about 3.0 m depth		S7	SS	22									0 0 0
	- becoming compact, moist at about 3.8 m depth		S8	SS	47		286							41 48 11
	- becoming dense to very dense at about at 4.8 m depth		S9	SS	43									
			S10	SS	60		284							0 0 0
	- becoming compact to dense, moist at about 6.9 m depth, trace roots at about 6.9 m to 7.6 m depth		S11	SS	21									52 40 8
	- becoming damp, occasional cobbles at about 7.6 m depth		S12	SS	30		282							0 0 0
			S13	SS	32									0 0 0
			S14	SS	35									
			S15	SS	32		280							0 0 0
			S16	SS	24									
278.9			S17	SS	13		278							0 44 56
11.4	SILT AND SAND - compact to loose, brown, moist to wet, occasional 5 to 10 mm clay seam, trace organics, some oxidation		S18A	SS	8									0 0 0
277.7			S18B	SS	7									
12.7	SAND - loose to compact, brown, moist to wet, trace gravel, fine grained						276							0 18 82
276.6			S19	SS	9									
13.7	SILT - loose, brown, moist, some sand						274							0 0 0
275.1			S20	SS	25									
15.3	SAND - compact, light brown, moist, trace to some silt, very fine grained						272							0 89 11
			S21	SS	11									
			S22	SS	14		270							0 0 0
			S23	SS	26									
			S24	SS	37		268							0 0 0
			S25	SS	25									
266.8														
23.5	End of Borehole													
	- No groundwater encountered during drilling													
	- Borehole dry upon completion													

ONTARIO MOT F-11111 ALGOMA CENTRAL RAIL CP OVERHEAD SITE - WAWA - BRAMPTON GPJ ONTARIO MOT.GDT 4/20/11

RECORD OF BOREHOLE No BH-2

1 OF 1

METRIC

W.P. 5142-06-01 LOCATION Algoma Central Rail CR Overhead (Site No. 38C-006) ORIGINATED BY EF
 DIST Algoma HWY 17 BOREHOLE TYPE CME 55 Trackmout / HSA COMPILED BY AM
 DATUM Geodetic DATE 3.19.11 - 3.19.11 CHECKED BY BP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	"N" VALUE		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			20	40	60	80	100		
281.0	Sand and Gravel		S1	AUGER			280							
280.9	SAND AND GRAVEL (FILL) - frozen, brown, trace silt, some organics		S2	AUGER										
	SAND - frozen, dark brown, trace gravel in upper 0.8 m, trace silt, trace organics in upper 0.8 m, fine grained - becoming loose, light brown, damp at about 0.8 m depth		S3	SS	7									
			S4	SS	5									
			S5	SS	5									
277.9			S6	SS	8		278							
3.1	SILT AND SAND - loose to compact, light brown, moist, very fine grained		S7	SS	16									
276.4			S8	SS	17		276							
4.6	SAND - compact, light brown, moist, some silt to silty, very fine grained		S9	SS	18									
			S10	SS	20									
			S11	SS	19		274							
	- becoming fine grained at about 7.6 m depth		S12	SS	17									
			S13	SS	20		272							
			S14	SS	15									
			S15	SS	15									
			S16	SS	15		270							
			S17	SS	24									
			S18	SS	25									
			S19	SS	23		268							
			S20	SS	28									
			S21	SS	14		266							
			S22	SS	30									
265.1	End of Borehole													
15.9	- No groundwater encountered during drilling - Borehole dry upon completion													

ONTARIO MOT F-11111 ALGOMA CENTRAL RAIL CP OVERHEAD SITE - WAWA - BRAMPTON GP J. ONTARIO MOT GDT 4/20/11

RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W.P. 5142-06-01 LOCATION Algoma Central Rail CR Overhead (Site No. 38C-009) ORIGINATED BY EF
 DIST Algoma HWY 17 BOREHOLE TYPE CME 55 Trackmout / HSA COMPILED BY AM
 DATUM Geodetic DATE 3.19.11 - 3.19.11 CHECKED BY BP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	"N" VALUE			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 X LAB VANE							
281.0	Sand and Gravel							20 40 60 80 100							
0.0	SAND AND GRAVEL (FILL) - frozen, dark brown, trace organics	XXXX	S1	AUGER											
280.2															
0.8	SAND - loose to compact, brown to red, damp, trace gravel in upper 2.3 m, trace silt, fine grained - becoming brown to light brown at about 1.5 m depth - becoming moist, silty, very fine grained at about 2.3 m depth	S2	SS	10		280								
			S3	SS	10										
			S4	SS	5										
			S5	SS	5		278								0 72 28
			S6	SS	12										
276.4															
4.6	SILT - compact, light brown, moist, some sand		S7	SS	12		276								0 14 86
275.6															
5.4	SILT TO SAND - compact, light brown, moist, very fine grained - 120 mm layer of wet sand at about 6.1 m depth		S8	SS	18										
			S9	SS	17										
			S10	SS	14		274								0 47 53
			S11	SS	16										
			S12	SS	19										
271.8							272								
9.2	SAND - compact, light brown, damp to moist, trace to some silt, very fine grained	S13	SS	25										
			S14	SS	22										
			S15	SS	17		270								
			S16	SS	22										
			S17	SS	19										
			S18	SS	20		268								
			S19	SS	22										
			S20	SS	23		266								
265.1			S21	SS	17										
15.9	End of Borehole - No groundwater encountered during drilling - Borehole dry upon completion														

ONTARIO MOT F-11111 ALGOMA CENTRAL RAIL CP OVERHEAD SITE - WAWA - BRAMPTON GPJ ONTARIO MOT.GDT 4/20/11

RECORD OF BOREHOLE No BH-4

1 OF 1

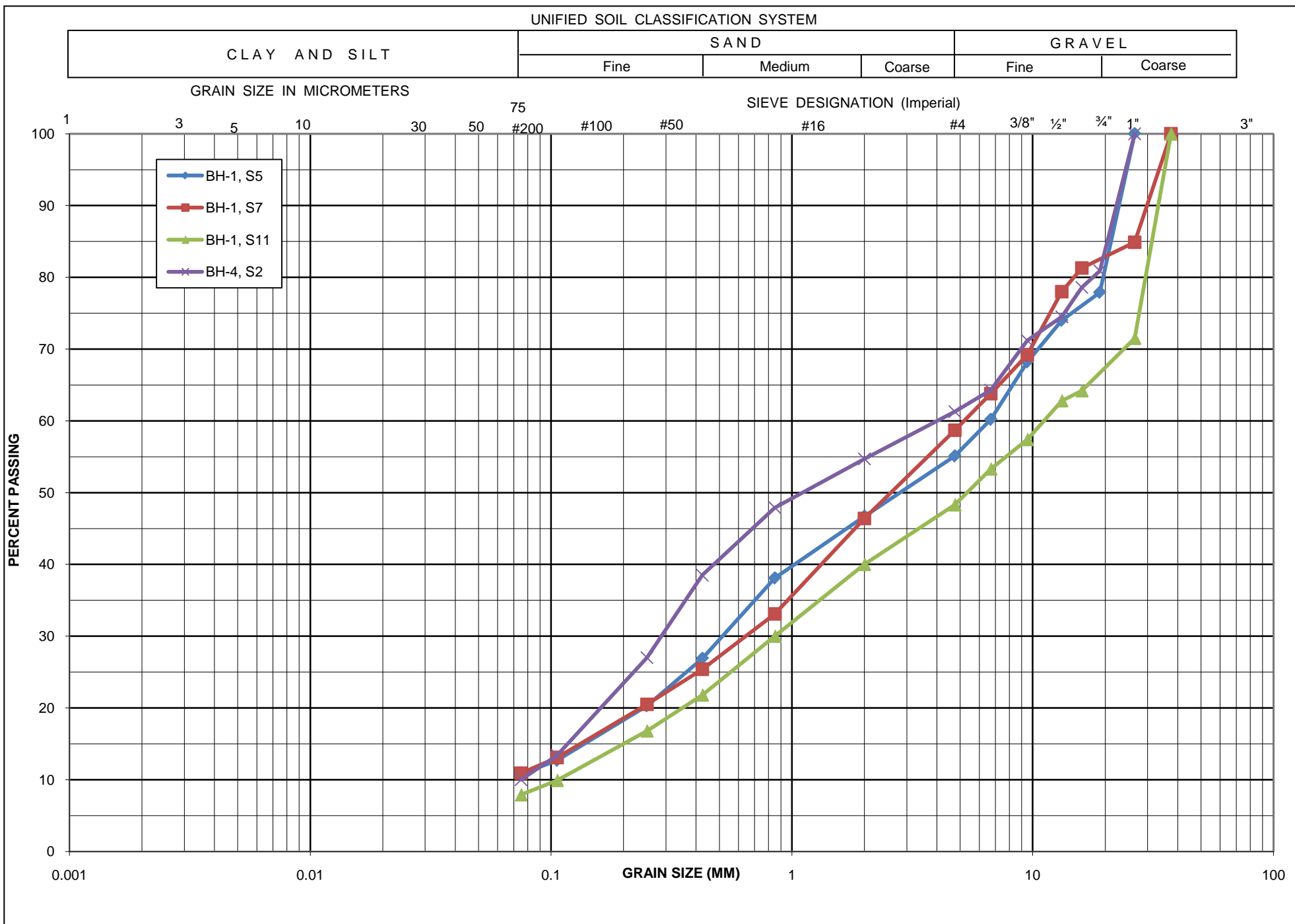
METRIC

W.P. 5142-08-01 LOCATION Algoma Central Rail CR Overhead (Site No. 38C-006) ORIGINATED BY EF
 DIST Algoma HWY 17 BOREHOLE TYPE CME 65 Trackmout / HSA COMPILED BY AM
 DATUM Geodetic DATE 3.20.11 - 3.20.11 CHECKED BY BP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	"N" VALUE		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			20	40					
281.1	Sand and Gravel						20	40	60	80	100		
0.0	SAND AND GRAVEL (FILL) - frozen, dark brown, trace silt, some organics - becoming loose, brown, damp at about 0.8 m depth		S1	AUGER									
279.4	- becoming very loose, trace peat at about 1.5 m depth		S2	SS	8								39 51 10
1.7			S3A	SS	2								0 0 0
	SILT TO SAND - loose, light brown, damp to moist, fine grained		S3B	SS	2								
			S4	SS	8								0 76 24
	- becoming very fine grained, 100 mm layer of wet sand and silt at about 3.0 m depth		S5	SS	9								0 0 0
	- becoming compact, moist, interbedded wet sand and silt layers at about 3.8 m depth		S6	SS	14								
			S7	SS	17								0 51 49
			S8	SS	20								
	- wet layers at about 6.1 m depth		S9	SS	15								0 13 87
			S10	SS	13								0 61 39
	- some silt to silty, fine grained at about 7.6 m depth		S11	SS	12								0 0 0
272.0			S12	SS	16								
9.2	SAND - compact, brown, damp to moist, trace to some silt, fine grained		S13	SS	18								0 86 14
			S14	SS	19								
			S15	SS	21								0 89 11
			S16	SS	22								
			S17	SS	12								0 0 0
			S18	SS	23								
266.6			S19	SS	23								0 0 0
14.5	End of Borehole - refusal to auger and SPT - No groundwater encountered during drilling - Borehole dry upon completion												

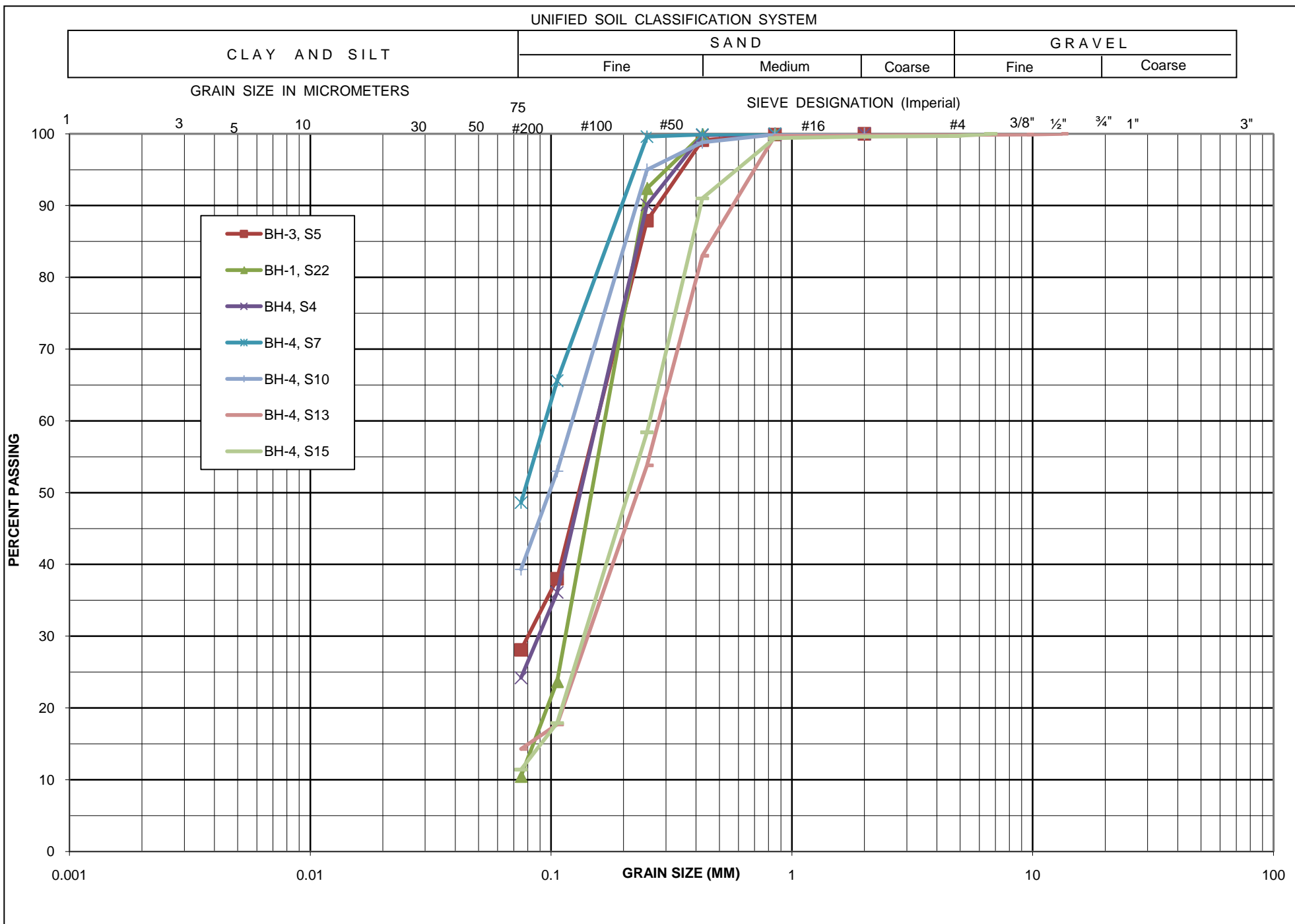
ONTARIO MOT F-11111 ALGOMA CENTRAL RAIL CP OVERHEAD SITE - WAWA - BRAMPTON.GPJ ONTARIO MOT GDT 4/20/11

APPENDIX D: LABORATORY DATA



GRAIN SIZE DISTRIBUTION - Sand and Gravel (Fill)
 Foundation Investigation and Design Algoma Central Rail CP Overhead (Site
 No. 386-006) - Geocres No. 41N-18, Wawa, Ontario

FIGURE No. 1
 Ref. No. ADM-00011658-AO
 DATE March 30, 2011

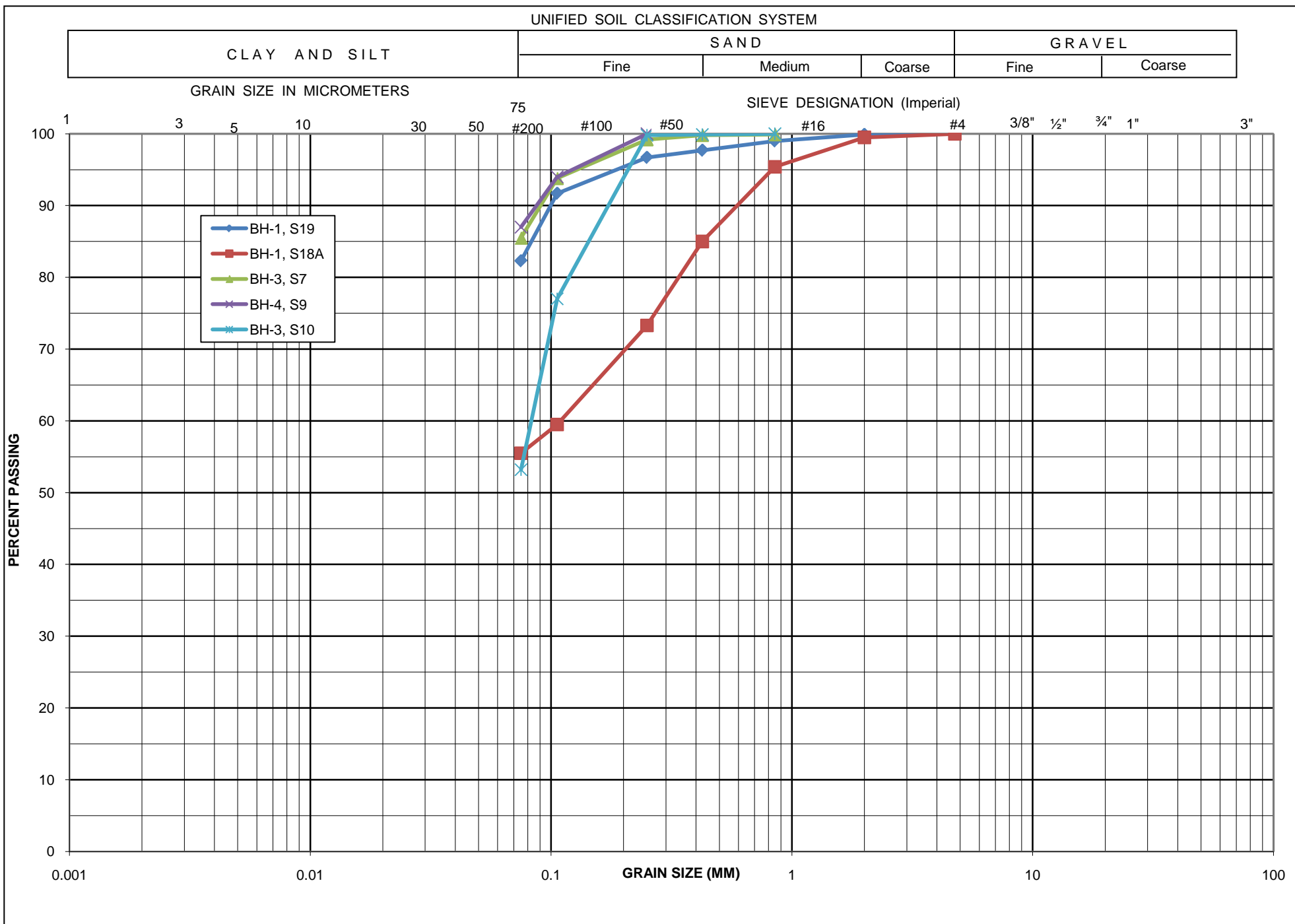


GRAIN SIZE DISTRIBUTION - Sand
 Foundation Investigation and Design Algoma Central Rail CP Overhead (Site
 No. 386-006) - Geocres No. 41N-18, Wawa, Ontario

FIGURE No. 2

Ref. No. ADM-00011658-AO

DATE: March 30, 2011



GRAIN SIZE DISTRIBUTION - Silt and Sand
 Foundation Investigation and Design Algoma Central Rail CP Overhead (Site
 No. 386-006) - Geocres No. 41N-18, Wawa, Ontario

FIGURE No. 3
 Ref. No. ADM-00011658-AO
 DATE: March 30, 2011