

FINAL REPORT

**Foundation Investigation and Design Report
Storage Building at Walden Patrol Yard
- Waters Township, RR 55, Sudbury Area
WO 2011-11001, District 54
MTO GEOCRES No. 41I-271**

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March 30, 2011

ADM-00011530-A0

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1. Part I: FOUNDATION INVESTIGATION

1.1 Introduction

This report presents the results of a geotechnical investigation carried out by Trow Associates Inc. (Trow) for the proposed new storage structure located at Walden Patrol Yard, Waters Township on Sudbury Regional Road 55, District 54, Sudbury Area. The proposed 24.4 m x 42.7 m storage structure will replace the existing two sand domes. The structure will allow for inside loading and dumping. It will be similar to the building structures constructed at the Cartier Patrol Yard located on Hwy 144 in Sudbury Area for which Trow carried out Foundation Investigation and Design.

The work was undertaken under Agreement # 5009-E-0060, Assignment No. 1. The terms of reference were as presented in the Ministry of Transportation (MTO) letter dated December 03, 2010.

The purpose of the investigation is to establish the existing subsurface conditions at the proposed location of the Patrol Yard structure within the construction limits. The site specific geotechnical investigation was carried out by means of borehole drilling, bedrock coring, in situ testing, and subsequent geotechnical laboratory testing on selected samples. 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 proposed Patrol Yard is located on Sudbury Regional Road 55 in the Township of Waters approximately 2 km north of the Highway 17 interchange, MTO Northeastern Region (see Key Map on Drawing 1). The terrain at the structure site is relatively flat as shown on the photographs in Appendix A. At about 50 m northeast of the proposed structure, there is a steep depression. At present, there are two 30 m diameter sand domes at the proposed structure site and will be replaced by the new storage structure.

The site plan is as shown on the drawings in Appendix B (from the site map PLAN H-564-17-1, provided by MTO).

1.2.2 Geological Setting

According to Bedrock Geology of Ontario Map 2544 (Ministry of Northern Development and Mines, Ontario), the bedrock underlying the site is from the Paleoproterozoic geologic era (approximately 1.6 to 2.5 billion years old) and falls under Southern and Superior rocks which consists of volcanic rocks including mafic, intermediate and felsic metavolcanic rocks, intercalated metasedimentary rocks and epiclastic rocks.

According to Surficial Geology Map by the Province of Ontario's Ministry of Northern Development, Mines and Forestry (MNDMF), the surficial deposit in this area is a discontinuous layer of drift Precambrian deposit.

1.3 Investigation Procedures

1.3.1 General

The current field investigation was carried out between February 1 and 11, 2011, during which time five (5) boreholes (BH-1, BH-2, BH-3, BH-4, and BH-5) were drilled. The locations of the boreholes were strategically located adjacent to the existing sand domes to permit geotechnical investigation for the foundation of the proposed new building. Drawing No. 1 in Appendix B shows the locations of five boreholes. The depths of the boreholes were: 18 m (BH-1), 17.2 m (BH-2), 14.3 m (BH-3), 39.7 m (BH-4) and 19.8 m (BH-5).

The boreholes were advanced using track-mounted CME 55 drill rig, equipped with continuous flight hollow stem augers. All borehole drilling/sampling were operated by a specialist drilling contractor, LandCore Drilling Co. Ltd. During the drilling operation, soil samples were obtained using a 51 mm outside diameter split-spoon sampler 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 relative density of non-cohesive soils. At BH-1 from 1.5 to 3.1 m, BH-3 below 9.1m, and BH-4 below 15.2 m depths, wash boring was utilized to facilitate taking representative samples at designated elevation with reasonable accuracy. A standpipe piezometer was installed in BH-3. Bedrock coring was performed in BH-2 and BH-4. NQ coring equipment was used to retrieve rock cores. After completion, boreholes were sealed in accordance with accepted practice for decommissioning of boreholes.

Field vane testing was completed in the boreholes throughout the cohesive soils to measure the in-situ undrained shear strength of the soils. The field vane used had dimensions of 150 mm long and 80 mm diameter. The field vane testing was conducted in accordance with ASTM D2573-08. Three 50 mm diameter "Shelby" tube samples

were also obtained in cohesive deposits to provide undisturbed samples for laboratory testing.

The fieldwork was co-ordinated and supervised by a member of Trow engineering staff. They located the boreholes, directed the drilling and sampling operation, logged borehole data in accordance with MTO Soils Classification System for foundation report, and retrieved soil samples for subsequent laboratory testing and identification. All of the recovered soil samples were placed in appropriate labeled moisture-proof containers and transported to Trow's Sudbury and Brampton laboratories for further detailed visual examination and laboratory testing.

Details of the soil strata encountered in the boreholes are included in attached borehole log sheets in Appendix C, and plotted on the cross sections in Appendix B. The borehole locations and the ground surface elevations along the cross sections were surveyed by Trow personnel, with reference to the benchmark at the east of the south dome as shown in the site survey map provided by MTO (PLAN H-564-17-1) (Elev. 261.131 m).

1.3.2 Laboratory Testing

On all of the samples returned to the laboratory, further visual examination and classification were carried out. The laboratory testing program included natural water content (LS-701), grain size distribution tests (LS703/704) and Atterberg limits (LS-703/704) on approximately 25% of the collected soil samples. Consolidation and strength testing (unconfined compression test) were carried out on selected specimens from the recovered undisturbed samples.

The laboratory test results are provided on the borehole logs in Appendix C. The results of the grain size analyses, Atterberg limits, consolidation and strength test are also included 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. The "Explanation of Terms Used in Report" is shown in the first page of the borehole logs sheets in Appendix C and should be read in conjunction with this report.

Appendix B shows the borehole location plan and three cross section soil profiles. It has to be underlined 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 field vane and Standard Penetration Tests results. 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 proposed structure site consists of top sand fill, underlain by upper silt deposits. The upper silt is underlain by silty clay followed by lower silt and underlain by weathered bedrock. A brief summary of the soil and groundwater conditions encountered in the boreholes is provided below.

1.4.1 Asphalt

At BH-3, BH-4 and BH-5, asphalt was encountered at ground surface. It has a thickness of about 25 mm and its top elevation is between about 261.0 and 261.6 m.

1.4.2 Sand and Gravel Fill

A layer of sand and gravel fill was encountered in all boreholes with a thickness ranging from 2.5 m to 3.1 m, corresponding to approximate bottom elevations of 258.5 and 257.9 m, respectively. At BH-3, BH-4 and BH-5, the sand and gravel fill was overlain by 25 mm thick asphalt whereas at BH-1 and BH-2 it was exposed at the ground surface.

The sand and gravel layer typically consisted of fine to coarse sand, fine to medium gravel, and trace to some silt. An approximately 0.1 m thick peat layer was encountered in BH-5 at elevation of 258.7 m. The sand and gravel layer is brown in color and frozen up to approximately 1.22 m depth. Below the frost line the fill is wet. Uncorrected SPT “N” values in the fill were in the range of 4 to 50 blows per 300 mm of penetration, corresponding to loose to dense compactness conditions, but more typically loose to compact conditions.

The results of the laboratory testing performed on selected samples of the sand and gravel fill layer are as follows:

- Moisture content:
 - 5.8% to 71.5% (peat)
- Grain size distribution:
 - 24 % to 28% gravel;
 - 40% to 42% sand; and
 - 32% to 34% silt and clay

The results of moisture content and grain size distribution tests are presented on the

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

1.4.3 Upper Silt

Below the fill, a layer of silt (named upper silt layer) was encountered in all boreholes with a thickness ranging from about 1.4 m to 4.1 m. It extends to depths between 4.0 m and 7.0 m, corresponding to approximate elevations of 257.0 and 254.5 m, respectively. At BH-1, BH-3, BH-4 and BH-5, the upper silt layer was underlain by a silty clay layer, while at BH-2 it was underlain by sand and gravel.

The upper silt layer consists of trace to some of clay and trace of sand. At BH-2, the upper silt contains trace of organics. The upper silt is grey in color and wet. The uncorrected SPT “N” values range between 6 and 15 blows per 300 mm of penetration, classifying the upper silt as loose to compact in compactness condition.

The results of the laboratory testing performed on selected samples of the upper silt layer are as follows:

- Moisture content:
 - 19.8% to 31.6%
- Grain size distribution:
 - 1% to 2% sand;
 - 86% to 91% silt; and
 - 8% to 12% clay

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

1.4.4 Silty Clay

Beneath the upper silt, a stratum of silty clay was encountered in BH-1, BH-3, BH-4 and BH-5. This silty clay layer has a thickness ranging from about 5.2 m to 11.7 m. It extends to depths between 10.7 (BH-3) m and 16.2 (BH-4 and BH-5) m, corresponding to approximate elevations of 250.9 and 244.8 m, respectively.

The silty clay layer was grey in color, wet. At BH-5, the layer of deposit was varved with clayey silt. The individual layers of laminations varied in thickness from a few millimeters to a few centimeters, but in general were about one centimeter thick.

The field vane undrained shear strength values ranged from 18 kPa to 49 kPa indicating a soft to firm consistency. The vane strength distribution with depth is shown on Figure 9, Appendix D. The undrained shear strength of a silty clay sample (from BH-5 at Elev. 250.3 m) measured in the unconfined compression tests was 19 kPa. Sensitivity ranged from 2.5 to 6, indicating the silty clay is low to medium sensitive.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution, Atterberg Limits, consolidation and unconfined compression tests. The test results are as follows:

- Moisture content:
 - 22.2% to 49.3%
- Grain size distribution:
 - 1% to 2% sand;
 - 37% to 71% silt; and
 - 28% to 61% clay
- Atterberg limits:
 - Plastic limit, PL = 14-20%;
 - Liquid limit, LL = 26-49%; and
 - Plasticity index, PI = 10-35%
- Consolidation properties:
 - Preconsolidation pressure, $P_v' = 150$ kPa
 - Compression index, $C_c = 0.22$
 - Recompression index, $C_r = 0.02$
- Unconfined compressive strength:
 - 38 kPa

The details of the moisture content and grain size distribution tests results are presented on the record of the borehole sheet in Appendix C. The results of the grain size distribution tests are also presented on Figure 3 in Appendix D. The consolidation and unconfined compression strength test results are shown on Figures 6 and 7, respectively, in Appendix D. The plasticity chart showing the Atterberg limits test results is included on Figure 8, Appendix D.

1.4.5 Sand and Gravel

In BH-2 a sand and gravel layer was encountered below the upper silt layer. It is about 2.1 m thick and extends from Elev. 254.5 m to 252.4 m.

This sand and gravel layer includes trace of silt. It is brown in color and wet. The uncorrected SPT “N” value is 22 blows per 300 mm penetration indicating a compact relative density. The natural water content performed on a selected sample of the sand and gravel layer was 8.6%. The result of moisture content test is presented on the record of the borehole sheet in Appendix C.

1.4.6 Lower Silt

A layer of silt (named lower silt layer) was encountered in BH-1, BH-3, BH-4 and BH-5 below the silty clay layer and in BH-2 below the sand and gravel layer. This lower silt layer has a thickness ranging from about 3.0 m to 8.2 m. It extends to depths between 14.1 m and 24.4 m, corresponding to approximate elevations of 247.4 and 236.6 m, respectively. BH-5 was terminated in this layer at a depth of approximately 19.8 m, elevation of 241.2 m. At BH-1, BH-2 and BH-3, the lower silt layer was underlain by weathered bedrock, while at location of BH-4 the layer was underlain by sand.

The lower silt layer consists of trace of gravel, sand and clay. It is grey in color and wet. Based on “N” values (6 to 19) obtained from the SPT, the compactness of the lower silt was loose to compact.

The results of the laboratory testing performed on selected samples of the lower silt layer are as follows:

- Moisture content:
 - 21.8% to 34.8%
- Grain size distribution:
 - 0% to 1% gravel;
 - 0% to 2% sand;
 - 90% to 97% silt; and
 - 2% and 10% clay

The details of the moisture content and grain size distribution tests results are presented on the record of the borehole sheet in Appendix C. The results of the grain size distribution tests are also presented on Figure 4 in Appendix D.

1.4.7 Sand

Sand was encountered in BH-4 below the lower silt and was underlain by highly weathered bedrock. This layer of sand is about 11.9 m thick and extends from Elev. 236.6 m to 224.7 m.

The deposit consists of some of silt and clay. It is grey in color and wet. The uncorrected SPT “N” values range between 9 and 22 blows per 300 mm, classifying the sand as very loose to very dense in compactness condition.

The results of the laboratory testing performed on selected samples of this sand layer are as follows:

- Moisture content:
 - 15.5% to 23.3%
- Grain size distribution:
 - 83% sand; and
 - 17% silt and clay

The details of the moisture content and grain size distribution test results are presented on the record of the borehole sheet in Appendix C. The results of the grain size distribution tests are also presented on Figure 5 in Appendix D.

1.4.8 Bedrock

Beneath the lower silt layer at BH-1, BH-2 and BH-3 and the sand layer at BH-4, weathered rock was encountered at depths ranging from 13.7 m to 36.3 m below existing ground surface (Elev. 247.9 m to 224.7 m). Bedrock coring was performed in BH-2 and BH-4. At BH-2, this weathered rock extends from about 14.1 m (approximately at elevation of 247.4 m) to 17.2 m (approximate elevation of 244.3 m) depth at the borehole termination. At BH-4, it extends from about 36.3 m (approximately at elevation of 224.7 m) to 39.7 m (approximate elevation of 221.4 m) depth at the borehole termination.

The total core recovery (TCR) was good, with values ranging from 88.3% to 100%. On the basis of the rock quality designation (RQD) index values which range between 50% to 100%, the rock quality is estimated to be “fair” to “excellent”, and the average of value of approximately 76% suggesting a rock of generally “good” quality.

Rock core photographs are presented in Appendix E.

1.5 Groundwater Conditions

The groundwater levels at the site were estimated during field borehole drilling and the change of the sample moist contents in depth. In addition, the groundwater level is measured in a piezometer installed in BH-3. The ground water levels encountered in the boreholes are also shown in Table 1.1. It should be noted that the groundwater level is subject to seasonal fluctuations.

Table 1.1 Groundwater levels at the site

Borehole No.	Date of drilling	Water level	
		Depth, (m)	Elevation, (m)
BH-1	February/02/2011	3.2	257.8
BH-2	February/02/2011	3.2	258.3
BH-3	February/11/2011	3.2	258.4
BH-4	February/07/2011	3.2	257.8
BH-5	February/09/2011	3.6	257.4

1.6 Closure

Field staff from Trow's Sudbury office supervised the field work. This report has been prepared by S. Micic, Ph.D., P.Eng and A. Geremew, Ph.D., and reviewed by S. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact.

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 foundation to support the proposed new building located at Walden Patrol Yard, Waters Township on Regional Road 55, Sudbury Area. The recommendations are based on interpretation of the factual boreholes data obtained during the field investigation. The proposed building will consist of a conventional building for storage of road sand, and will allow for inside loading and dumping. It is anticipated that the proposed building will be similar to that at the Cartier Patrol Yard, Cartier Township, in the Sudbury area. The building will have a footprint of about 24.4 x 42.7 m. Even though it is unlikely that deep foundations can be considered feasible for structures of this nature given their operating life and the high cost of pile foundations, two alternatives, spread footing on native soil and driven H-piles into bedrock, were described and evaluated in this report for completeness.

This report will address the geotechnical design of the foundation for the proposed building 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 latest edition of the *Canadian Highway Bridge Design Code (CHBDC)* (November 2006), the *Canadian Foundation Engineering Manual (CFEM)* (2006), and good practice.

Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the Terms of Reference from MTO letter dated December 03, 2010. It is assumed that the sand would be set on grade protected by an asphaltic concrete surface. Based on MTO experience, it is anticipated that stockpiling scenarios may include, but are not limited to, the followings:

- Salt stockpiled to the rear of the facility to the maximum allowable height of the “push wall” with the stock periodically replenished through out the winter (assume 1000 tonnes max at a given time), and
- Winter sand stacked to the maximum allowable height of the “push wall” at the rear of the facility occupying $\frac{3}{4}$ of building’s footprint with a ~500 tonne salt stock pile within the front $\frac{1}{4}$ of the building.
- In the future, there is a possibility that the storage facility will be loaded to its full allowable capacity. This scenario would consist of winter sand stacked to the

maximum allowable height of the “push wall” with a stockpile area covering the entire footprint of the building.

2.2 Geotechnical Design Considerations for Foundations

2.2.1 Shallow Foundations

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 stratigraphic sequence at the site typically consists of a 2.5 to 3.1 m thick layer of sand and gravel fill at the ground surface followed by a deposit of upper silt with thickness ranging from 1.4 m to 4.1 m. In the north-west and south-west parts of the site the upper silt is underlain by a layer of silty clay, approximately between 5.2 m and 11.7 m thick. In the north-east part, the upper silt is underlain with a 2.1 m thick layer of sand and gravel. Below the silt clay and sand and gravel layers a layer of silt is present. Overburden deposits are underlain by weathered bedrock at depths ranging from 13.4 m to 36.3 m below existing ground surface.

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. Lightly to moderately loaded structures and those structures where some total and differential settlements are permitted may be supported on shallow foundations bearing on the existing fill materials and following native silt materials. Shallow foundation should consist of strip footings which typically for this kind of structure have a width of 3 m (exp. the Cartier Patrol Yard project). The feasibility of shallow foundations depends on whether the structure can be accommodated in ground conditions with the axial resistance and settlement conditions described below.

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.

2.2.1.1 Geotechnical Resistance at Ultimate Limit States

Based on the results of the geotechnical investigation, the following recommendations for Ultimate Limit State design is presented:

- Ultimate Geotechnical Resistance at Ultimate Limit State of the foundation soil (competent fill and silt layers) is about 500 kPa assuming that the foundation width is 3 m.
- Factored Geotechnical Resistance is 250 kPa using a Geotechnical Resistance factor of 0.5 assuming that the foundation width is 3 m.

2.2.1.2 Geotechnical Resistance at Serviceability Limit State

Serviceability Limit States generally consider the unfactored 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. However, assuming a variety of configurations of a sand pile inside the building as suggested in Section 2.1, settlement is calculated using the Settle 3D software and the results are presented in Table 2.1. The consolidation parameters used on settlement analyses are based on the experimental data documented in Figure 6, Appendix D. According to the analyses, if 50 mm of total settlement and 25 mm differential are acceptable, then the geotechnical resistance at the Serviceability Limit States (SLS) is 90 kPa in the foundation area, assuming that the fill heights would not exceed the height of the “push wall”. Conditions would be expected to be better than calculated noting that two sand domes currently occupy significant areas of the proposed building area. Accordingly, assuming the above settlement criteria are accepted, a geotechnical reaction at SLS of 100 kPa is considered acceptable. If the acceptable settlement is reduced to 25 mm total, then the SLS value should be reduced to 75 kPa, again assuming that the fill heights would not exceed the height of the “push wall”.

2.2.1.3 Resistance to Lateral Loads

Section 6.7.5 of the CHBDC should be issued to compute resistance to lateral forces/sliding resistance between the subgrade and concrete. An unfactored value of 0.35 can be measured for the coefficient of friction $\tan \phi$ between the base of concrete footing and the in situ granular soils below frost level. When calculating lateral resistance of factor a 0.8 should be applied in accordance with the CHBDC.

2.2.1.4 Frost Protection

According to Ontario Provincial Standard Drawing (OPSD – 3090.101), the frost depth in Waters Township is about 2.1 m. Consequently, all footings exposed to

seasonal freezing conditions should be protected from frost action by at least 2.1 m of soil cover or equivalent insulation.

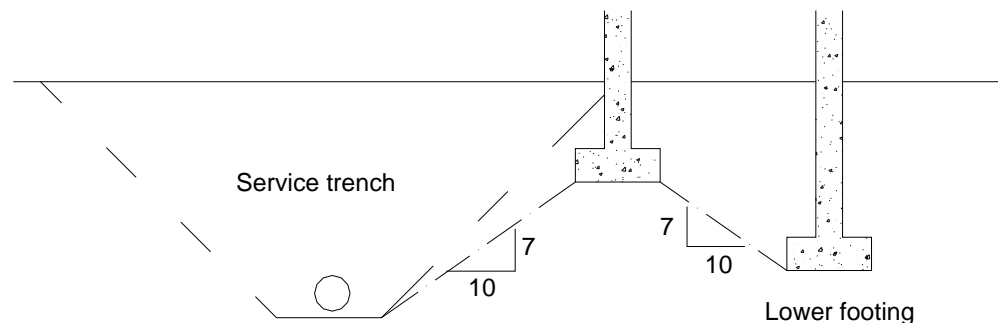
Table 2.1 Estimated settlement for different stockpiling scenarios

Pressure at Foundation Level (@ 2m bgs) (kPa)	Pressure in Middle Area (kPa)	Max. Settlement in Foundation Area (mm)	Max. Settlement in Middle Area (mm)
60	40	30	31
	50	34	38
	60	38	44
	70	42	52
	80	63	46
	90	51	76
	100	57	92
70	40	33	32
	50	36	38
	60	40	45
	70	44	52
	80	49	63
	90	54	77
	100	61	92
80	40	35	32
	50	39	39
	60	43	45
	70	47	53
	80	53	64
	90	58	78
	100	65	93
90	40	37	33
	50	41	39
	60	46	46
	70	51	53
	80	56	65
	90	63	79
	100	70	94
100	40	40	33
	50	44	40
	60	49	46
	70	54	54
	80	60	65
	90	67	79
	100	74	95
120	40	45	34
	50	51	41
	60	56	47
	70	63	55
	80	69	67
	90	76	81
	100	85	97

Loading/stockpiling patterns can be constrained to meet specific serviceability requirements as set by the structural designs and measure such as cross bracing can be used to enhance stability.

2.2.1.5 Foundation Elevation

The footings which are to be placed at different elevations 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 or existing service trench, 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.

2.2.1.6 Strip Footing Construction and Permanent Drainage

The wall of the proposed structure may be constructed as a cantilever retaining wall with an extended heel toward the inside of the structure and founded on native soils. Structural steel bars should be provided in the footings and in the walls. The asphalt floor can be designed inside the structure. The construction of spread footing and subgrade for the asphalt floor may be carried out in accordance with the following recommendations:

Prior to footing and asphalt floor construction, all obviously unsuitable material should be fully removed from the entire underfooting and underfloor area. Following rough grading, the exposed subgrade should be proofrolled with a roller under the full-time supervision of qualified geotechnical personnel. Any soft spots detected during proofrolling should be sub-excavated and replaced with approved materials compacted to 98 percent of the Standard Proctor Maximum Dry Density (SPMDD). The prepared subgrade should be covered with at least 200 mm compacted OPS Granular A, crowned slightly in the central area.

Around the perimeter of the building the ground surface should be sloped on a positive grade away from the structure to promote surface water run-off and reduce groundwater infiltration adjacent to the foundations. Perimeter drains are not required if the interior base is set at least 300 mm above the exterior grade and the grade is sloped away from the structure.

2.2.2 Deep Foundations

As mentioned, it is unlikely that deep foundations can be considered feasible for structures of this nature given their operating life and the high cost of pile foundations. However, for completeness, the deep foundation alternative is described and evaluated.

If the capacity and settlement for shallow foundations as described in the preceding sections are deemed inadequate for the proposed structure or other measures such as loading constraints not accepted, deep foundation can be considered. Although such foundations are typically not feasible from an economical standpoint for this type of application, information is provided here for completeness. Based on the borehole information obtained at this site, steel H-piles driven to bedrock can be used to support the structure. It is assumed that the sand would be stored on asphaltic concrete set on grade. The estimated tip elevations for the piles are based on the depth to bedrock encountered in the boreholes. The boreholes indicate bedrock sloping down from the north (BH-2 and BH-3) to the south (BH-4 and BH-5). The information indicated likely tip elevations near Elev. 247.5 m (BH-2 and BH-3) in the north, Elev. 244.3 m (BH-1) in the central section and Elev. 224.7 m (BH-4) in the south.

2.2.2.1 Geotechnical Axial Resistance

For HP310X110 piles, driven to practical refusal on the underlying bedrock, a factored axial resistance at ULS of 2,000 kN may be used. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the

factored axial resistance at ULS, since the bedrock is considered to be unyielding. Consequently, ULS conditions will govern for this foundation type.

2.2.2.2 Downdrag

Compressible deposits are present above bedrock and consequently downdrag loading may be induced as a result of the addition of sand after pile installation is complete and settlement of soil relative to the piles occurs. The structural design of pile should be based on an estimated unfactored downdrag load acting on the HP310X110 of 150 kN per pile. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section C6.8.4 of the Commentary to the CHBDC for ULS conditions.

2.2.2.3 Set Criteria

The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The criteria need to be set to also avoid overdriving and possibly damaging the piles. A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap a minimum of 10% piles to confirm the state after adjacent piles have been driven.

Piling should conform to the latest Special Provision, SP903S01. Driving shoes in accordance with OPSD 3000.10 should be incorporated to assist seating and minimize pile damage issues.

2.2.2.4 Resistance to Lateral Loads

Lateral loads can be resisted fully or partially by the use of battered piles. This is the preferred approach to lateral resistance.

If vertical piles only are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. The evaluation of the piles subjected to lateral loads should take into account such as factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects. The resistance to lateral loading in front of a vertical pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_n (kPa/m), is based on the equations given below:

For cohesionless soil:

$$k_n = \frac{n_h z}{B}$$

where: n_h = the constant of horizontal subgrade reaction (kPa/m), assume 1500 kPa/m for loose to very dense silt to sand and gravel
 z = the depth (m)
 B = the pile diameter or width (m)

For cohesive soils:

$$k_n = \frac{67 S_u}{B}$$

where: S_u = the undrained shear strength of the soil (kPa) assume 30 kPa for the silty clay
 B = the pile diameter or width (m)

In the calculation of lateral resistance of the pile, a depth of soil of $1.5 \times B$ in the upper zone should be ignored to account for disturbance (BROMS). Group action should be considered by applying a reduction factor (in accordance with DM- 7 NAVFAC).

2.2.3 Evaluation of Foundation Alternatives

As mentioned before, considering the high cost of pile foundations and the structure's operating life it is unlikely that deep foundations can be considered feasible for this patrol yard structure. However, an evaluation of these two foundation alternatives is included in this report. Advantages and disadvantages of spread footings and driven steel H-piles are presented in Table 2.2.

Given the subsurface conditions at the site the impact on settlements at the foundations of the structure will be influenced by the operating/stockpiling practices. It is our understanding that the structure will accommodate stockpiles of both road sand and salt at strategic locations within the structure. Based on the information mentioned in Section 2.1, the maximum loading condition is likely to be sand stockpiled to at least the level of the "push wall" over the full footprint. Mounding in the centre at the angle of repose is also a possibility.

These types of structures generally have service lives of about 20 years. Typically, in settings of poor soil conditions, the approach would be to mitigate potential distress for a shallow foundation supported on it rather than employ expensive deep foundations for building support. Mitigation can include stockpiling constraints and/or structure support using bracing or the like in order to enhance serviceability.

Table 2.2 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on Insitu Granular Material	1*	<ul style="list-style-type: none"> ▪ Straightforward construction 	<ul style="list-style-type: none"> ▪ Fairly low geotechnical resistance available ▪ Depending on conditions, some stockpiling constraints may be necessary 	<ul style="list-style-type: none"> ▪ Significantly lower relative cost compare to piles 	<ul style="list-style-type: none"> ▪ Risk of differential settlements due to loading patterns in the past and during operations ▪ Possible constraints on a storage volume
Steel H-Piles Driven to Bedrock	2	<ul style="list-style-type: none"> ▪ Straightforward construction 	<ul style="list-style-type: none"> ▪ Not typical for this type of structure ▪ Depth to founding levels on sloping rock ▪ Possible issues with sitting of piles on sloping rocks 	<ul style="list-style-type: none"> ▪ Higher relative costs compared with shallow foundations ▪ Unlikely to be economically feasible at the site 	<ul style="list-style-type: none"> ▪ Seating piles on sloping rock ▪ Not viable due to cost

* If geotechnical resistance is adequate, otherwise stockpiling constraints may be necessary. Stockpiling constraints can be considered to conform to specific requirements by the structural designer. Lateral bracing can also be used to mitigate conditions.

2.2.4 Liquefaction Considerations

The first 4.0 to 7.0 m below the ground surface the site mainly consists of granular materials (i.e., sand, gravel and upper silt) with SPT N-values ranging from 4 to 50. The water level in BH-3 after completion of drilling is at about 3.2 m depth. According to the observations of SPT's values, the subsoil could potentially be susceptible to liquefaction. Accordingly, liquefaction analyses have been performed using the Seed's approach, which is recommended by Canadian Foundation Engineering Manual (4th Edition 2006; Chapter 6, pg.101). This approach defines a factor of safety against liquefaction as the ratio of the induced cyclic stress ratio over the cyclic resistance ratio. The calculated factor of safety for the subsoil is generally more than 1.9. As a result, liquefaction is not likely to occur in the upper soils at the project site for the earthquake having 10% probability of exceedance in a 50-year period.

In addition, silty clay and lower silt can be classified as fine-grained soils. As shown in the grain size distribution analysis, they have a significant portion (over 98%) of fines passing through #200 sieve.

To delineate liquefaction susceptibility, this report adopted the empirical criteria recommended in Canadian Foundation Engineering Manual (Chapter 6, pg. 111):

- (1) $w/w_L \geq 0.85$ and $I_p \leq 12$: Susceptible to liquefaction or cyclic mobility
- (2) $w/w_L \geq 0.80$ and $10 \leq I_p \leq 12$: Moderately susceptible to liquefaction
- (3) $w/w_L < 0.85$ and $I_p \geq 12$: No liquefaction or cyclic mobility

Based on the above criteria, the liquefaction potential for the silty clay and lower silt is assessed to be "moderately susceptible".

2.2.5 Earthquake Considerations

Recommendations for the geotechnical aspects to determine the earthquake loading are presented below.

2.2.5.1 Subsoil Conditions

The subsoil and groundwater information at this site have been examined in relation

to Section 4.1.8.4 of the Ontario Building Code (OBC, 2006). The subsoil generally consists of sand and gravel fill followed by silt and silty clay layer underlain by bedrock. It is expected that the foundations will be founded in the sand and gravel fill underlain by silt and silty clay layers. The reported N-values for the soil below the founding level ranged from 3 to 50, with an average value less than 15. The undrained shear strength of the silty clay layer is between 18 and 49 kPa.

2.2.5.2 Corrected N-Values N_{60}

The Average Standard Penetration Resistance shown in Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2006 refers to N_{60} which is defined as “Average Standard Penetration Resistance for the top 30 m, corrected to a rod energy efficiency of 60% of the theoretical maximum”. It should be noted that the drillers in the Sudbury area do not have their rod energy efficiencies measured and therefore, computed N_{60} values are not available for this site.

In our opinion, the reported N-values could be considered as an approximate equivalent to the normalized N_{60} values as noted in the OBC 2006 for the purpose of establishing the site classification.

2.2.5.3 Depth of Boreholes

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2006 indicated that the average properties in the top 30 m are to be used to determine the site classification. The five (5) boreholes advanced for building construction at this site were approximately 14.3 to 39.7 m deep. The overburden soils mainly consist of sand, gravel, silt and silty clay having the average undrained shear strength of 30 kPa.

2.2.5.4 Site Classification

Based on the above assumptions and interpretations, and the soil conditions, the Site Class for this site is estimated to be “E” as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2006.

These parameters should be reviewed by the structural engineer.

2.3 Backfill

It should be possible to reuse most of the excavated native materials for backfilling. With some adjustments to their natural moisture contents, it should be feasible to re-compact them to a high density.

Backfills under areas to be paved, side walks, under buildings, and all areas where long term settlement is to be avoided, should be placed in 200 mm loose lifts and compacted to minimum 95% SPMDD. Under pavement, the upper 600mm of the subgrades should be compacted to 98% SPMDD.

2.4 Excavation and Groundwater Control

For the construction of the proposed building, excavations at least about 2 m depth will be required. The excavations are expected to encounter mostly sand and gravel.

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 2 soils.

No unusual construction conditions are expected for the excavations in the sand and gravel. The sand and gravel is hard and contains cobbles and boulders. 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 expected to be fairly shallow and well above groundwater levels measured during 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.

2.5 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 S. Micic, Ph.D., P.Eng and A. Geremew, Ph.D., and reviewed by S. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation 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.

A. Geremew, Ph.D
Geotechnical Specialist


Silvana Micic, Ph.D, P.Eng.
Geotechnical Engineer




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



APPENDIX A: PHOTOGRAPHS

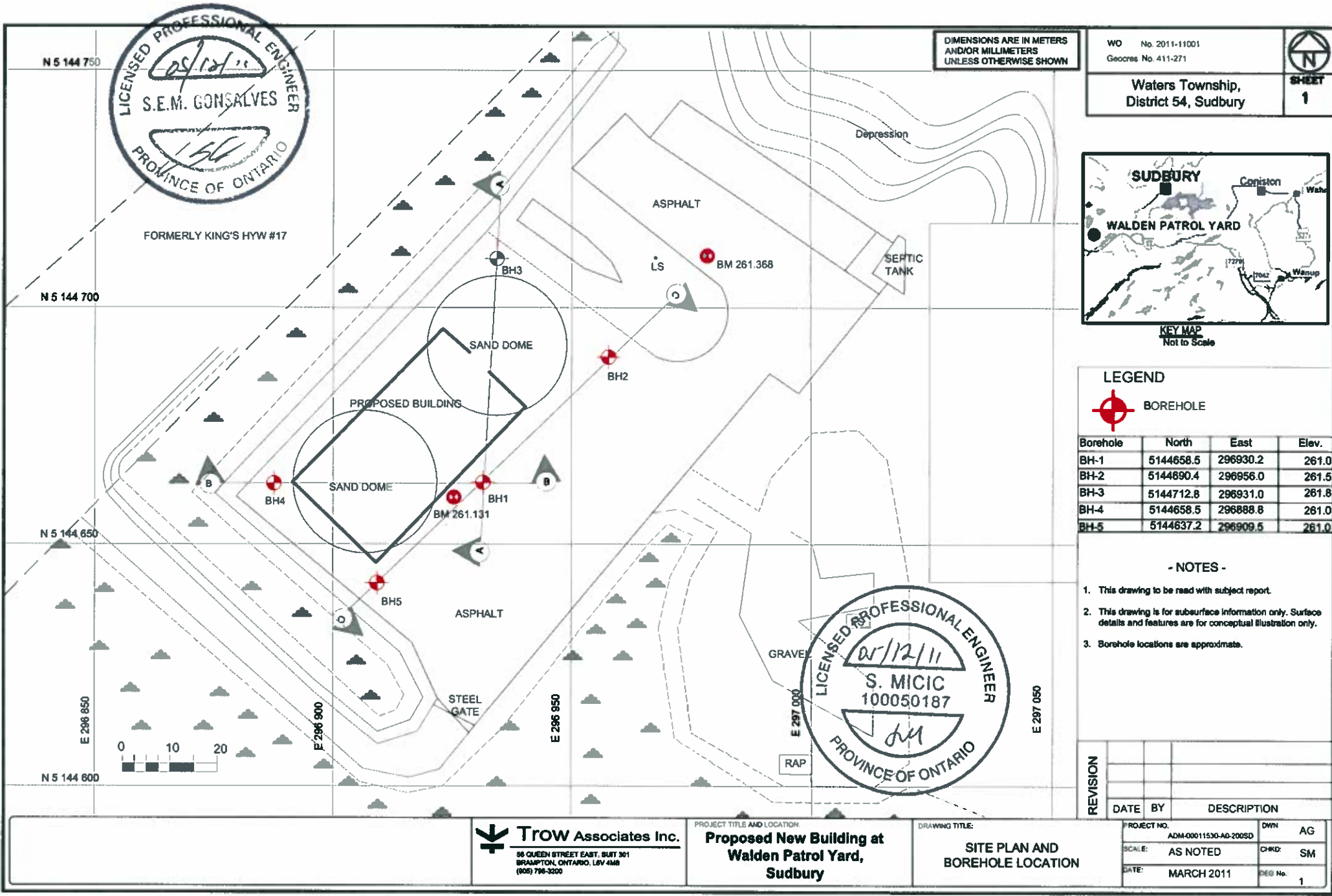


Photograph 1. Site View (facing to southwest)



Photograph 2. Site View (facing to northeast)

APPENDIX B: DRAWINGS

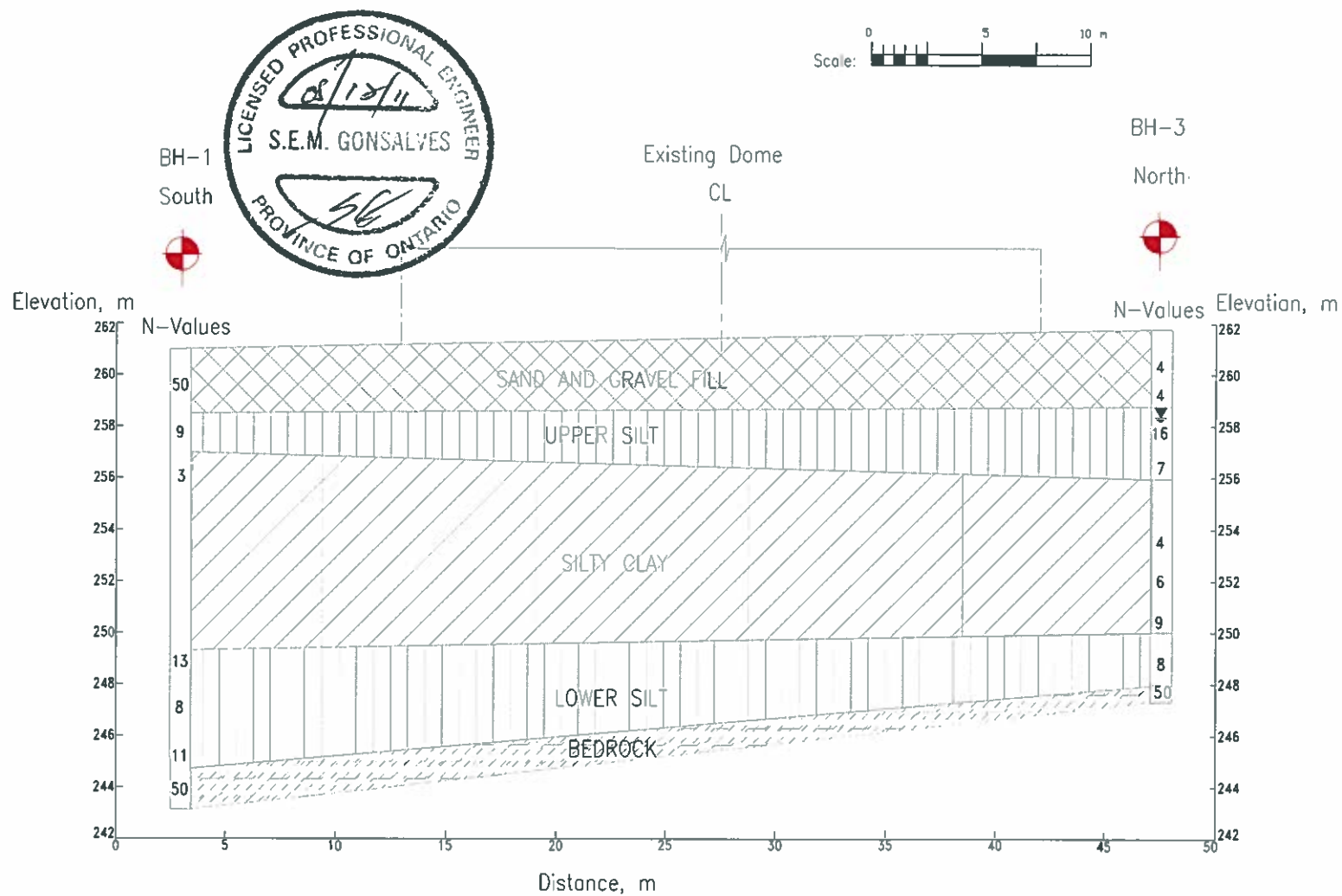


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(905) 796-3200

**Proposed New Building at
Walden Patrol Yard,
Sudbury**

**SITE PLAN AND
BOREHOLE LOCATION**

A-A Cross Section

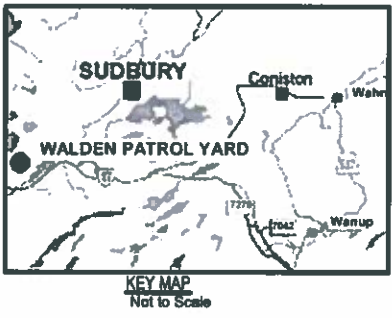


DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

WO No. 2011-11001
Geocres No. 411-271

Waters Township,
District 54, Sudbury

SHEET
2



LEGEND

BOREHOLE

Water Level

NOTES

- 1. This drawing to be read with subject report.
- 2. This drawing is for subsurface information only. Surface details and features are for conceptual illustration only.
- 3. Borehole locations are approximate.

REVISION	DATE		BY	DESCRIPTION	

PROJECT NO.	ADN-00011530-AD-2005D	DWN:	AG
SCALE:	AS NOTED	CHKD:	SM
DATE:	MARCH 2011	DEG No:	2

SOIL STRATA SYMBOLS:

SAND AND GRAVEL FILL

SILT

SILTY CLAY

BEDROCK

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(905) 706-3200

PROJECT TITLE AND LOCATION
Proposed New Building at
Walden Patrol Yard,
Sudbury

DRAWING TITLE:
A-A CROSS-SECTION

B-B Cross Section

DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

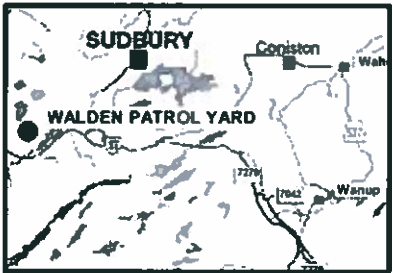
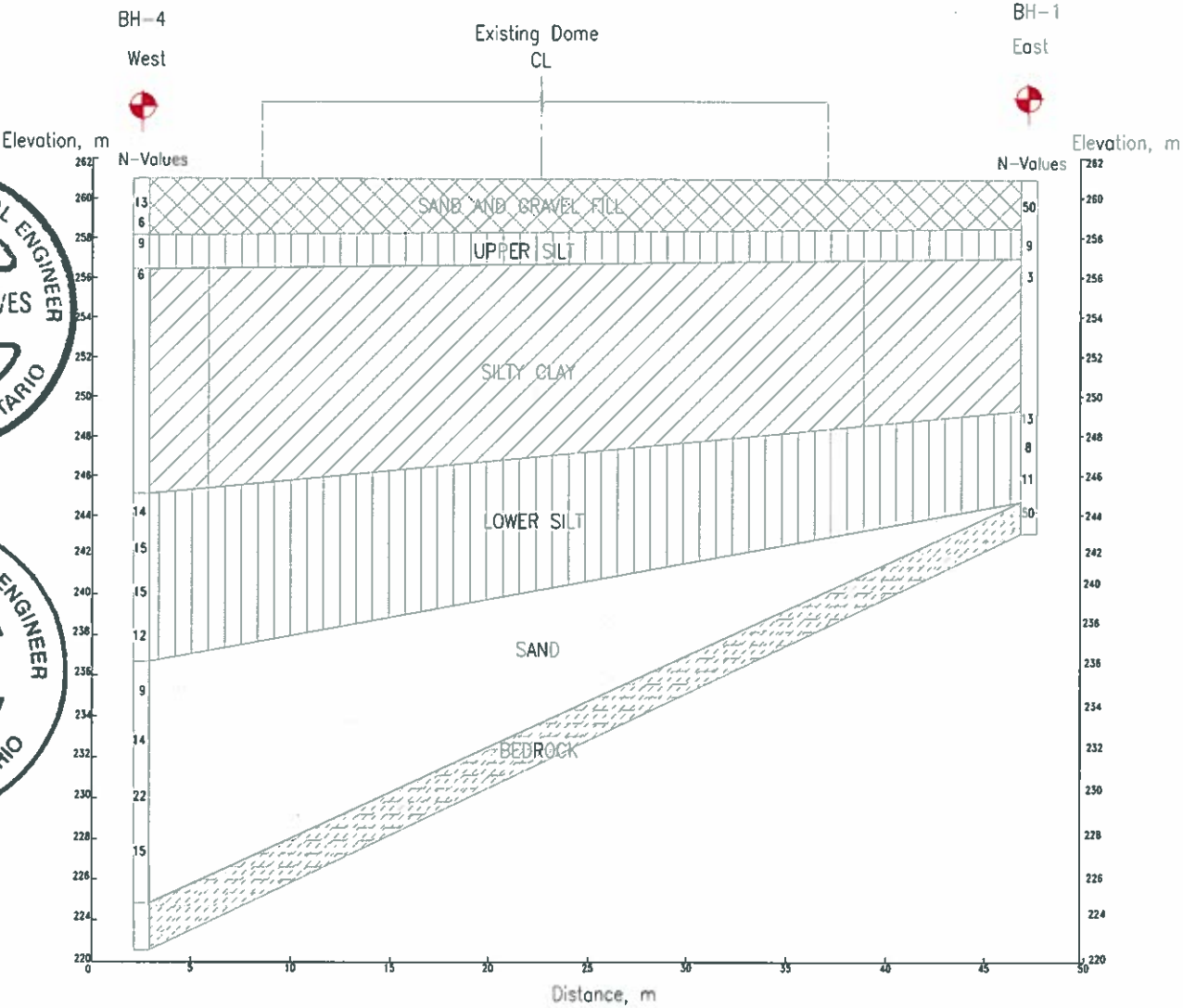
WO No. 2011-11001
Geocres No. 411-271



Waters Township,
District 54, Sudbury

SHEET
3

Scale: 0 5 10 m



KEY MAP
Not to Scale

LEGEND



- NOTES -

1. This drawing to be read with subject report.
2. This drawing is for subsurface information only. Surface details and features are for conceptual illustration only.
3. Borehole locations are approximate.

SOIL STRATA SYMBOLS:



SAND AND GRAVEL FILL



SILT



SILTY CLAY



SAND



BEDROCK



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PROJECT TITLE AND LOCATION:

**Proposed New Building at
Walden Patrol Yard,
Sudbury**

DRAWING TITLE:

B-B CROSS-SECTION

REVISION

DATE BY

DESCRIPTION

PROJECT NO.
ADM-00011530-A0-2005D

DWN: AG

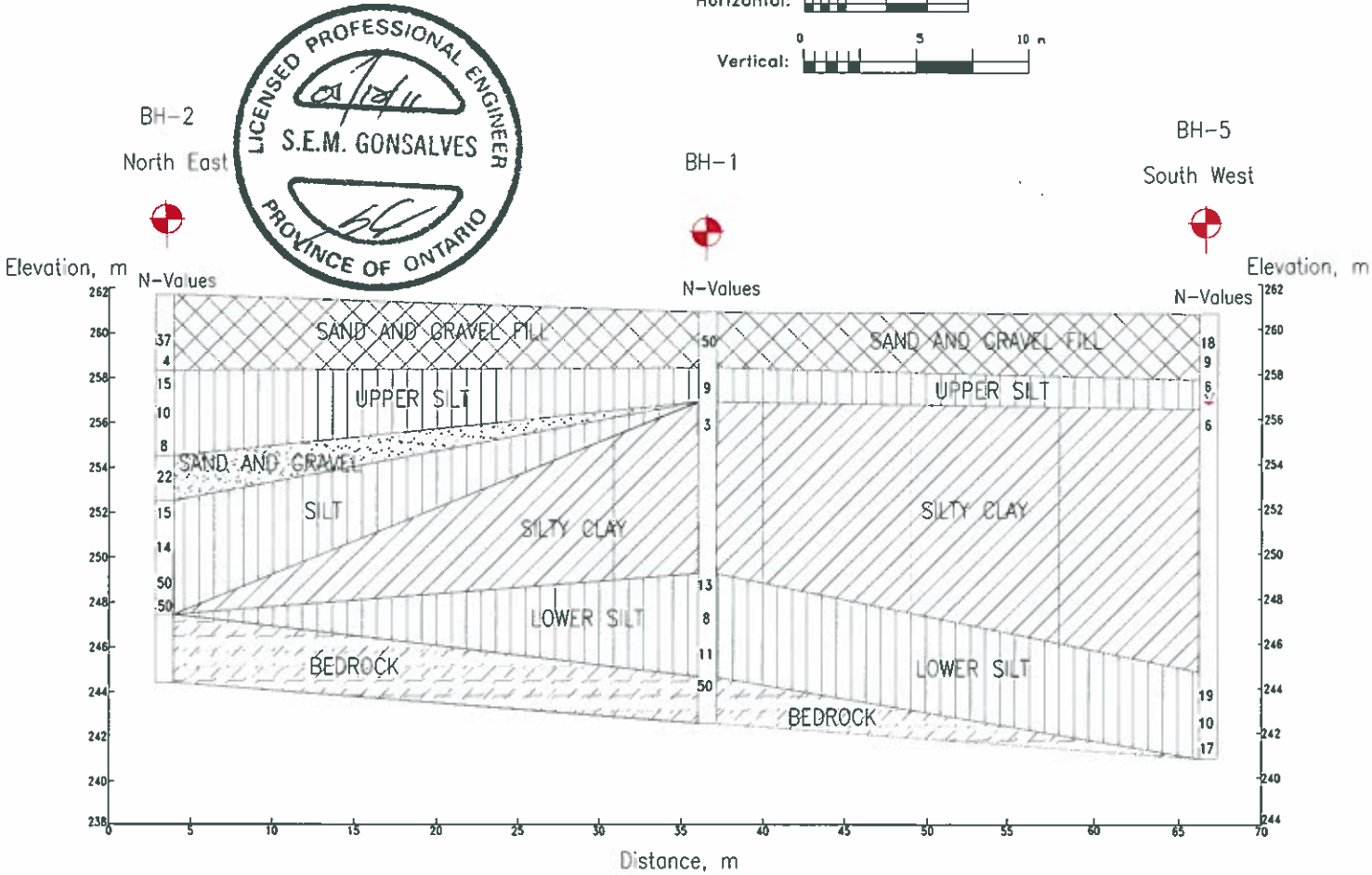
SCALE: AS NOTED

CHKD: SM

DATE: MARCH 2011

DEG No.: 3

C-C Cross Section



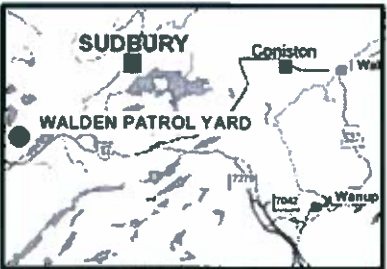
DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

WO No. 2011-11001
Geocres No. 411-271

Waters Township,
District 54, Sudbury



SHEET
4



KEY MAP
Not to Scale

LEGEND



- NOTES -

1. This drawing to be read with subject report.
2. This drawing is for subsurface information only. Surface details and features are for conceptual illustration only.
3. Borehole locations are approximate.

REVISION

DATE	BY	DESCRIPTION

SOIL STRATA SYMBOLS:



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PROJECT TITLE AND LOCATION:
**Proposed New Building at
Walden Patrol Yard,
Sudbury**

DRAWING TITLE:
C-C CROSS-SECTION

PROJECT NO. ADM-00011530-A0-2005D	DWN: AG
SCALE AS NOTED	CHKD: SM
DATE MARCH 2011	DEG No. 4

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. WO 2011-11001 LOCATION Walden Patrol Yard, North Sand Dome ORIGINATED BY CS
 DIST 54, Sudbury HWY 17 - RR 55 BOREHOLE TYPE CME Hollow Steam Auger/Diamond COMPILED BY AG
 DATUM Geodetic DATE 2011 02 02 - 2011 02 02 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
261.0	Ground Surface							20 40 60 80 100							
0.0	SAND and GRAVEL FILL (SW), trace to some silt, brown, damp (frozen to 1.2 m)		1	AS											
			2	SS	50										28 40 (32)
258.5	- hard augering from 1.8 to 2.3 m - wash bored/cored from 1.5 to 3.1 m														
2.5	SILT (ML), trace clay, trace sand, grey, wet, loose		3	SS	9										
257.0															
4.0	SILTY CLAY (CL), trace sand, grey, wet, soft		4	SS	3										
			5	SS	WH										0 1 63 36
			6	SS	WH										
			7	SS	WH										
			8	SS	WH										
249.3															
11.7	SILT (ML), trace clay, grey, wet, compact		9	SS	13										0 0 90 10
	- loose		10	SS	8										
			11	SS	11										
	- compact														
244.3															
16.8	WEATHERED BEDROCK				50										
243.1															
18.0	END OF BOREHOLE														
NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others. 3. "WH" means "Weight of Hammer"															

ONTARIO_MTO APPENDIX C - BOREHOLE LOGS - FINAL VERSION - MAY 8, 2011.GPJ_ONTARIO MOT.GDT_05/10/11

RECORD OF BOREHOLE No BH-2

1 OF 1

METRIC

W.P. WO 2011-11001 LOCATION Walden Patrol Yard, North Sand Dome ORIGINATED BY CS
DIST 54, Sudbury HWY 17 - RR 55 BOREHOLE TYPE CME Hollow Steam Auger/Diamond COMPILED BY AG
DATUM Geodetic DATE 2011 02 01 - 2011 02 02 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+	FIELD VANE								
								● QUICK TRIAXIAL	×	LAB VANE								
261.5	Ground Surface		1	AS			20	40	60	80	100							
0.0	SAND FILL (SW), some fine to coarse gravel, trace to some silt, brown, damp (frozen)																	
	- dense																	
259.4			2	SS	37													
2.1	GRAVEL FILL (GW), some sand, brown, wet, very loose		3	SS	4													
258.6																		
2.9	SILT (ML), trace clay, trace sand, trace organics, grey, damp, compact		4	SS	15													
	- wet		5	SS	10													
	- wet, loose		6	SS	8													
254.5																		
7.0	SAND and GRAVEL (SW), trace silt, brown, wet, compact		7	SS	22													
252.4																		
9.1	SILT (ML), trace sand, trace gravel, grey, wet, compact		8	SS	15													
	- cobbles retrieved between 12.2 and 13.7 m		10	SS	50													
247.4	- weathered igneous rock, grey		11	SS	50													
14.1	WEATHERED BEDROCK																	
	ROCK DRILLING STARTED AT ~14.1 m DEPTH				TCR													
	1st Run (from 14.1 to 14.9 m)			NQRC	100%													
	2nd Run (from 14.9 to 15.6 m)			NQRC	100%													
	3rd Run (from 15.6 to 17.2 m)			NQRC	100%													
244.3																		
17.2	END OF BOREHOLE																	
NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others.																		

ONTARIO_MTO_APPENDIX C - BOREHOLE LOGS - FINAL VERSION - MAY 8, 2011.GPJ_ONTARIO MOT.GDT_05/10/11

1 OF 1

METRIC

DATUM	Geodetic	DATE	2011 02 10 - 2011 02 11	CHECKED BY	SM
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ONTARIO MTO APPENDIX C - BOREHOLE LOGS - FINAL VERSION - MAY 8, 2011.GPJ ONTARIO MOT.GDT 05/10/11

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

RECORD OF BOREHOLE No BH-4

1 OF 2

METRIC

W.P. WO 2011-11001 LOCATION Walden Patrol Yard, North Sand Dome ORIGINATED BY CS
 DIST 54, Sudbury HWY 17 - RR 55 BOREHOLE TYPE CME Hollow Steam Auger/Diamond COMPILED BY AG
 DATUM Geodetic DATE 2011 02 07 - 2011 02 09 CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
261.0	Ground Surface		1	AS1										
260.0	ASPHALT		2	SS	13									
	SAND and GRAVEL FILL (SW), some clay, trace silt, brown, damp, loose to compact (frozen to 0.9 m)		3	SS	6									
258.1	- wet, loose		4	SS	9									24 42 (34)
258.1	SILT (ML), some clay, trace sand, grey, wet, loose													0 2 86 12
256.5	SILTY CLAY (CL), trace sand, grey, wet, firm		5	SS	6									
			6	TW										
			7	SS	WH									
			8	SS	WH									
			9	SS	WH									
			10	SS	WH									
			11	SS	WH									
	- wash bored from 15.2 m		12	SS	WH									
244.9	SILT (ML), trace sand, trace clay, grey, wet, compact		13	SS	14									
16.2			14	SS	15									
			15	SS	15									0 2 97 2

ONTARIO_MTO_APPENDIX C - BOREHOLE LOGS - FINAL VERSION - MAY 8, 2011.GPJ_ONTARIO MOT.GDT_05/10/11

Continued Next Page

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

RECORD OF BOREHOLE No BH-4

2 OF 2

METRIC

W.P. WO 2011-11001 LOCATION Walden Patrol Yard, North Sand Dome ORIGINATED BY CS
DIST 54, Sudbury HWY 17 - RR 55 BOREHOLE TYPE CME Hollow Stem Auger/Diamond COMPILED BY AG
DATUM Geodetic DATE 2011 02 07 - 2011 02 09 CHECKED BY SM

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 (%)
								20	40	60				
	SILT (ML), trace sand, trace clay, grey, wet, compact (<i>continued</i>)		16	SS	12		237							
236.6							236							
24.4	SAND (SM), fine grained, some silt, grey, wet, loose						235						0 83 (17)	
			17	SS	9		234							
							233							
	- compact						232							
			18	SS	14		231							
							230							
	- fine to medium grained, trace silt, grey, wet, compact						229							
			19	SS	22		228							
							227							
							226							
			20	SS	15		225							
224.7							224							
36.3	WEATHERED BEDROCK						223						RQD = 50%	
	ROCK DRILLING STARTED AT ~36.3 m DEPTH				TCR									
	1st Run (from 36.3 to 36.8 m)			NQRC	100%								RQD = 100%	
	2nd Run (from 36.8 to 38.3 m)			NQRC	88.3%									
	3rd Run (from 38.3 to 39.7 m)			NQRC	100%		222						RQD = 100%	
221.4														
39.7	END OF BOREHOLE													
	NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others. 3. "WH" means "Weight of Hammer"													

ONTARIO_MTO_APPENDIX C - BOREHOLE LOGS - FINAL VERSION - MAY 8, 2011.GPJ_ONTARIO.MOT.GDT_05/10/11

RECORD OF BOREHOLE No BH-5

1 OF 1

METRIC

W.P. WO 2011-11001 LOCATION Walden Patrol Yard, North Sand Dome ORIGINATED BY CS
DIST 54, Sudbury HWY 17 - RR 55 BOREHOLE TYPE CME Hollow Steam Auger/Diamond COMPILED BY AG
DATUM Geodetic DATE 2011 02 09 - 2011 02 09 CHECKED BY SM

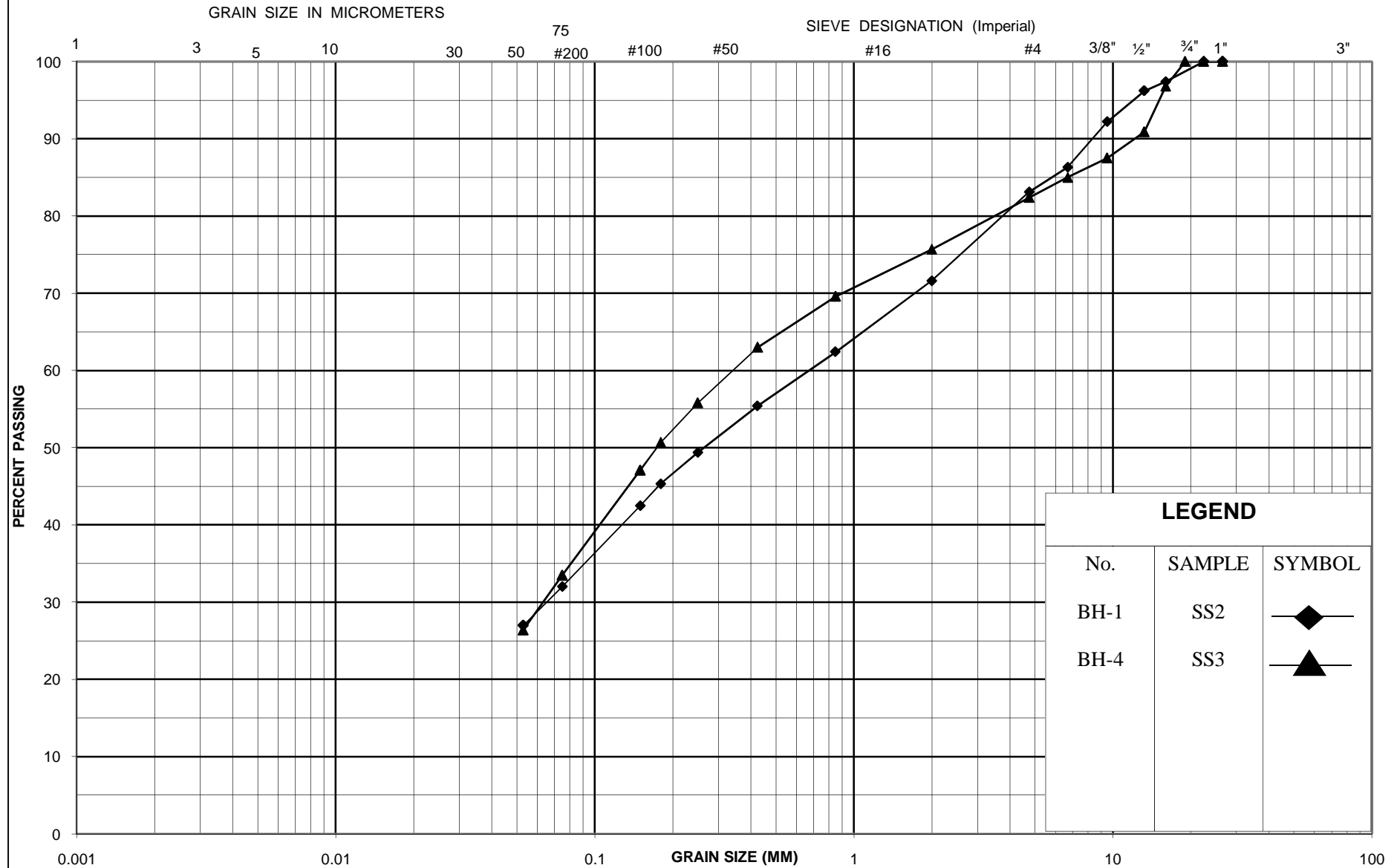
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
261.0	Ground Surface							20 40 60 80 100						
260.0	ASPHALT		1	AS				20 40 60 80 100						
	SAND FILL (SW), some fine gravel, trace silt, brown, damp, loose to compact (frozen to 1.2 m)		2	SS	18									
	- wet at 1.8 m		3	SS	9									
	- 0.1 m peat, black, wet													
257.9														
3.1	SILT (ML), trace clay, trace sand, grey, wet, loose		4	SS	6									
256.5														
4.5	SILTY CLAY (CL), trace sand, grey, wet, soft		5	SS	6									
	- becomes varved, firm													
			6	SS	WH									
			7	SS	WH									
			8	SS	WH									
			9	TW										
			10	SS	WH									
			11	SS	WH									
			12	TW										
244.8														
16.2	SILT (ML), trace clay, grey, wet, compact		13	SS	19									
			14	SS	10									
241.2			15	SS	17									
19.8	END OF BOREHOLE													
	NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others. 3. "WH" means "Weight of Hammer"													

ONTARIO_MTO_APPENDIX C - BOREHOLE LOGS - FINAL VERSION - MAY 8, 2011.GPJ_ONTARIO MOT.GDT_05/10/11

APPENDIX D: LABORATORY DATA

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL (SW)

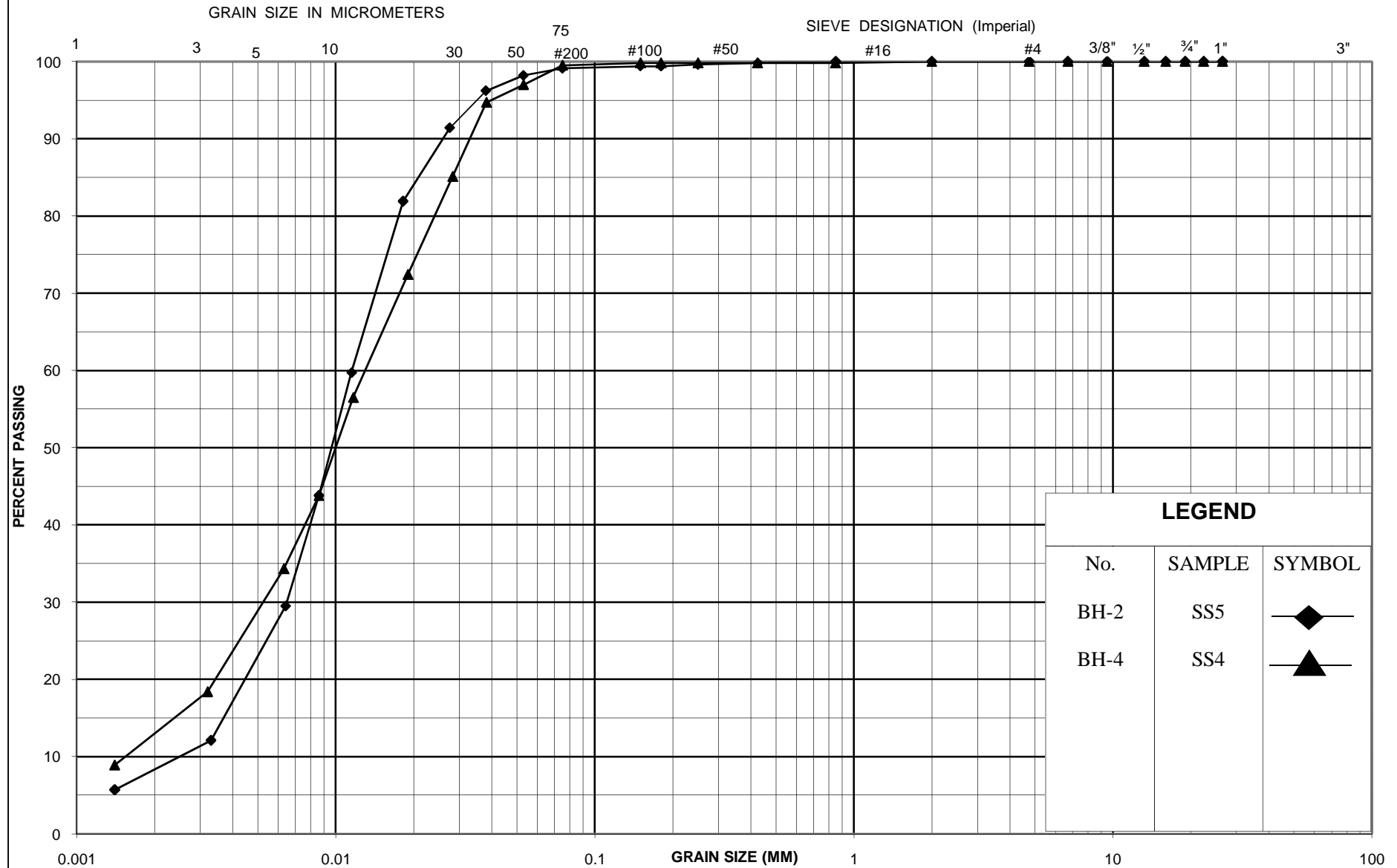
FIGURE No. 1

WO:

DATE March/30 /2011

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



LEGEND

No.	SAMPLE	SYMBOL
BH-2	SS5	◆
BH-4	SS4	▲

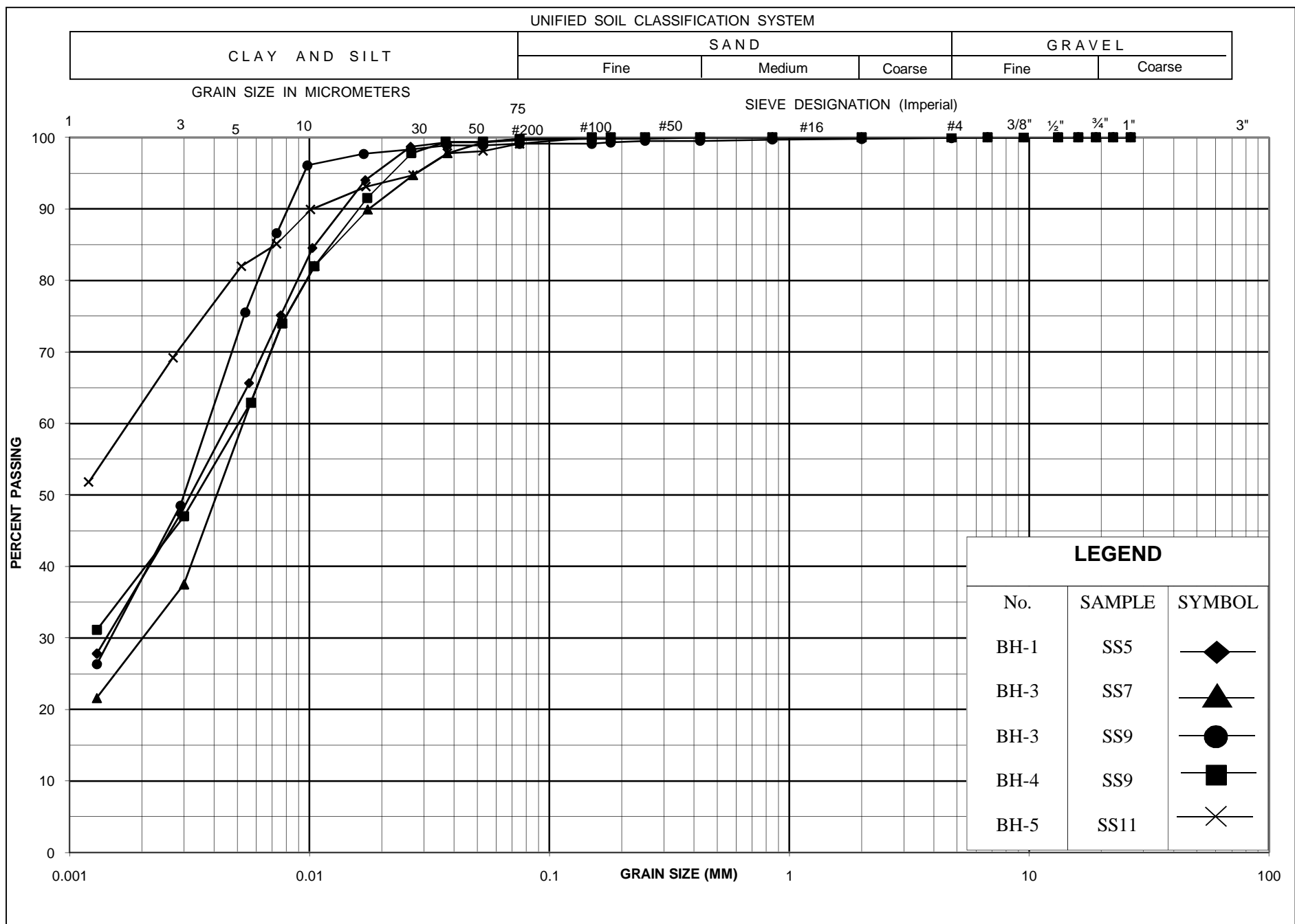


GRAIN SIZE DISTRIBUTION
UPPER SILT (ML)

FIGURE No. 2

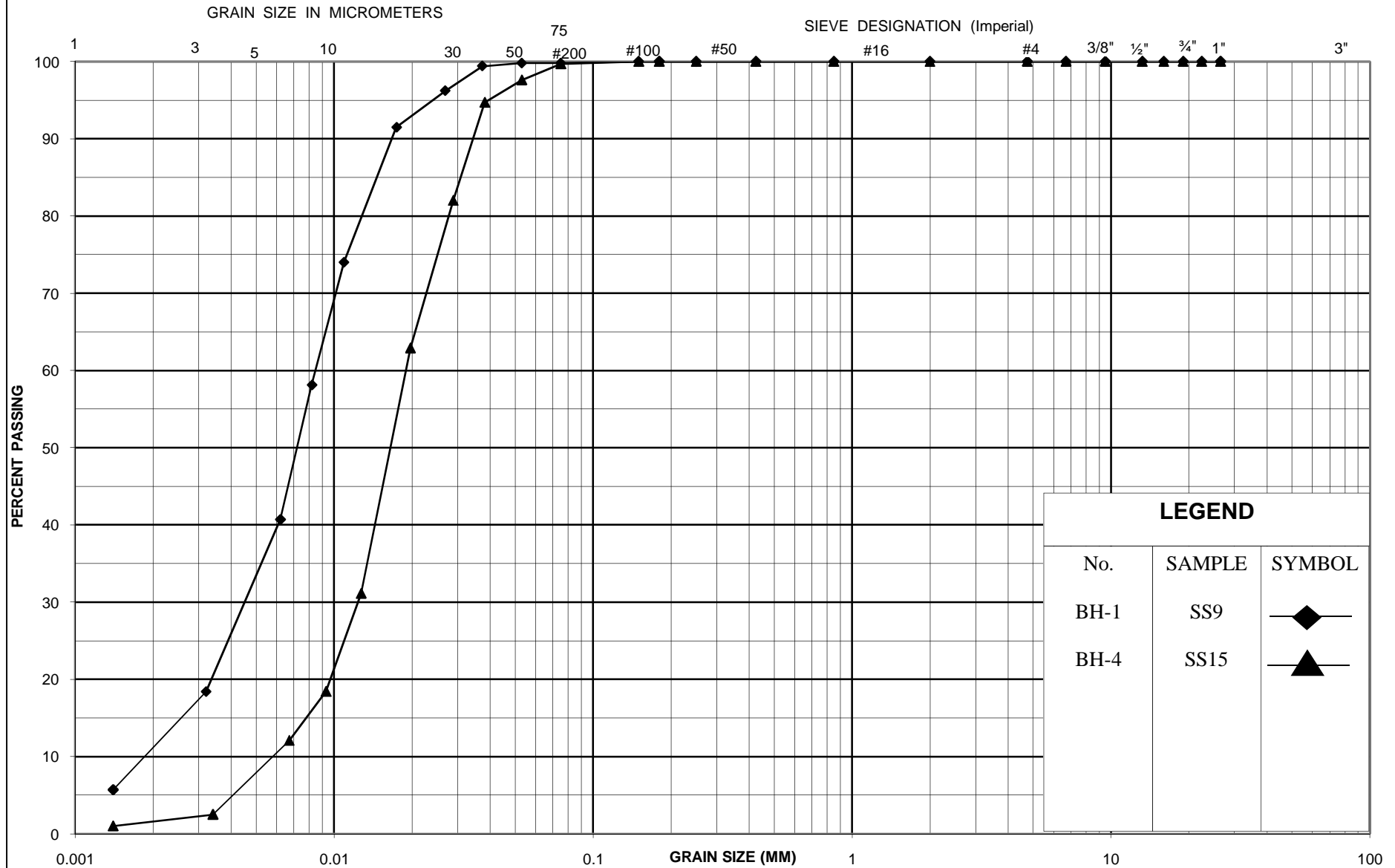
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DATE March/ 30/2011



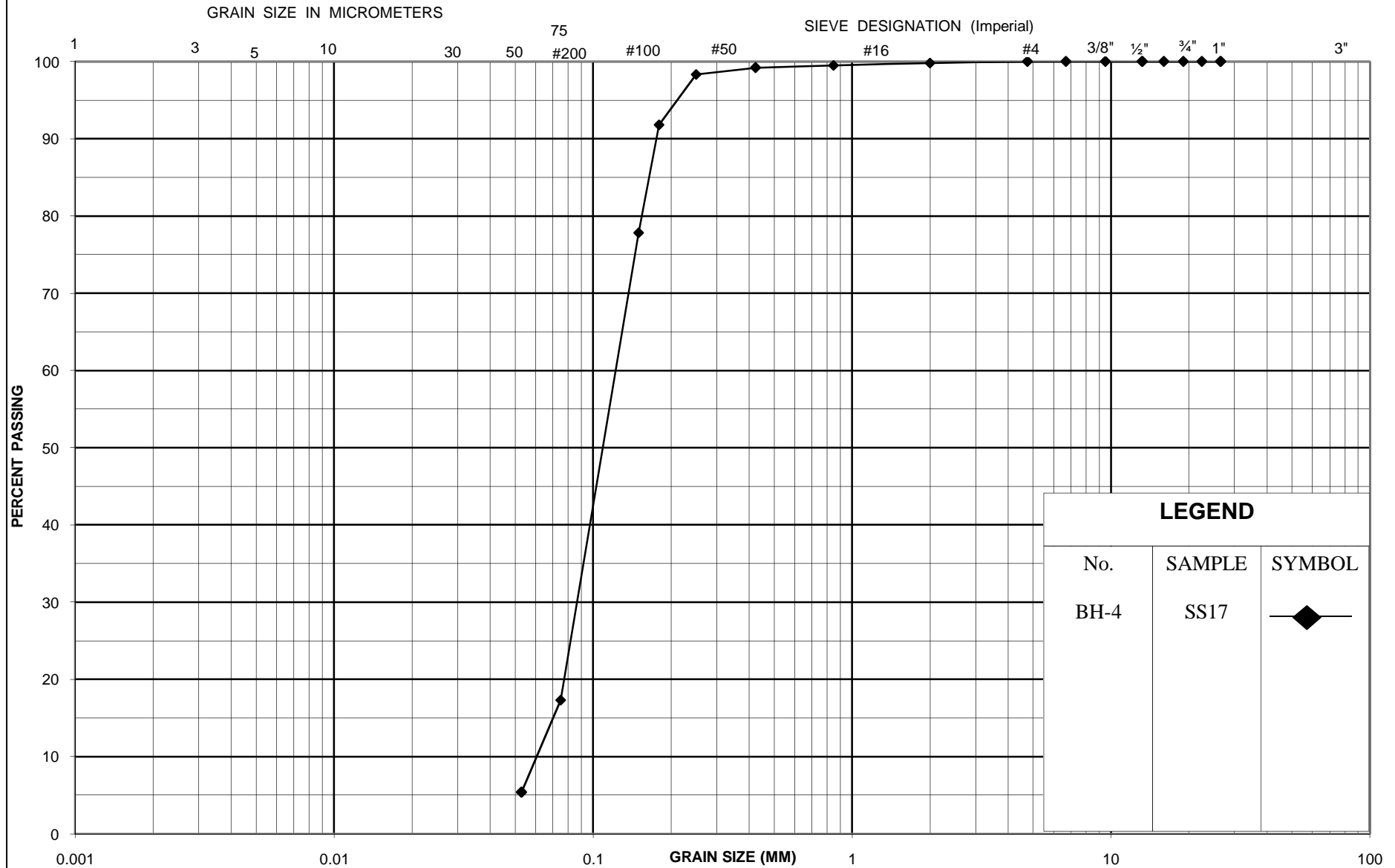
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION
SAND (SM)

FIGURE No. 5

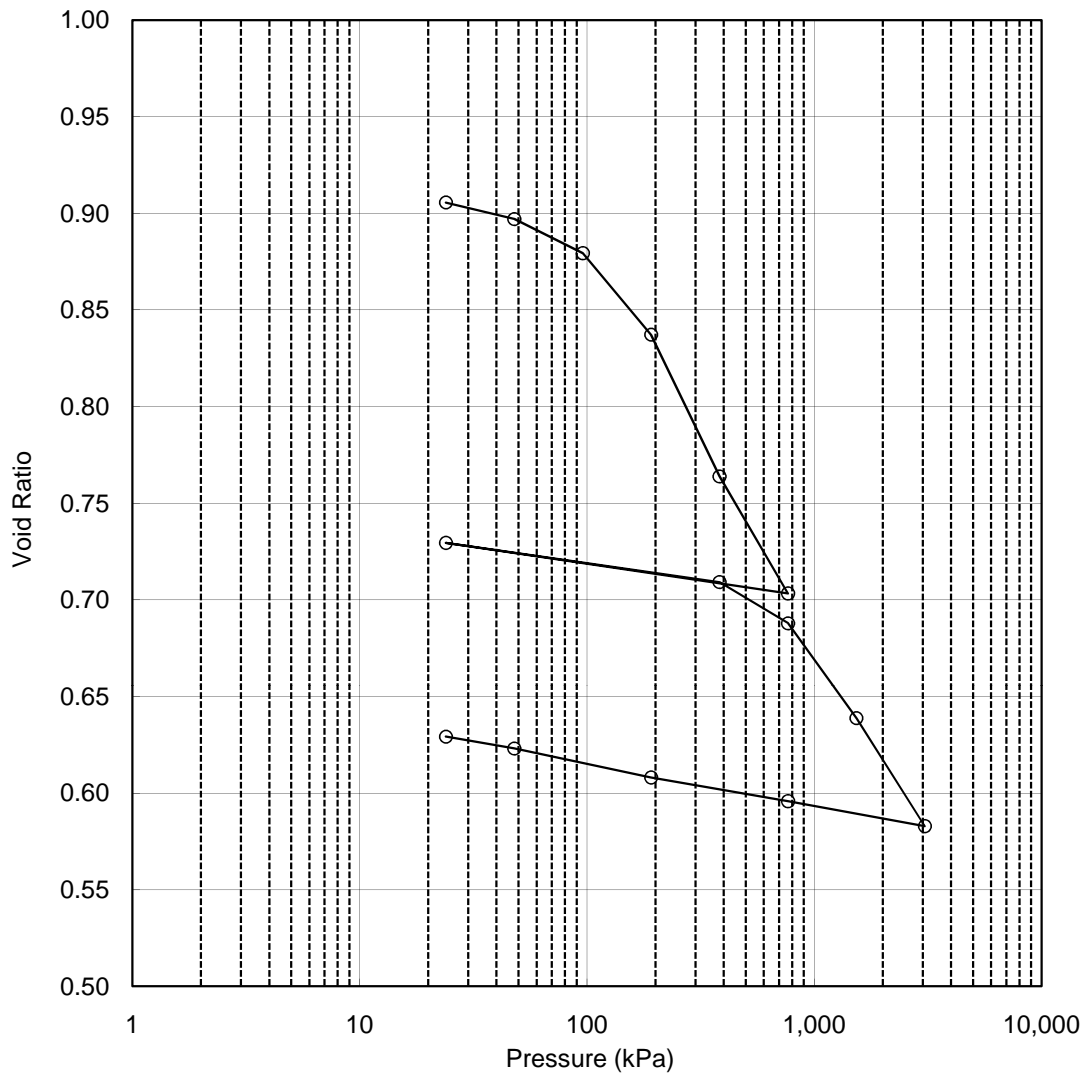
WO:

DATE March/ 30/ 2011

BH-5 TW9, 10.9m depth

FIGURE 6

CONSOLIDATION TEST
e vs Pressure



$e_o = 0.92$

$\omega = 37\%$

Ground Elev.= 261.0 m

GWL Elev. = 257.4 m

GWL Depth= 3.6 m

Sample Depth= 10.9 m

$P_v' = 118 \text{ kPa}$

$P_c' = 190 \text{ kPa}$

$C_c = 0.22$

$C_r = 0.024$

$OCR = 1.6$

Project No. : ADM-00011530-A0-200SD

Date : March/30 /2011



Prepared By : TS

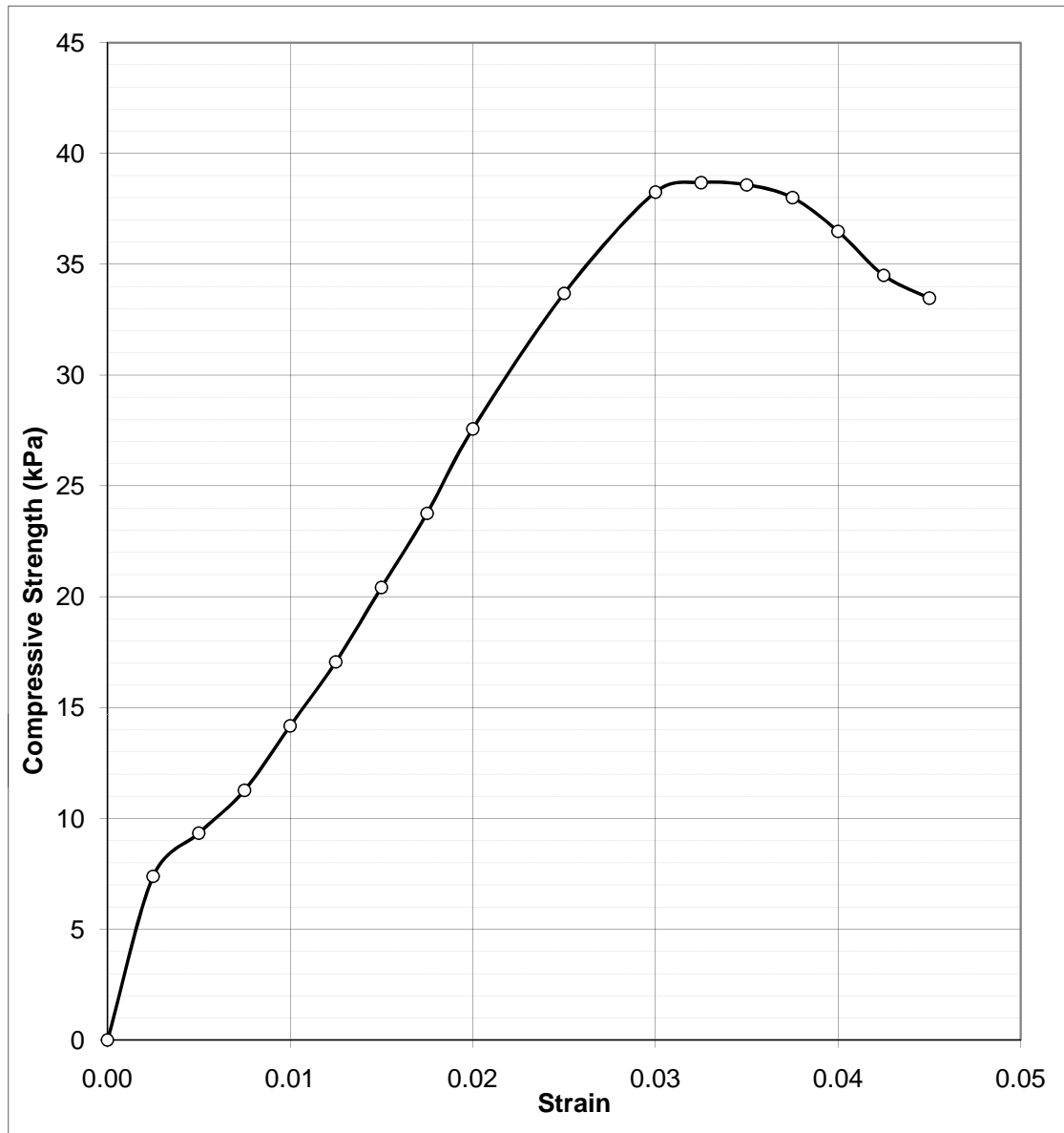
Checked By : SM

C:\Documents and Settings\T S Ahn\Desktop\EXP\Vermer and Walden Patrol Yard - A Geremew\Walden Patrol Yard\Excel files\Walden Patrol Yard-BH5 10.9 m Consolidation_Trow.xls

BH-5 TW9, 10.7-11.1m depth

FIGURE 7

Unconfined Compressive Test
Compressive Strength vs. Strain



$\omega = 37\%$

Ground Elev.= 261.0 m

GWL Elev. = 257.4 m

GWL Depth= 3.6 m

Sample Depth= 10.7-11.1 m

Project No. : ADM-00011530-A0-200SD

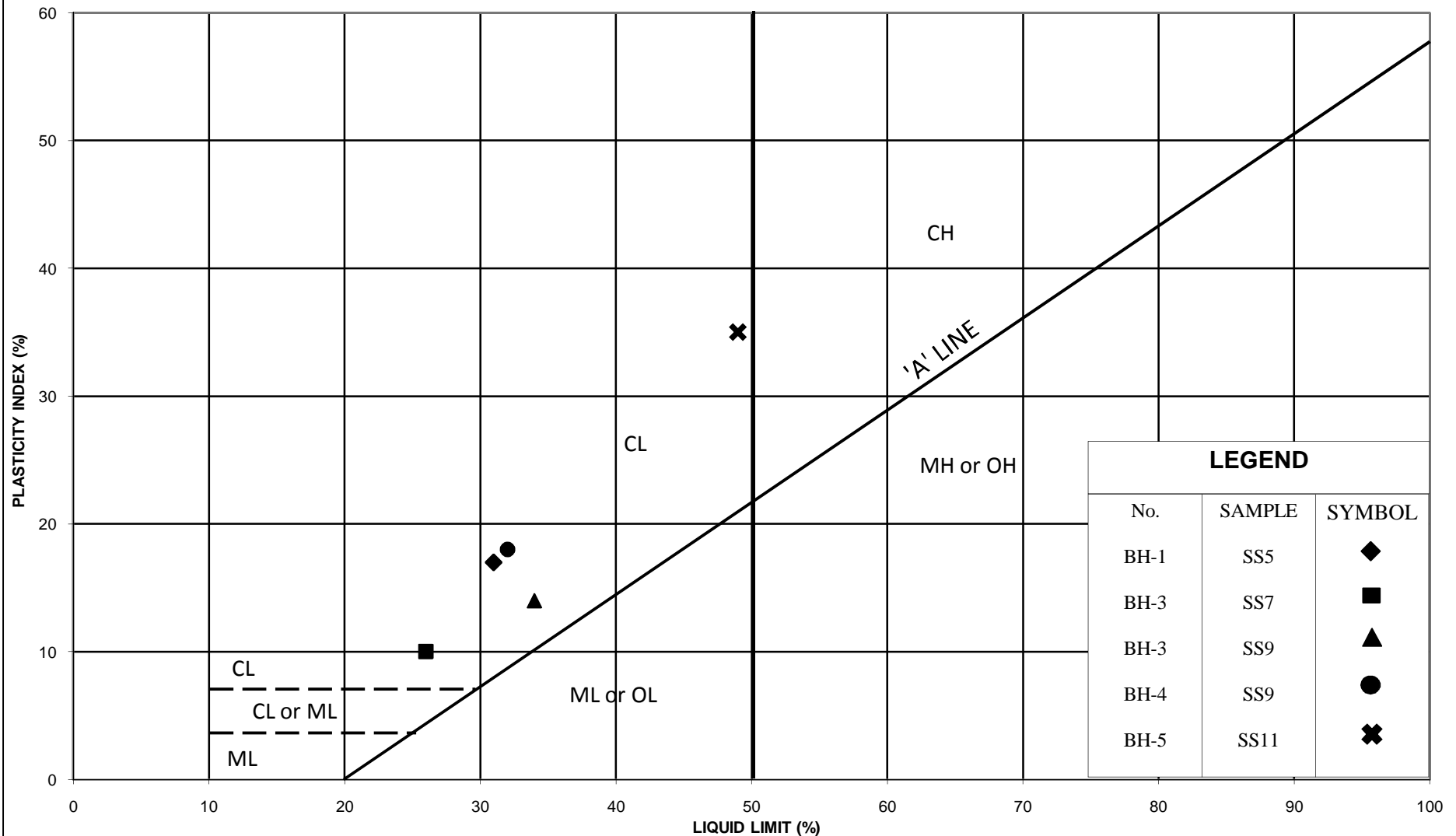
Date : March/ 30 /2011



Prepared By : TS

Checked By : SM

WALDEN PATROL YARD

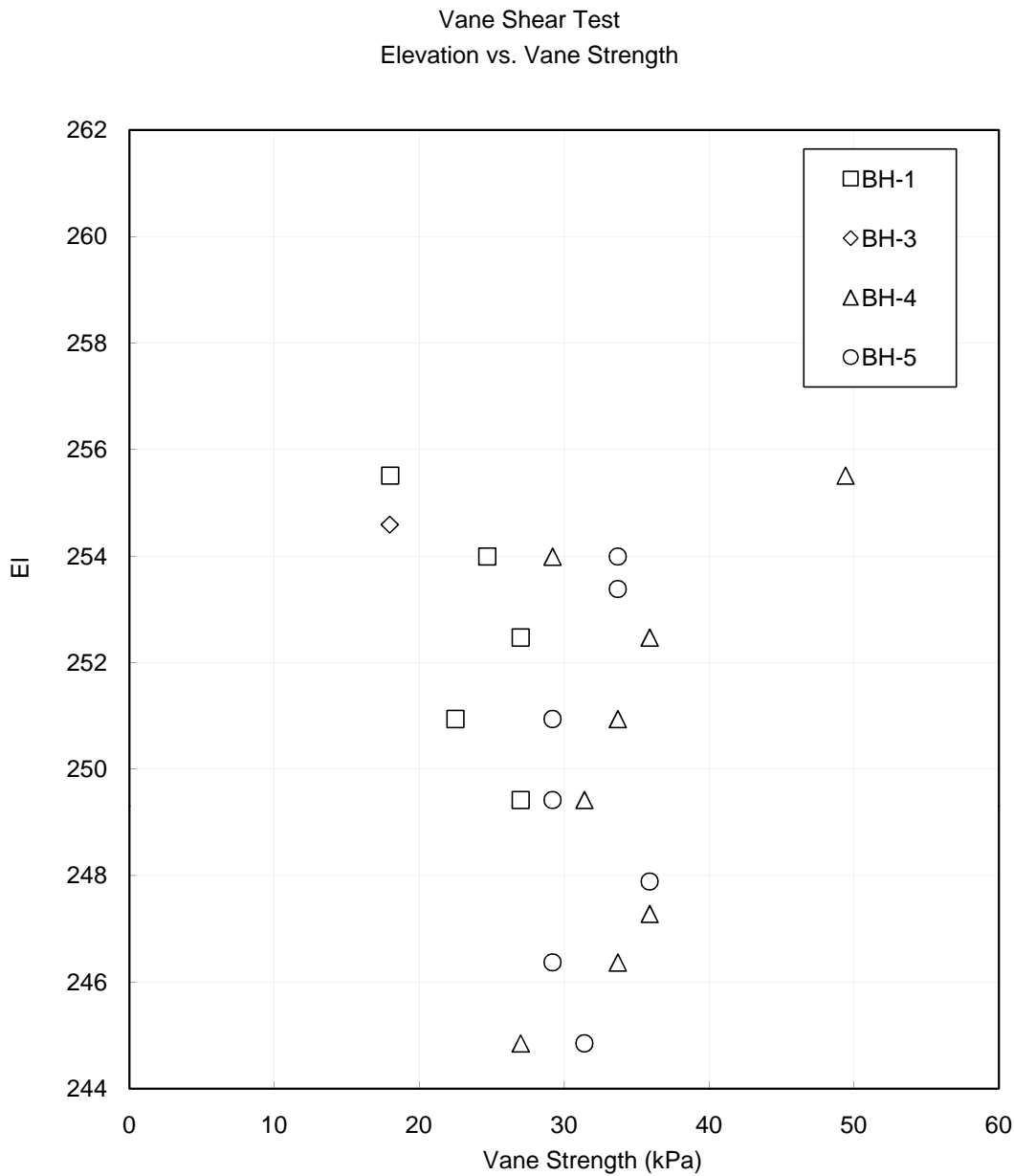


PLASTICITY CHART: UNIFIED SYSTEM
SILTY CLAY (CL)

FIGURE No. 8

WO:

DATE March/ 30/2011



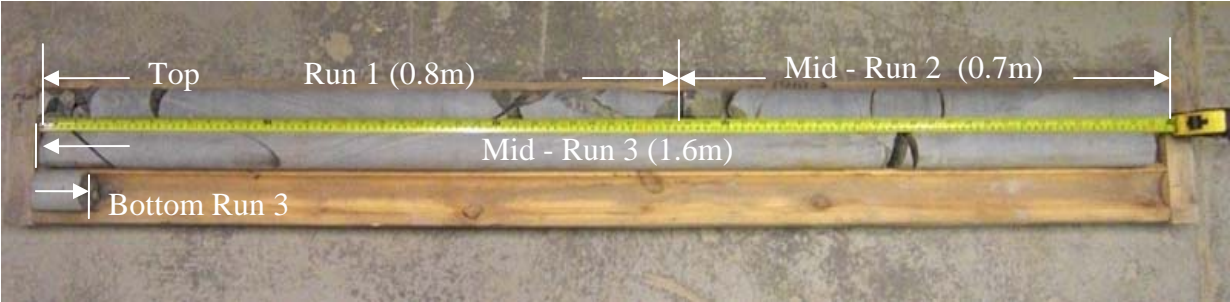
Ground Elev.= 261 - 261.5 m

GWL Elev. = 257.8 (BH 3) and 257.4 (BH 5) m

APPENDIX E: ROCK CORE PHOTOGRAPHS

PHOTOGRAPH 1 BH-2

(a) Overall



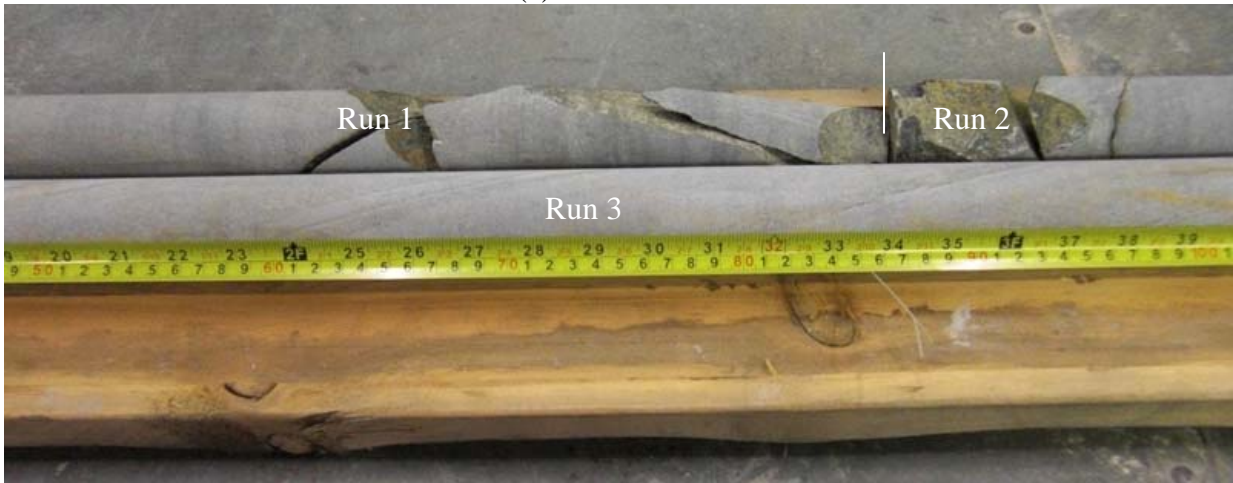
(b) Top



PHOTOGRAPH 1

BH-2

(c) Middle -1



(d) Middle -2



PHOTOGRAPH 1

BH-2

(e) Middle -3



(f) Bottom



PHOTOGRAPH 2 BH-4

(a) Overall



(b) Top



PHOTOGRAPH 2

BH-4

(c) Middle - 1



(d) Middle - 2



PHOTOGRAPH 2

BH-4

(e) Bottom

