

**MTO Agreement No. 5011-E-0010  
WO No. 2011-11031  
Proposed Sand/Salt Storage Facility  
Englehart Patrol Yard  
Foundation Investigation and Design  
Report  
Geocres No. 31M-95  
August 2012**

Prepared for:  
Ontario Ministry of Transportation  
Northeastern Region  
447 McKeown Avenue  
North Bay, Ontario  
CANADA P1B 9S9

Prepared by:  
GENIVAR Inc.  
294 Rink Street, Suite 103  
Peterborough, Ontario K9J 2K2

Project No. 121-17876-00



Project No. 121-17876-00

August 13, 2012

Mr. Jean-Pierre Perron, P. Eng.  
MTO Project Manager  
Ontario Ministry of Transportation  
Northeastern Region  
447 McKeown Avenue  
North Bay, Ontario P1B 9S9

**Re: MTO Agreement No. 5011-E-0010/WO No.: 2011-11031 Proposed Sand/Salt  
Storage Facility - Englehart Patrol Yard  
Foundation Investigation and Design Report (Geocres No. 31M-95)**

Dear Mr. Perron:

We are pleased to submit our Foundation Investigation and Design Report for the proposed Sand/Salt Storage Facility at the Ontario Ministry of Transportation Northeastern Region (MTO) Englehart Patrol Yard in Englehart, Ontario. A borehole and laboratory testing program was conducted to assess soil and groundwater conditions at the site and provide recommendations for foundation design for the proposed structure.

This report presents the investigation methodology and findings, and was completed in accordance with the Terms of Reference provided in MTO Agreement #5011-E-0010.

We trust that this report meets your current requirements. Please contact us if you have any questions.

Yours truly,  
**GENIVAR Inc.**

A handwritten signature in blue ink, appearing to read "J. Stephen Ash", with a stylized flourish at the end.

J. Stephen Ash, P. Eng., P. Geo.  
Consulting Engineer/Business Unit Leader

JSA:nah

# Table of Contents

Transmittal Letter  
Table of Contents

1. INTRODUCTION.....	1-1
2. SITE DESCRIPTION AND REGIONAL GEOLOGY .....	2-1
2.1 Site Description .....	2-1
2.2 Regional Geology .....	2-1
3. HISTORIC REPORT REVIEW .....	3-1
4. INVESTIGATION PROCEDURES .....	4-1
4.1 Subsurface Investigation .....	4-1
4.2 Laboratory Testing .....	4-3
5. SUBSURFACE CONDITIONS .....	5-1
5.1 Soil Profile Summary .....	5-1
5.1.1 Asphalt Pavement .....	5-1
5.1.2 Granular Fill .....	5-1
5.1.3 Silt .....	5-2
5.1.4 Clayey Silt .....	5-2
5.1.5 Silt and Clay .....	5-3
5.1.6 Dynamic Cone Penetration Testing .....	5-4
5.2 Groundwater Conditions .....	5-4
6. GEOTECHNICAL DESIGN CONSIDERATIONS .....	6-1
6.1 "Red Flag" Conditions .....	6-1
6.2 Mitigation of Settlement Potential .....	6-1
6.3 Structure Foundation Design Options.....	6-3
6.4 Frost Penetration Depth.....	6-4
6.5 Preferred Foundation Option .....	6-4
6.6 Other Foundation Options .....	6-5
6.6.1 Slab-on-Grade.....	6-5
6.6.2 Deep Foundations .....	6-6
6.6.2.1 Drilled and Cast-in-Place Concrete Foundation (Caisson) .....	6-6
6.6.2.2 Driven Piles.....	6-6
6.7 Resistance to Lateral Loads .....	6-7
6.8 Earth Pressure Design.....	6-7
6.9 Seismic Design .....	6-8
6.10 Dewatering and Drainage .....	6-9
6.11 Excavations.....	6-9
6.12 Engineered Fill and Foundation Backfill .....	6-10
7. MISCELLANEOUS INFORMATION.....	7-1
8. CLOSURE .....	8-1

## Drawings

Drawing 1	Borehole Location Plan
Drawing 2	Soil Strata

## Tables

Table 4-1	Borehole Numbers, Drilling Depths and Elevations.....	4-2
Table 4-2	Soil Testing Program – Englehart Yard .....	4-3
Table 5-1	Consolidation Test Results .....	5-3
Table 5-2	Summary of Groundwater Levels .....	5-5
Table 6-1	Foundation Design Alternatives .....	6-3
Table 6-2	Recommended Minimum Pavement Structural Thickness .....	6-8
Table 6-3	Recommended Unfactored Parameters for Temporary Shoring Design.....	6-9
Table 7-1	Summary of Task Responsibilities and Personnel .....	7-1

## Appendices

Appendix A	Borehole Explanation Forms, Borehole Logs
Appendix B	Summary of Particle Size Distribution Results (Table B1), Particle Size Distribution Analyses (Figures B1 to B4), Plasticity Charts (Figures B5 to B7), Consolidation Test Results (Figures B8 to B10)
Appendix C	Site Photographs

# 1. Introduction

GENIVAR Inc. (GENIVAR) was retained by the Ontario Ministry of Transportation Northeastern Region (MTO) to undertake a geotechnical investigation for the proposed construction of a sand / salt storage facility at the Englehart Patrol Yard, located on Highway 11 at the intersection of Third Street in the Town of Englehart, Ontario. The purpose of the investigation was to assess subsurface conditions at the site and provide recommendations for foundation design at the designated structure location.

The geotechnical investigation was conducted in accordance with MTO Agreement #5011-E-0010. This Foundation Investigation and Design Report includes factual results of the geotechnical investigation carried out at the Englehart site, including the field and laboratory testing information, and geotechnical recommendations for foundation design and construction, including a discussion on foundation design alternatives.

## 2. Site Description and Regional Geology

### 2.1 Site Description

The Englehart Patrol Yard (site) is located on Highway 11 at the northwest corner of the Highway 11 / Third Street intersection in the Town of Englehart, Ontario. The site layout is shown in Drawing 1 and colour photographs of the site are included in Appendix C.

The site is fairly level with a slight slope to the southwest, and is currently fenced with a drainage ditch along the southern boundary. Surrounding land uses include a baseball diamond, a school, mixed residential, and a Union Gas yard on the east side of Third Street.

The site is an operational MTO Patrol Yard, with access off of Third Street, and is currently occupied by a 6-bay garage/office, a storage shed, and two (2) sand/salt domes. The eastern most dome is structurally condemned and cannot be entered. An above ground diesel fuel storage tank is located off the north side of the storage shed. There is also an oil/water separator for the garage and an enclosed laydown area for pipes and culverts near the shed. The active site zone is partially paved, with a few soil / grass covered areas.

There are two (2) existing monitoring wells on the site, one located on the south side of the entrance at Third Street and one located at the northwest corner of the western most sand/salt dome. Construction details for the wells are not known, but water levels were measured as input to this report. The wells were not sampled.

### 2.2 Regional Geology

Two different map sources were consulted to determine the regional geology in the Englehart area: i) Map 2661 and Map P2292 'Quaternary Geology – Englehart Area' published by the Ontario Geological Survey (OGS), and ii) Map 5021 'Northern Ontario Engineering Geology Terrain Study Data Base Map – New Liskeard' published by the Ministry of Natural Resources (MNR).

Based on the mapping information, the site is reportedly covered by fine sand, silt, and clay glaciolacustrine deposits. Silty clay is the dominant sediment type in the Englehart area and deposits of this material are reportedly up to 100 m thick. The glaciolacustrine sediments are underlain by Precambrian bedrock, which are part of the Cobalt Group (Huronian Supergroup), a complex metasedimentary formation that includes the following lithologies: feldspathic to quartz arenites, arkoses, pebbly conglomerates, argillites, paraconglomerates, feldspathic wacke, orthoconglomerates, and laminated to massive siltstones. Bedrock was not encountered in the current site investigation, so actual bedrock types below the site and proposed structure are not known.

### 3. Historic Report Review

A previous geotechnical report for the Englehart Patrol Yard was obtained from the MTO Geocres Library in Downsview, Ontario. The report (Geocres No. 31M-22), titled '*Installation of Service Tank at D.H.O. Yard at Englehart*', was completed in 1962 for the installation of a service tank at the Englehart Yard. The investigation consisted of sampling two (2) borings and two (2) dynamic cone penetration tests in the northern portion of the site. Encountered soil conditions included approximately 0.5 m of sand, gravel, and occasional boulders, overlying silt with loose relative density to a depth of approximately 4.5 m below ground surface. Below 4.5 m, clayey silt with occasional thin silt seams was encountered to the borehole termination depth of 7.0 m below ground surface. Dynamic cone penetration testing was conducted from 7.0 m to a maximum depth of 15.2 m below ground surface. Based on the reported results, the consistency of the clayey silt was soft increasing to medium stiff with depth. Groundwater level was observed to be at elevation 208.4 m to 208.8 m.

## 4. Investigation Procedures

### 4.1 Subsurface Investigation

A borehole investigation was performed at the subject site between May 28 and May 30, 2012. The investigation consisted of advancing four (4) exploratory boreholes, designated as BH12-1 through BH12-4, commencing from existing ground level. Borehole locations are shown on Drawing 1 and were located at each of the four corners of the proposed storage structure, as required in the Terms of Reference.

MTO minimum requirements for the borehole investigation outlined a maximum drilling depth of 15.0 m, unless refusal was encountered at shallower depth, or justification for deeper drilling was authorized by the MTO Project Manager. In each of the four boreholes, stiff to firm clayey silt to silt and clay layers, as described in detail in Section 5, were encountered at 15.7 m below ground surface. Augering was terminated at this depth in all four boreholes. Approval was given by the MTO Project Manager to drive Dynamic Cone Penetration Tests (DCPT's) an additional 10 m depth at BH12-1 and BH12-2, and to refusal depth at BH12-3. Ultimately, BH12-3 was terminated on refusal at 43.9 m below ground surface (elevation 165.8 m).

The longitude and latitude of the individual borehole locations were obtained using a hand-held GPS unit in the WGS 84 reference system. These coordinates were provided to the MTO Project Manager for conversion to MTO standard coordinates (Northing and Easting). Borehole elevations were surveyed to a known benchmark: the cut cross in the footing wall of the eastern sand/salt dome, with a reported geodetic elevation of 209.957 metres above sea level (masl) was used. Borehole elevations and coordinates are shown on Drawing 1, and are provided on the borehole logs included in Appendix A.

Drilling and soil sampling was completed using a truck-mounted drill rig operating under the supervision of an experienced GENIVAR soils technician. The boreholes were advanced to the sampling depths by means of continuous flight hollow stem augers. Standard Penetration Test (SPT) N values were recorded for the sampled intervals as the number of blows required to drive a split spoon sampler 305 mm into the soil, using a 63.5 kg drop hammer falling 750 mm (ASTM D1586 procedure). Refusal depth for the purposes of this investigation was defined in the MTO Terms of Reference as the depth at which SPT N values exceed 100 blows for 305 mm of penetration. SPT N values are used in this report to assess consistency for cohesive soils and relative density for non-cohesive materials.

Soil samples were collected using SPT procedures at approximately 0.75 m intervals to 5.0 m depth, 1.5 m intervals to 20 m depth, and 3 m intervals beyond 20 m depth, as per the Terms of Reference. The sampled soil materials from discrete units were logged in the field using visual and tactile methods, and were then placed in labelled plastic bags for transport, future reference, possible laboratory testing, and storage. Soils for laboratory moisture content testing were placed in sealed laboratory jars for transport.

In cohesive deposits, where the consistency of the soil permitted, relatively undisturbed samples (TW) were taken with 70 mm diameter thin-walled (Shelby) tubes, which were pushed into the bottom of the borehole using the hydraulic ram rod of the drill rig. In situ undrained shear strength ( $c_u$ ) of the soil was measured using an ASTM tapered field vane and standardized procedures.

Dynamic cone penetration testing (DCPT) was completed below 15.7 m depth in boreholes BH12-1, BH12-2, and BH12-3 to further evaluate soil consistency at depth. In the DCPT, a 51 mm diameter, 60° Apex cone point, screw-attached to the tip of A-size rods, is driven into the ground using the same driving energy as in the SPT method. By recording the number of blows to drive the cone/rod assembly into the soil every 305 mm, a qualitative record of relative density/consistency is obtained. Although the interpretation of the test results may be difficult because no soil samples are obtained through this method, and the penetration resistances are not necessarily equivalent to N values or undrained shear



strengths, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic effects which may affect SPT N values. In some deposits, soil adhesion to the drill rod assembly may affect DCPT results, and therefore should be taken into account in the geotechnical assessments.

Groundwater conditions within the boreholes were observed during drilling, prior to backfilling. In addition, one (1) groundwater monitoring well was installed in borehole BH12-2 at a depth of 4.6 m below ground surface, to measure static groundwater levels at the site. The monitoring well was installed to meet Ontario Regulation (O. Reg.) 903 requirements, and consists of 51 mm (2 inch) outside diameter environmental grade PVC pipe, with a 1.5 m long No. 10 machine-slotted screen embedded within a sand pack. The sand pack was installed from the bottom of the monitoring well to a depth of approximately 0.3 m above the well screen. A bentonite seal was then placed between the top of the sand pack and the ground surface. The monitoring well is intended for temporary use only, and should be decommissioned prior to or during construction.

Remaining boreholes not completed as monitoring wells were backfilled with drill cuttings mixed with bentonite hole plug, and the top portion of the boreholes was sealed with emulsified asphalt. The backfill material was compacted with the drill rig. As such, the boreholes are abandoned in accordance with O. Reg. 903 requirements, as amended. Table 4.1 below summarizes the borehole numbers and drilling depths and the surveyed elevations.

**Table 4-1: Borehole Numbers, Drilling Depths and Elevations**

Borehole No.	Drilling Depth Below Existing Ground Surface (m) / Elevation (m)	Dynamic Cone Penetration Test Depth (m) / Elevation (m)	Monitoring Well
BH12-1	15.7/ 193.8	15.7 to 25.9 / 193.8 to 183.6	-
BH12-2	15.7/ 193.8	15.7 to 25.9 / 193.8 to 183.6	Monitoring well installed at 4.6 m depth/EI. 204.9 m
BH12-3	15.7/ 194.0	15.7 to 43.9 / 194.0 to 186.8	-
BH12-4	15.7/ 194.0	-	-

## 4.2 Laboratory Testing

The following soil testing program, as summarized in Table 4.2, was completed on selected soil samples to confirm the textural classifications and provide geotechnical parameters of the encountered materials.

**Table 4-2: Soil Testing Program – Englehart Yard**

Test	ASTM Standard	Number of Samples
Natural Moisture Content	ASTM D2216	50
Particle Size Analysis	ASTM D422	16
Atterberg Limits	ASTM D4318	14
Consolidation	ASTM D2435/D2435M-11	1

The minimum number of laboratory tests was set at 25 percent of the samples, according to the MTO Terms of Reference. Low complexity soil tests were completed at GENIVAR's RAQ's certified laboratory in Peterborough. Medium complexity (i.e. consolidation) tests were subcontracted to Golder Associates RAQ's certified laboratory in Mississauga. Laboratory testing results are presented on the borehole logs and in Appendix B. A summary of the particle size distribution results is also included as Table B1 in Appendix B.

## 5. Subsurface Conditions

The subsurface conditions were explored at the four (4) borehole locations designated as BH12-1 to BH12-4. Borehole locations are shown on Drawing 1 while the subsurface stratigraphic profile for the site is shown on Drawing 2. Detailed borehole logs are provided in Appendix A, and laboratory test results are included in Appendix B.

### 5.1 Soil Profile Summary

All four of the boreholes encountered a thin layer of asphalt overlying compact granular fill. A silt with some clay layer was encountered beneath the fill, and a relatively thick and firm clayey silt to silt and clay deposit was subsequently penetrated, extending to the borehole termination depth of 15.7 m below ground surface. Based on DCPT results, similar material likely extends to depth. BH12-3 encountered harder material below about 40 m depth (approximate elevation 170 m). Descriptions of the individual soil units are provided in the following subsections.

#### 5.1.1 Asphalt Pavement

A 65 mm to 100 mm thick layer of asphaltic concrete (hot laid mix) was encountered at the surface at each of the borehole locations.

#### 5.1.2 Granular Fill

Below the asphalt pavement, boreholes BH12-1 to BH12-4 encountered a granular fill layer (pavement base/subbase), consisting of 0.2 m to 0.3 m of sand and gravel to gravelly sand, underlain by sand with some gravel extending to the depths (metres below ground surface; mbgs) and elevations (geodetic) shown below:

<u>Borehole No.</u>	<u>Depth to Bottom of Fill Layer (Elevation)</u>
BH12-1	0.7 mbgs (208.8 m)
BH12-2	1.4 mbgs (208.1 m)
BH12-3	2.1 mbgs (207.6 m)
BH12-4	2.0 mbgs (207.7 m)

Laboratory particle size distribution analyses for two (2) samples from the fill layer were completed, and results according to the Unified Soil Classification System (USCS) are summarized below and shown on Figure B1 of Appendix B:

- Gravel (greater than 4.75 mm size) - 24 % to 45 %
- Sand (0.075 mm to 4.75 mm size) - 48 % to 67 %
- Silt and Clay (less than 0.075 mm size) - 7 % to 9 %

Standard Penetration Test results (N Values) recorded in the fill layer ranged between 13 and 16 blows per 305 mm of penetration, indicating compact relative density.

Laboratory determined moisture contents ranged between 4 % and 15 % for samples of the fill, indicating moist to wet material.

### 5.1.3 Silt

Beneath the granular fill layer, a layer of silt with a trace to some clay and a trace to some fine sand was encountered extending to depths (metres below ground surface; mbgs) and elevations (geodetic) shown below:

<u>Borehole No.</u>	<u>Depth to Bottom of Silt Layer (Elevation)</u>
BH12-1	7.1 mbgl (202.2 m)
BH12-2	7.0 mbgl (202.5 m)
BH12-3	5.7 mbgl (204.0 m)
BH12-4	3.7 mbgl (206.0 m)

Thus, the thickness of the silt layer varied from 1.7 m at borehole BH12-4 to 5.6 m at borehole BH12-2.

Laboratory particle size distribution analyses for six (6) samples from the silt layer were completed, and results according to USCS are summarized below and shown on Figure B2 of Appendix B:

- Gravel (greater than 4.75 mm size) - 0 % to 1 %
- Sand (0.075 mm to 4.75 mm size) - 1 % to 17 %
- Silt (0.002 mm to 0.075 mm size) - 70 % to 84 %
- Clay (less than 0.002 mm size) - 8 % to 15 %

Standard Penetration Test results (N values) recorded in the silt deposit ranged from 3 to 24 blows per 305 mm of penetration. Undrained shear strengths as measured by Field Vane methods ranged from 37 kPa to 42 kPa. Based on these results, the consistency of the silt deposit is described as firm. The sensitivity of the silt layer ranged from 1.9 and 3.7 (low to medium sensitivity).

Atterberg Limits tests performed on three (3) samples from the silt deposit yielded the following index values:

- Liquid Limit ( $w_L$ ) - 19 % to 22 %
- Plastic Limit ( $w_P$ ) - 18 % to 20 %
- Plasticity Index ( $I_P$ ) - 1 % to 3 %

From the USCS plasticity chart included as Figure B5 in Appendix B, the samples may be classified as inorganic silt of low plasticity (ML).

The natural moisture content of samples recovered from the silt layer ranged from 13 % to 30 % based on laboratory testing.

### 5.1.4 Clayey Silt

A relatively thick clayey silt layer was encountered beneath the silt layer in all four boreholes. In borehole BH12-1 and BH12-2, the clayey silt layer extended to the borehole termination depth of 15.7 m below

ground surface, while in boreholes BH12-3 and BH12-4 the clayey silt layer extended to a depth of 12.7 m below ground surface, where the material changed to silt and clay (see subsection 5.1.5).

Laboratory particle size distribution analyses for six (6) samples of the clayey silt unit were completed, and results are summarized below and shown on Figure B3 of Appendix B:

- Gravel (greater than 4.75 mm size) - 0 % to 4 %
- Sand (0.075 mm to 4.75 mm size) - 1 % to 6 %
- Silt (0.002 mm to 0.075 mm size) - 69 % to 78 %
- Clay (less than 0.002 mm size) - 21 % to 30 %

Standard Penetration Test results (N values) recorded for the clayey silt layer ranged from 0 to 7 blows per 305 mm of penetration. Undrained shear strength, as measured by Field Vane tests, ranged from 37 kPa to 70 kPa. Based on the field results, the consistency of the clayey silt deposit is described as firm to stiff. A weak horizontal varved structure was occasionally observed in borehole samples, as noted in the logs. The sensitivity of the clayey silt layer ranged from 2.1 and 3.0 (medium sensitivity). Atterberg Limits tests for three (3) samples from the deposit yielded the following index values:

- Liquid Limit ( $w_L$ ) - 21 %
- Plastic Limit ( $w_P$ ) - 16 % to 19 %
- Plasticity Index ( $I_P$ ) - 2 % to 5 %

From the USCS plasticity chart included as Figure B6 in Appendix B, the samples may be classified as inorganic clay to silt of low plasticity (CL-ML).

One (1) oedometer test (one dimensional consolidation) was performed on a 70 mm diameter Shelby tube (TW) sample taken from a depth of 8.7 m below ground surface in borehole BH12-3. The results are presented in Figures B8 through B10 in Appendix B. Using the Casagrande method it was determined that the existing overburden pressure ( $\sigma_o^l$ ) is equal to the pre-consolidation pressure ( $\sigma_c^l$ ). Therefore, the soil is considered to be normally consolidated. A summary of the consolidation test results is provided in Table 5.1.

**Table 5-1: Consolidation Test Results**

Parameter	Measurement
Measured Bulk Unit Weight, $\gamma$ ( $\text{kN/m}^3$ )	18.8
Compression Index, $C_c$	0.22
Swelling Index, $c_s$	0.027
Coefficient of Consolidation, $c_v$ ( $\text{cm}^2/\text{s}$ )	0.008

Laboratory determined moisture content ranged between 27 % and 34 % for the clayey silt samples, indicating wet material with moisture content above the Liquid Limit ( $w_L$ ).

### 5.1.5 Silt and Clay

Underlying the clayey silt layer, a layer of silt and clay with a trace of sand was encountered at boreholes BH12-3 and BH12-4, at a depth of 12.7 m below ground surface (elevation 197.0 m). The unit extended to the borehole termination depths of 15.7 m (elevation 194.0 m).

Laboratory particle size distribution analyses for two (2) samples from the silt and clay layer were completed, and results are summarized below and shown in Figure B4 of Appendix B:

- |                                      |   |              |
|--------------------------------------|---|--------------|
| ➤ Gravel (greater than 4.75 mm size) | - | 0 %          |
| ➤ Sand (0.075 mm to 4.75 mm size)    | - | 2 %          |
| ➤ Silt (0.002 mm to 0.075 mm size)   | - | 56 % to 61 % |
| ➤ Clay (less than 0.002 mm size)     | - | 37 % to 42 % |

Undrained shear strength, as measured by Field Vane tests, ranged from 37 kPa to 50 kPa, indicating the consistency of the silt and clay deposit is firm. The sensitivity of the silt and clay ranged from between 2.0 and 2.5 (medium sensitivity). It is notable that SPT N values were less than 1 in this material (weight of hammer).

Atterberg Limits tests for two (2) samples from the deposit yielded the following index values:

- |                              |   |              |
|------------------------------|---|--------------|
| ➤ Liquid Limit ( $w_L$ )     | - | 25 % to 28 % |
| ➤ Plastic Limit ( $w_P$ )    | - | 16 %         |
| ➤ Plasticity Index ( $I_P$ ) | - | 9 % to 12 %  |

From the USCS plasticity chart included as Figure B7 of Appendix B, the samples may be classified as inorganic clay of low plasticity (CL).

Laboratory determined moisture content ranged from 30 % to 34 % for the silt and clay samples indicating wet material with moisture content above the liquid limit.

### 5.1.6 Dynamic Cone Penetration Testing

Dynamic cone penetration testing (DCPT) was performed below the borehole termination depth of 15.7 m at boreholes BH12-1, BH12-2, and BH12-3. The DCPT's extended to a depth of 25.9 m below ground surface (elevation 183.6 m) at BH12-1 and BH12-2, and to 43.9 m below ground surface (elevation 165.8 m) at BH12-3. Refusal, defined by MTO as 100 blows per 305 mm of penetration, was encountered at BH12-3 at a depth of 41.5 m below ground surface (elevation 168.2 m).

Refusal conditions extended to 43.9 m (elevation 165.8 m), upon which the DCPT was terminated in the very dense/hard material.

The DCPT results indicate that the firm to stiff clayey silt and/or silt and clay deposits may extend to a depth of 41.5 m below ground surface (elevation 168.2 m).

## 5.2 Groundwater Conditions

Groundwater conditions were observed in the open boreholes upon completion of drilling. The static water levels in the BH12-2 monitoring well (MW) and the two (2) existing onsite monitoring wells also were measured. Results are summarized in Table 5.1.

**Table 5-2: Summary of Groundwater Levels**

Location	Measured Groundwater Depth mbgs (elevation m)	Date Measured
BH12-1	0.9 (208.6)	28 May 2012
BH12-2 (MW)	1.0 (208.5)	30 May 2012 (one day after completion)
BH12-3	1.0 (208.7)	30 May 2012
BH12-4	1.0 (208.7)	30 May 2012
MW (entrance)	0.7 (n/a)	30 May 2012
MW (NW corner sand dome)	1.2 (n/a)	30 May 2012

Note: mbgs = metres below ground surface; MW signifies monitoring well.

Based on the water level measurements and moisture condition of the inspected soil samples, the groundwater level within the footprint of the proposed structure, at the time of the field investigation, was estimated to be 1.0 m below ground surface.

It should be noted that groundwater levels may fluctuate seasonally and in response to climatic conditions. Due to the fine-grained soils at the site, a potential for development of perched groundwater exists after wet seasons and periods of rainfall, and groundwater may rise close to the ground surface.

## 6. Geotechnical Design Considerations

The proposed sand/salt storage facility at Englehart Patrol Yard will replace an existing condemned dome, and will have a rectangular footprint of approximate dimensions 18.3 m x 24.4 m. Foundation engineering guidelines presented in this section have been developed based on the soil conditions investigated and described in Section 5, and in accordance with the most recent edition of the Canadian Highway Bridge Design Code (CHBDC) and the most recent edition of the Canadian Building Code in effect for MTO projects.

### 6.1 “Red Flag” Conditions

Soil and groundwater conditions at the Englehart site present some challenges for design and construction of the foundation for the new sand/salt storage facility.

A relatively thick clayey silt to silt and clay deposit is present below the structure and is prone to consolidation settlement due to the structural loadings, and more importantly loadings imposed by the sand/salt stockpiles. It is recognized that an existing dome is present in the location of the future structure, and that previous material stockpiles have created a pre-consolidation effect. However, since boreholes could not be advanced within the center of structure where the consolidation effect should be greatest, recommendations in this report are based soil information obtained just outside of the structure near the edge of the load influence zone. Therefore, theoretical settlement potential was predicted using the consolidation data for the clay layer obtained from the One-Dimensional Consolidation Test as outlined in Section 6.2. The settlement analysis assumes a 5.0 m thick of upper silt layer underlain by a 35.0 m thick layer of clayey silt to silt and clay, and considers three scenarios for the loading imposed by sand and salt stockpiles. Settlement potential and mitigation measures also are discussed in Section 6.2, and foundation design options subsequently discussed in Section 6.3 are presented under the assumption that settlement potential due to the loadings is mitigated in advance of construction. Otherwise, structural adjustments and building maintenance may be required.

A high groundwater table, generally within 1 m of the ground surface presents construction challenges for foundation construction. Groundwater may have to be pumped from construction excavations. Wet silt layers at shallow depths are prone to disturbance by construction equipment and workers, and protective measures are required to maintain adequate stability and foundation bearing capacity. Mitigation measures for groundwater are provided in Section 6.7.

### 6.2 Mitigation of Settlement Potential

As described, the proposed storage structure at the Englehart yard is underlain by a firm to stiff clayey silt to silt and clay layer estimated to be about 35 m thick. It is understood that the existing storage dome, which has been condemned due to structural damage related to settlement, will be demolished and removed, and that the new building will be erected at the same location. It is inferred that previous surcharge loadings from stockpiled fill have consolidated the storage structure area; however, some residual settlement potential from the proposed loading may remain. No borehole information is available inside the old dome, and there is a possibility that the new structure footprint could move slightly. Therefore, mitigation of the settlement potential is required.



Based on laboratory test results for soil samples taken in boreholes at the corners of the proposed building, outside of the existing dome and former stockpile loading area, the following consolidation data for the clayey unit was obtained for settlement considerations:

- Initial void ratio ( $e_0$ ) = 0.89
- Bulk unit weight ( $\gamma$ ) = 18.8 kN/m<sup>3</sup>
- Compression Index ( $C_c$ ) = 0.22
- Swelling Index ( $c_s$ ) = 0.027
- Coefficient of Consolidation ( $c_v$ ) = 0.008 cm<sup>2</sup>/s

The following three scenarios were considered to evaluate settlement potential due to loadings imposed by the sand and salt stockpiles within the storage facility:

- Scenario No.1: Salt stockpiles placed to the rear of the facility to the maximum allowable height of the “push wall”, with the stockpile periodically replenished throughout the winter, assuming 9800 kN (1000 Tonnes) loading;
- Scenario No.2: 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 4900 kN (500 Tonnes) salt stock pile within the front  $\frac{1}{4}$  of the building.
- Scenario No.3: The storage facility will be loaded to its full allowable capacity. This scenario would consist of winter sand stacked to the maximum 3.6 m allowable height of the “push wall”, with a stockpile area covering the entire footprint of the building.

The estimated effective stress increase ( $\Delta p$ ), and the the total and differential settlements for each case scenario are as follows;

Scenario No.	Effective Stress Increase ( $\Delta p$ ), kN/m <sup>2</sup>	Total Settlement (mm)	Differential Settlement(mm)
Scenario No.1	6.0	50	25
Scenario No.2	10.5	95	50
Scenario No.3	15.0	120	55

Therefore consolidation settlement for the worst case scenario, assuming a 35.0 m thick layer of normally of consolidated clayey soil is estimated at 110 mm. In addition, there is 10 mm of immediate settlement potential associated with the shallow, 5 m thick silt layer below the foundation. Thus, the total settlement potential under the proposed stockpile/structural loading is estimated at 120 mm. Differential settlement potential is estimated at 55 mm.

If warranted by building design tolerances for deflection, the settlement potential due mainly to stockpile loading could be mitigated with a preloading program using a fill surcharge and possibly vertical wick drainage. Wick drains, if used, would need to extend to a depth of 30 m to 40 m below ground and be installed on a triangular grid at an approximate spacing of 2 m. The height of the surcharge should be at least 2.5 m and extend at least 2.5 m beyond the structure footprint, and should remain in place for at least six (6) months. At least five total station survey markers and/or settlement plates should be used within the building footprint area (i.e. building corners and centre) to monitor ground movement. We expect that the preloading program would reduce total foundation settlement potential from the future stockpile loading to less than 25 mm, under the heaviest loading condition. Applying a higher surcharge loading for a longer period could alleviate the need for wick drains, but the duration of preloading in this case would be in excess of one (1) year.

If the proposed building can tolerate up to 110 mm of settlement (50 mm differential), or if the structure can be equipped with adjustable supports that MTO can maintain, then the preloading option may not be

required. Alternately, if reduced stockpile loadings can be used and building settlements can be monitored and adjusted as required, then pre-loading may not be required.

## 6.3 Structure Foundation Design Options

Based on the results of this investigation, several foundation options are available, including shallow and deep foundations. The preferred foundation option should be determined in view of following factors:

- Existing Subsurface Conditions
- Serviceability
- Advantages\Disadvantages
- Reliability
- Risks /Consequences

Comments for consideration are provided in the following table.

**Table 6-1: Foundation Design Alternatives**

Foundation Type	Subsurface Conditions	Advantages\Disadvantages	Reliability	Risk / Consequences
<b>Strip Footing on Native Silt Layer</b>	Relatively low geotechnical resistance	Low cost, lower foundation capacity verses deep foundation, High total and differential settlement, Relatively difficult construction if above ground-water table Requires greater effort to control groundwater and prevent subgrade disturbance	Good provided silt is undisturbed (good construction practices required). Foundation must be below frost or insulated.	Risk of high groundwater; risk of subgrade disturbance and subexcavation being needed;; pumping may be required depending on seasonal conditions; difficulties during construction since excavation will be within the water bearing silt layer; and shoring will be necessary
<b>Strip Footing on Engineered Fill</b>	Medium geotechnical resistance	Low to medium cost, higher foundation capacity verses footing on native silt, larger foundation settlement versus deep foundation	Good, provided that good fill quality and compaction is used. Insulation may be required to protect against frost heaving.	Risk of high groundwater; risk of subgrade disturbance and subexcavation being needed; pumping may be required to control groundwater depending on seasonal conditions;. excavation will be within the water bearing silt layer; and shoring will be necessary
<b>Slab-on-Grade</b>	Medium geotechnical resistance	Medium cost,-insulation required, larger foundation settlement versus deep	Good. Insulation required and must extend beyond structure.	Removal of shallow deleterious material and/or existing soil improvement is required. Larger excavation/disturbed

		foundation		area required for insulation component.
<b>Drilled and Cast-in-Place Concrete Foundation</b>	High geotechnical resistance	High bearing resistance, low settlement, protection of subgrade against disturbance not as critical as for shallow foundations, high cost	Good	Must extend to deeper competent material. Liner may be required. Required additional drilling to prove bedrock
<b>Steel H Piles</b>	High geotechnical resistance	High bearing resistance, negligible settlement, protection of subgrade against disturbance not as critical as for shallow foundations, high cost	Good	Must extend to deeper competent material. Vibrations and/or soil disturbance may be an issue for nearby structures. Required additional drilling to prove bedrock

## 6.4 Frost Penetration Depth

The recommended design frost protection depth for the site area is 2.2 m (Source: MTO Pavement Design and Rehabilitation Manual). Therefore, a permanent soil cover of about 2.2 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. In case of rockfill, only one-half of the rockfill thickness should be assumed to be effective in providing frost protection. In consideration of the depth of required soil cover for frost protection and the high groundwater conditions, it is assumed that a grade raise around the structure area is not an option.

## 6.5 Preferred Foundation Option

Based on an assessment of foundation design alternatives, the preferred foundation design option is to construct the foundation using shallow strip footings based on the firm silt some clay layer between elevation 207 m and 208 m. If groundwater and disturbance of the silt bearing surface is a concern, then the footing excavation can be widened and progressively backfilled with OPSS 19 mm diameter clear stone wrapped in medium duty non-woven geotextile. In this case, the footing should be constructed at elevation 208.7 m (groundwater table measured), and approximately 1 m of clear stone would be required below the footing grade. The top of the stone layer should be at least 100 mm wider than the footing at each side, and the sides of the stone layer should slope at 1H:2V or flatter to the base of the excavation. Thus, the anticipated width of the base of the stone layer is approximately 1.5 m to 2 m. Progressive

backfill procedures require that short sections of trench be excavated, lined with geotextile and filled with stone, to maintain stability of the saturated silty soil.

The clear stone is not frost susceptible so lateral foundation insulation is not required if this design procedure is used. Since the footing would be located above frost depth, however, placement of 50 mm of high density insulation on both sides of the foundation wall is recommended for protection against frost adhesion. If conditions at the time of construction preclude the ability to construct the strip footings directly on the silt layer, then an equivalent thickness of stone base should be used around the entire perimeter of the foundation.

For minimum 1.5 m wide strip footings placed on the undisturbed native silt layer, the following geotechnical resistances would be available:

- Factored Geotechnical Resistance at Ultimate Limit State (ULS) = 100 kPa
- Geotechnical Resistance at Serviceability Limit State (SLS) = 65 kPa

Design resistances will increase to 240 kPa (ULS)/ 150 kPa (SLS) if the clear stone base layer is used.

In either case, existing fill materials overlying the silt layer are not suitable as structural material and should be removed to full depth. The founding subsoil must be inspected by the Geotechnical Engineer to confirm that it is suitable to support the design loads, and to confirm that all disturbed or loose soils are properly removed from below all footing areas. It should be noted that the silt layer at the anticipated founding level can be easily disturbed by foot traffic. Thus, the base must be covered with a minimum 50 mm thick mud slab immediately after inspection and approval.

## 6.6 Other Foundation Options

If reasons determine that the preferred option cannot be used, the following information is provided to assist designers with alternate design strategies.

### 6.6.1 Slab-on-Grade

The site is suitable for construction of a thickened slab on grade foundation after the removal of the fill layer and any weak soils to a depth ranging from 0.8 m to 2.1 m below ground surface.

The exposed subgrade should be inspected and approved by a Geotechnical Engineer and then should be surface rolled using a heavy roller. Any weak zones that become evident should be removed and replaced with drier, compactable material that is compatible with the subgrade soil. The subgrade should be compacted using a suitable heavy compactor, and a geotextile separator (e.g. Terrafix 270R or equivalent) should be placed on top of the subgrade prior to the placement of engineered fill.

The engineered fill layer could be compacted granular material (OPSS 1010-3 Granular B or equivalent) placed in maximum 200 mm thick lifts, with each lift should be uniformly compacted to at least 98% Standard Proctor Maximums Dry Density (SPMDD) per ASTM D698 procedures. Alternately, if groundwater is a problem for compaction, OPSS 19 mm diameter clear stone could be used. Filtered perimeter drains may be necessary for temporary groundwater control to place the stone.

A minimum 150 mm thick layer (after compaction) of OPSS 1010-3 Granular "A" or equivalent, should be placed immediately beneath the floor slab, and should be compacted to at least 100 % of SPMDD.

## 6.6.2 Deep Foundations

Considering the groundwater, soil disturbance and settlement potential for construction of shallow foundation options, some designers may wish to consider a deep foundation alternative. Cost will be considerably higher, and as such these options are not recommended.

### 6.6.2.1 Drilled and Cast-in-Place Concrete Foundation (Caisson)

Founding depths for potential caissons was not determined as part of this investigation, but DCPT results indicate that the native soils became very dense/hard below depth/elevation of 40 m/168.0 m. Thus, augured piers or caissons would likely have to be founded at minimum depth/elevation of 41 m/167.0 m. The following resistance values could be considered for caisson design:

- Factored Geotechnical Resistance at ULS = 750 kPa
- Geotechnical Resistance at SLS = 500 kPa

The above geotechnical resistance assume that proper construction procedures are followed during construction to ensure that if necessary the excavations are properly dewatered and that the founding soils are not disturbed prior to concrete replacement. The expected groundwater is below 1.0 m and, based on the presence of silt layer, a caisson excavation should be advanced in conjunction with a liner to prevent side wall failures.

### 6.6.2.2 Driven Piles

The firm to stiff clayey silt to silt and clay overlying very dense / hard soil is suitable for use of low displacement piles, such as steel H-piles or steel tube piles.

The most practical option appears to drive the piles into the very dense / hard soils underlying the clay deposit, thus utilizing both side resistance and end bearing.

Recommended pile resistances for HP 310 × 110 steel piles with minimum pile tip depth/elevation of 43.0 m/166.0 m are:

- Factored Geotechnical Resistance at ULS = 1,200 kN/pile
- Geotechnical Resistance at SLS = 800 kN/pile

The pile will need to be driven using a hammer capable of delivering a rated energy of at least 55 kilojoules/blow, but not more than 70 kilojoules/blow. Driving should be monitored in the field using a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles by the Hiley Formula can be calculated by dividing the recommended axial resistance at ULS by a resistance factor of 0.4, as per current MTO practice. As the actual driving of the piles in the field will be governed by the Hiley Formula, the recommended minimum pile tip depth/elevation of 43.0 m/166.0 mASL is for general guidance purposes only, and actual pile lengths may vary.

During the diving process, piles which have already been driven will need to be monitored to determine if they are heaving due to the effects of driving of adjacent piles. If heaving occurs, the effected piles will need to be re-driven.

Due to the expected 3.6 m height of the sand/salt stockpiles and the presence of normally consolidated clay, some downdrag settlement of the piles should be expected.

In cohesive soils, the coefficient of horizontal subgrade reaction may be estimated from:

$$K_s = 67 C_u / d$$

Where:

$K_s$  = coefficient of horizontal subgrade reaction

$C_u$  = undrained shear strength

$d$  = width of pile

For estimating purposes, the recommended lateral resistance for HP 310 × 110 steel H-piles are as follows:

- Factored Lateral Resistance at ULS = 120 kN/pile
- Lateral Resistance at SLS = 50 kN/pile

## 6.7 Resistance to Lateral Loads

Resistance to lateral forces/sliding between shallow concrete footings and subsoils should be calculated in accordance with Section 6.7.5 of the CHBDC. The adhesion ( $C_a$ ) which develops for cast-in-place concrete footings constructed on the undisturbed silt with some clay soil may be taken as 70 percent of the undrained shear strength ( $c_u$ ). Thus, the average adhesion value for foundation design should be taken as 25 kPa. This value shall be factored in accordance with the CHBDC, and a reduction factor of 0.8 is to be applied in calculating horizontal resistance. Resistance to lateral loads could be increased by constructing a shear key at the bottom of the footing. The design of shear keys would require a specific analysis taking into consideration the magnitude of the horizontal loading, the magnitude of the vertical loading, and any variations in the bearing pressure due to overturning moments.

The above guidelines assume that the subgrade materials will not be disturbed by construction activities.

## 6.8 Earth Pressure Design

Should temporary shoring be required to support excavations, shoring systems should be designed by a Professional Engineer experienced in this type of work.

In Ontario, shoring typically consists of soldier pile and timber lagging or sheet piling (with or without bracing/rakers). The shoring system should be designed so that the lateral movement of any portion of the supported excavation will not exceed the established criterion for the structural performance level.

Shoring walls below grade can be designed using the following expression:

$$P = K(\gamma h + q) \pm 2C\sqrt{K}$$

where:

$P$  = lateral earth pressure (kPa) acting at depth  $h$

$K$  = earth pressure coefficient

$\gamma$  = unit weight of backfill ( $\text{kN/m}^3$ )

$h$  = depth to point of interest in metres

$q$  = equivalent value of surcharge on the ground surface in kPa

$C$  = cohesion (kPa) acting at depth  $h$

( $C = 37$  kPa for the silt and clayey silt layer and 50 kPa for the silt and clay layer)

\*\* minus sign is to be used for active pressure and plus sign is to be used for passive pressure calculation

The above expression assumes that the perimeter drainage system prevents the build up of any hydrostatic pressure behind the wall and backfilling materials.

The coefficients of lateral earth pressure given in Table 6-3 may be used for the design of the temporary shoring systems, based on the borehole results.

**Table 6-2: Recommended Unfactored Parameters for Temporary Shoring Design**

Soil Type	$K_a$	$K_o$	$K_p$	$\gamma$ ( $\text{kN/m}^3$ )
Granular Fill	0.33	0.5	3.0	19.0
Firm Silt	0.38	0.55	2.6	18.0
Firm to Stiff Clayey Silt	0.35	0.51	2.8	18.5

## 6.9 Seismic Design

The Ontario Building Code (OBC) specifies that the structure should be designed to withstand forces due to earthquakes. For the purpose of earthquake design the information relevant to the geotechnical conditions at this site is the 'Site Class'. Based on the explored soil properties and in accordance with Table 4.1.8.4.A of the Ontario Building Code (2006), it is recommended that Site Class 'D' (stiff soil) be applied for structural design at this site.

Seismic information for the Englehart site is provided in the table below. Data from the 2005 National Building Code Seismic Hazard Calculation is provided in this table to be consistent with the 2006 Ontario Building Code.

Parameter	Englehart	Source
Site Class	D	2006 Ontario Building Code Table 4.1.8.4.A
$S_a(0.2)$	0.253	2005 National Building Code Seismic Hazard Calculation
$S_a(1.0)$	0.061	2005 National Building Code Seismic Hazard Calculation
$F_a$	1.3	2006 Ontario Building Code Table 4.1.8.4.B
$F_v$	1.4	2006 Ontario Building Code Table 4.1.8.4.C

Generally, the looser the sediment, and the higher the water table, the more susceptible the soil is to liquefaction. Based on the texture and density of the subsurface soils at the site and the high groundwater level, and since the silt and the clayey silt layer has a plasticity index ( $I_p \leq 12$  and a water content to liquid limit ratio ( $w/w_L \geq 0.85$ ), it is inferred that the fine-grained soils behave as "sand-like" (i.e. potentially susceptible to liquefaction). It is recommended that designers consider a residual strength value ( $S_r$ ) of 15 kPa for the silt unit and 20 kPa for the clayey silt/silt and clay.

## 6.10 Dewatering and Drainage

It is anticipated that the foundation excavation will encounter groundwater at a shallow depth, approximately 1.0 m below the ground surface. Fluctuating groundwater levels and/or perched groundwater should be anticipated at the site. Therefore, dewatering may be required to stabilize the soil and facilitate construction. We recommend that this condition be "red flagged" to the contractor.

It is believed that the groundwater can be lowered by about 0.75 m by pumping from strategically placed filtered sumps and using gravity drainage. For more extensive drawdown of water levels, vacuum well points and/or deep wells would be required. It is recommended that the Contractor be requested to submit dewatering schemes to the MTO Project Manager for approval. Dewatering procedures should follow the requirements and specifications of OPSS 517.

The predominant soils encountered in the boreholes range in texture from upper granular fill underlain by silt, clayey silt to silt and clay. Seepage from the granular fill and the silt layer soils will be expected to be medium to relatively high. The clayey silt to silty clay soils generally exhibit characteristics of low permeability and seepage from this layer into foundation excavations would be expected to be relatively slow.

## 6.11 Excavations

It is anticipated that the excavations for the construction will comprise the excavations of the footings and utility services. Temporary excavations must be carried out in accordance with the latest edition of Ontario Regulation (O.Reg.) 213/91 of the Occupational Health and Safety Act (OHSA) as well as MTO specifications SP 105 S19 – Protection Systems and SP 902 S01 – Excavations and Backfilling to Structure. The soils at the site may be classified as shown below, in accordance with the OHSA.

**Table 6-3: Soil Classification for Excavations**

Soil Type	Above Groundwater Level	Below Groundwater Level
Fill material	Type 3	Type 4
Firm silt some clay	Type 3	Type 4
Firm to stiff clayey silt / silt and clay	Type 2	Type 3

Type 2 excavations may have vertical sides for the bottom 1.2 m of the excavation and then should be cut with 1H:1V or flatter side slopes. Type 3 excavations should be cut with 1H:1V or flatter side slopes. Type 4 excavations should be cut with 3H:1V or flatter side slopes. If the appropriate side slopes cannot be achieved, the excavations must be properly supported (shored).



Excavations should be protected from exposure to precipitation and associated ground surface runoff and should be inspected regularly for signs of instability. If localized instability is noted during excavation or if wet conditions are encountered, excavation side slopes should be flattened as required to maintain safe working conditions.

All excavation and grading procedures should follow the requirements and specifications of SP 206, and management of excess material should follow the requirements of OPSS 180.

Depending on the construction procedures to be used, the Contractor should obtain a Permit to Take Water under Section 34 of the Ontario Water Resources Act if pumping rates will exceed 50,000 L/day.

## 6.12 Engineered Fill and Foundation Backfill

It is understood that the proposed sand/salt structure is to be constructed at the location of the currently condemned, eastern most dome, following its demolition.. Therefore, if engineered fill is required to replace the fill materials, loose native soil, and/or to develop the design grades and elevations, an approved clean soil fill should be used.

Based on the visual and tactile examination of the soil samples, on-site excavated fill material may be suitable for re-use as backfill material if it is environmentally acceptable and provided it does not contain particles greater than 25 mm in diameter, topsoil, or other deleterious materials. The excavated silt some clay soil will not be suitable for reuse as fill material due to inefficient response to the compaction efforts unless it is mixed with clay, sand and gravel to improve performance. If the soils are too wet, it will not be possible to achieve proper compaction.

All fill materials imported to the site must meet applicable MTO standards, and provincial and federal guidelines. Prior to the placement of engineered fill, it is recommended that all existing fill material, loose sand, and soft silty clay to clayey silt soils be stripped from beneath and at least 2 m beyond the proposed building and parking/apron area envelopes, and that the subgrade be proof-rolled. Any soft or wet areas which deflect excessively during the proof rolling, should be subexcavated and replaced with suitably compacted clean earth fill placed in maximum 200 mm thick lifts and compacted to a minimum of 98 percent Standard Proctor Maximum Dry Density (100% under foundations). If construction operations are undertaken in the winter, strict consideration should be given to the condition of the backfill material to make certain that frozen material is not used. It may be necessary to backfill excavations with imported granular fill material, such as OPSS 1010-3 Granular B, or an approved equivalent.

Within the building footprint, a minimum 150 mm thick layer (after compaction) of OPSS 1010-3 Granular A, or equivalent, should be placed immediately beneath the asphalt pavement, and should be compacted to at least 100 % of SPMDD.

## 7. Miscellaneous Information

The following GENIVAR personnel and subcontractors responsible for completion of this geotechnical investigation are summarized in Table 7-1.

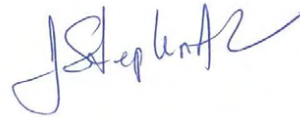
**Table 7-1: Summary of Task Responsibilities and Personnel**

Task	Name	Address	Phone
Buried Utility Locates	Peter Flowerday Central Cable Contractors	Wanapitae, ON	705-694-5256
Drilling	Kyle Gilmore Abraflex Drilling	Lively, ON	705-222-2272
Field Supervision	Dave Lembke, C.E.T., rcji GENIVAR Inc.	Peterborough, ON	705-743-6850
Project Coordinator	Jennifer Wales, P. Eng. GENIVAR Inc.	Peterborough, ON	705-743-6850
Laboratory Low Complexity	Kelly Whitney, C.E.T. GENIVAR Inc.	Peterborough, ON	705-743-6850
Laboratory Medium Complexity	Marijana Manojlovic, B.Sc. Golder Associates	Mississauga, ON	905-567-4444
Report Preparation	Raid Khamis, P. Eng. GENIVAR Inc.	Brampton, ON	905-799-8220
Report Review	Steve Ash, P. Eng., P. Geo. GENIVAR Inc.	Peterborough, ON	705-743-6850
RAQ's Key Contact	Andrew Hims, P. Eng. GENIVAR Inc.	Collingwood, ON	705-444-2788

## 8. Closure

The data presented in this geotechnical report, and the quality thereof, is based on a scope of work authorized by the Client. While we believe the borehole information to be representative of site conditions, subsurface conditions between and beyond the test hole locations may vary. GENIVAR accepts no liability for use of or reliance on the report information by third parties, without express written consent.

Prepared by:  
**GENIVAR Inc.**



Raid Khamis, P. Eng., PMP, LEED® AP BD+C  
Project Engineer

J. Stephen Ash, P. Eng., P.Geo.  
Consulting Engineer/Business Unit Leader

Reviewed by:



Andrew G. Hims, M.Sc., P. Eng.  
Consulting Engineer

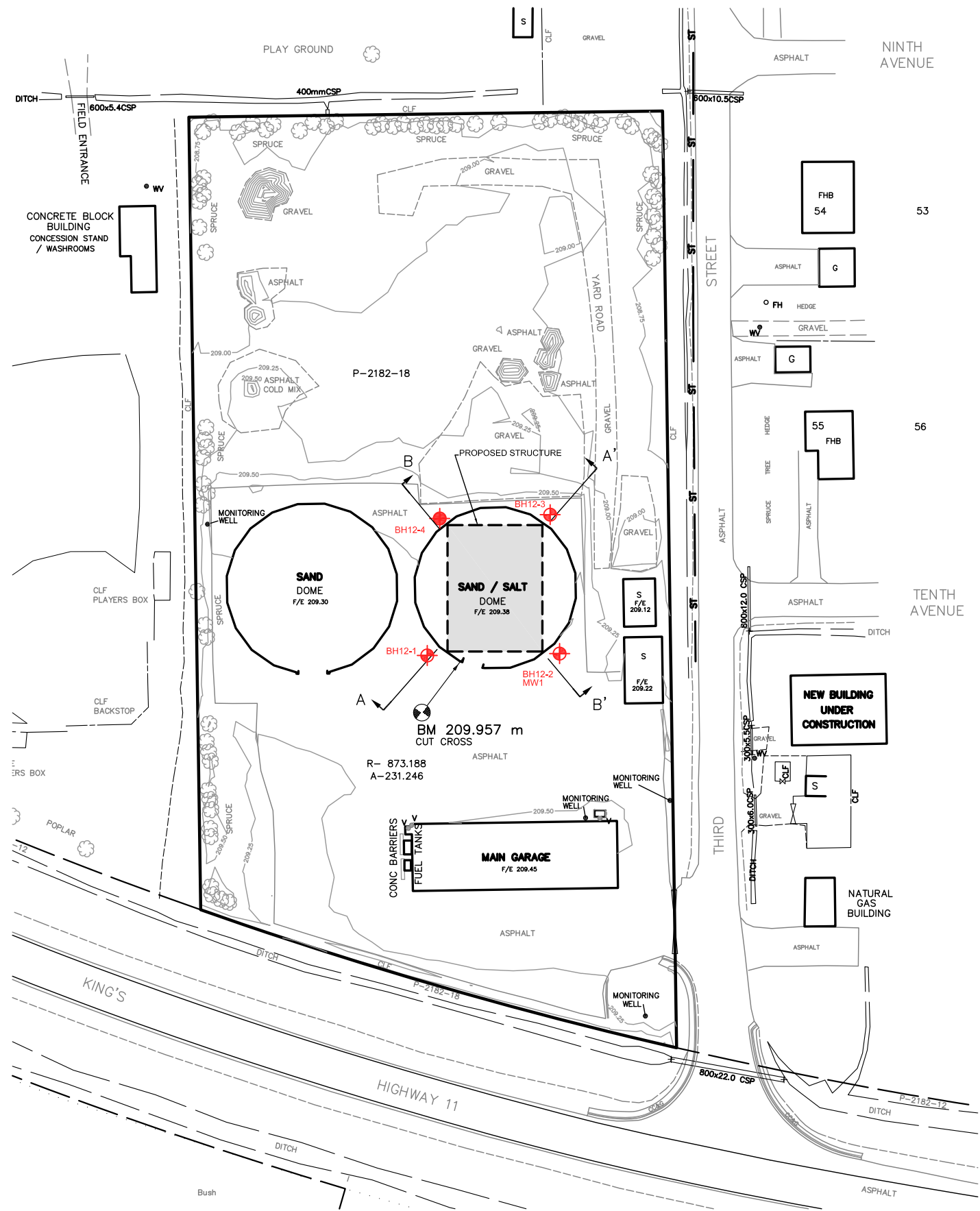
---

## Drawings

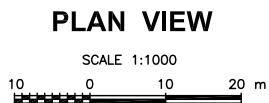
Drawing 1 – Borehole Location

Drawing 2 – Soil Strata

---



- NOTES:
1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
  2. COORDINATES AT BOREHOLE LOCATIONS WERE BY HANDHELD GPS.
  3. BOREHOLE ELEVATIONS WERE SURVEYED RELATIVE TO THE EXISTING CUT CROSS IN THE FOOTING WALL OF THE SAND/SALT DOME (EL. 209.957 m).



PROJECT: 121-17876-00 111-2

**METRIC**

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES

Agreement No.: 5011-E-0010  
WO No.: 2011-11031

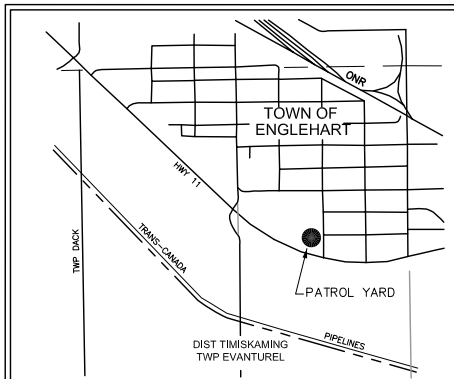
BOREHOLE LOCATION PLAN  
PROPOSED SAND/SALT STORAGE  
FACILITY  
ENGLEHART PATROL YARD  
KING'S HIGHWAY 11

Client: MTO - Northeastern Region



DRAWING

1



KEY PLAN 0 200 m

LEGEND

- Borehole
- Borehole and Cone

BH No	ELEVATION (mASL)	COORDINATES (NAD 83 Zone17)	
		NORTHING	EASTING
12-1	209.465	5296882.3	584290.9
12-2	209.473	5296882.7	584316.5
12-3	209.665	5296909.5	584314.6
12-4	209.671	5296908.8	584293.3

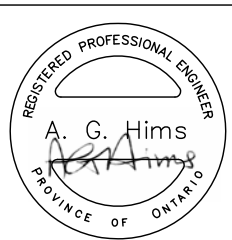
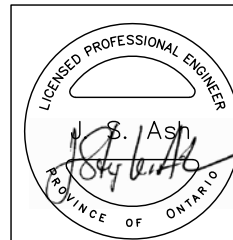
NOTE

THE ACTUAL SOIL STRATIFICATION HAS BEEN VERIFIED FROM DATA OBTAINED AT THE BOREHOLE LOCATIONS ONLY. THE INFERRED CONTACTS SHOWN ARE BASED ON GEOLOGICAL EVIDENCE AND THESE MAY VARY FROM THOSE SHOWN BETWEEN BORINGS.

REVISIONS			
	DATE	BY	DESCRIPTION

GEOCRES No. 31M-95

HWY No. 11				DIST NEW LISKEARD
SUBM'D ---	CHECKED JSH	DATE JUNE 2012	SITE ---	
DRAWN PLB	CHECKED ---	APPROVED ---	DWG ---	



SITE PLAN MAPPING REF. NO.:  
MTO PLAN H-286-11-1, CONT No WP No 2004-50-001, OCT 2005,  
PLATE 286-11/36-0, SHEET 1 OF 1.

**A** Southwest **A'** Northeast

BOREHOLE BH12-1 BOREHOLE BH12-3

Asphalt

GRAVELLY SAND FILL

Compact

Stiff To Firm

Very Stiff To Firm

SILT

CLAYEY SILT

WOH

PH

Stiff

Firm

SILT & CLAY

CONE

ELEVATION (mASL)

210 -

209 -

208 -

207 -

206 -

205 -

204 -

203 -

202 -

201 -

200 -

199 -

198 -

197 -

196 -

195 -

194 -

193 -

192 -

191 -

190 -

189 -

188 -

187 -

186 -

185 -

184 -

183 -

182 -

181 -

180 -

179 -

178 -

177 -

176 -

175 -

174 -

173 -

172 -

171 -

170 -

169 -

168 -

167 -

166 -

0 10 20 30 40 50 60

DISTANCE (m)

15.7

25.9

15.7

43.9

The diagram illustrates a geological cross-section between two boreholes, BH12-4 (Northwest) and BH12-2 (Southeast). The vertical axis represents ELEVATION (mASL) from 184 to 210. The horizontal axis represents DISTANCE (m) from 0 to 60. The soil profile is divided into several layers: GRAVELLY SAND FILL (top, cross-hatched), SILT (middle, vertical lines), CLAYEY SILT (lower middle, diagonal lines), and SILT & CLAY (bottom, diagonal lines). A dashed line indicates a boundary between the SILT and CLAYEY SILT layers. A solid line indicates the ground surface. A dashed line indicates the water table. A dashed line indicates the cone penetrometer test (CONE) result. The borehole logs show soil types and depths: BH12-4 (14, 14, 13, 6, 4, 4, WOH, Stiff, WOH, WOH, WOH, WOH, WOH, Stiff, WOH, Stiff, WOH, 15.7) and BH12-2 (15, 12, 6, 5, 3, PH, WOH, WOH, WOH, Stiff, WOH, CLAYEY SILT, WOH, CONE, 7, 15.7, 25.9). The CONE test result is shown as a vertical line with a scale from 0 to 100.

REVISONS			
	DATE	BY	DESCRIPTION
GEOGRES No. 31M-95			
HWY No 11		DIST NEW LISKEARD	
SUBM'D --	CHECKED JSA	DATE JUNE 2012	SITE --
DRAWN PLB	CHECKED --	APPROVED --	DWG --

SITE PLAN MAPPING REF. NO.:  
MTO PLAN H-286-11-1. CONT No WP No 2004-50-001, OCT 2005,  
PLATE 286-11/36-0, SHEET 1 OF 1.

---

## Appendix A

### Borehole Explanation Forms

### Borehole Logs

---

# BOREHOLE LOG EXPLANATION FORM

This explanatory section provides the background to assist in the use of the borehole logs. Each of the headings used on the borehole log, is briefly explained.

## DEPTH

This column gives the depth of interpreted geologic contacts in metres below ground surface.

## STRATIGRAPHIC DESCRIPTION

This column gives a description of the soil based on a tactile examination of the samples and/or laboratory test results. Each stratum is described according to the following classification and terminology.

<u>Soil Classification*</u>		<u>Terminology</u>	<u>Proportion</u>
Clay	<0.002 mm		
Silt	0.002 to 0.06 mm	"trace" (e.g. trace sand)	<10%
Sand	0.06 to 2 mm	"some" (e.g. some sand)	10% - 20%
Gravel	2 to 60 mm	adjective (e.g. sandy)	20% - 35%
Cobbles	60 to 200 mm	"and" (e.g. and sand)	35% - 50%
Boulders	>200 mm	noun (e.g. sand)	>50%

\* Extension of MIT Classification system unless otherwise noted.

The use of the geologic term "till" implies that both disseminated coarser grained (sand, gravel, cobbles or boulders) particles and finer grained (silt and clay) particles may occur within the described matrix.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

<u>COHESIONLESS SOIL</u>		<u>COHESIVE SOIL</u>		
Compactness	Standard Penetration Resistance "N", Blows / 0.3 m	Consistency	Standard Penetration Resistance "N", Blows / 0.3 m	Undrained Shear Strength (cu) (kPa)
Very Loose	0 to 4	Very Soft	0 to 2	0 to 12
Loose	4 to 10	Soft	2 to 4	12 to 25
Compact	10 to 30	Firm	4 to 8	25 to 50
Dense	30 to 50	Stiff	8 to 15	50 to 100
Very Dense	Over 50	Very Stiff	15 to 30	100 to 200
		Hard	Over 30	Over 200

The moisture conditions of cohesionless and cohesive soils are defined as follows.

### COHESIONLESS SOILS

Dry  
Moist  
Wet  
Saturated

### COHESIVE SOILS

DTPL - Drier Than Plastic Limit  
APL - About Plastic Limit  
WTPL - Wetter Than Plastic Limit  
MWTPL - Much Wetter Than Plastic Limit













## STRATIGRAPHY

Symbols may be used to pictorially identify the interpreted stratigraphy of the soil and rock strata.

## MONITOR DETAILS

This column shows the position and designation of standpipe and/or piezometer ground water monitors installed in the borehole. Also the water level may be shown for the date indicated.

	Standpipe		Geotextile Material / Liner		Granular Backfill
	Piezometer		Borehole Seal (Bentonite Grout)		Granular (Filter) Pack
	Screened Interval		Cement Seal		Native Soil Backfill / Cave / Slough
	Borehole Seal (Peltonite, Bentonite or Hole Plug)				

Where monitors are placed in separate boreholes, these are shown individually in the "Monitor Details" column. Otherwise, monitors are in the same borehole. For further data regarding seals, screens, etc., the reader is referred to the summary of monitor details table.

## SAMPLE

These columns describe the sample type and number, the "N" value, the water content, the percentage recovery, and Rock Quality Designation (RQD), of each sample obtained from the borehole where applicable. The information is recorded at the approximate depth at which the sample was obtained. The legend for sample type is explained below.

SS = Split Spoon	GS = Grab Sample
TW = Thin Walled Shelby Tube	CS = Channel Sample
AS = Auger Flight Sample	WS = Wash Sample
CC = Continuous Core	RC = Rock Core
PH = TW Advanced Hydraulically	

$$\% \text{ Recovery} = \frac{\text{Length of Core Recovered Per Run}}{\text{Total Length of Run}} \times 100$$

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

### RQD Classification

### RQD (%)

Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

## **TEST DATA**

The central section of the log provides graphs which are used to plot selected field and laboratory test results at the depth at which they were carried out. The plotting scales are shown at the head of the column.

Dynamic Penetration Resistance - The number of blows required to advance a 51 mm diameter, 60° steel cone fitted to the end of 45 mm OD drill rods, 0.3 m into the subsoil. The cone is driven with a 63.5 kg hammer over a fall of 750 mm.

Standard Penetration Resistance - Standard Penetration Test (SPT) "N" Value - The number of blows required to advance a 51 mm diameter standard split-spoon sampler 300 mm into the subsoil, driven by means of a 63.5 kg hammer falling freely a distance of 750 mm. In cases where the split spoon does not penetrate 300 mm, the number of blows over the distance of actual penetration in millimetres is shown as  $\frac{x\text{Blows}}{\text{mm}}$

Water Content - The ratio of the mass of water to the mass of oven-dry solids in the soil expressed as a percentage.

W<sub>P</sub> - Plastic Limit of a fine-grained soil expressed as a percentage as determined from the Atterberg Limit Test.

W<sub>L</sub> - Liquid Limit of a fine-grained soil expressed as a percentage as determined from the Atterberg Limit Test.

## **REMARKS**

The last column describes pertinent drilling details, field observations and/or provides an indication of other field or laboratory tests that were performed.

# RECORD OF BOREHOLE No BH12-1

1 OF 2

METRIC

LOCATION ENGLEHART PATROL YARD N 5 296 882.3; E 584 290.9






































ORIGINATED BY DCL

BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT

COMPILED BY JW

DATUM GEODETTIC DATE 5.28.12 - 5.28.12

CHECKED BY RK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W <sub>P</sub>	W	W <sub>L</sub>		GR	SA	SI	CL
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE			WATER CONTENT (%)							
209.5							20	40	60	80	100								
208.9	ASPHALT: 75 mm THICK		1	SS	16		20	40	60	80	100	○							
208.8	GRANULAR FILL: GRAVELLY SAND TO SAND SOME GRAVEL FILL BROWN, COMPACT, MOIST TO WET		2	SS	15								○						
0.7	SILT: SILT, SOME CLAY, SOME FINE SAND BROWN TO GREY, VERY STIFF TO FIRM, WTP		3	SS	24									○					
			4	SS	3										○			0 16 70 14	
			5	SS	4										○				
			6	SS	4										○				
			7	SS	3										H ○			0 10 76 14	
			8	SS	WOH														
																			
																			
																			
																			
202.2	CLAYEY SILT: CLAYEY SILT TRACE FINE SAND GREY, FIRM TO STIFF, WTP		9	SS	3														
7.3																			
																			
																			
			10	SS	WOH									H H			0 2 73 25		
																			
																			
			11	SS	WOH														
																			
																			
			12	SS	WOH									H H			0 1 69 30		
																			
																			
			13	TW	PH														
																			
																			
																			
																			
																			
																			
																			
																			
																			
																			

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

**METRIC**

ORIGINATED BY DCL

COMPILED BY JW

CHECKED BY           RK          

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH12-2

1 OF 2

METRIC

LOCATION ENGLEHART PATROL YARD N 5 296 882.7; E 584 316.5

ORIGINATED BY DCL

BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT

COMPILED BY JW

DATUM GEODETIC DATE 5.29.12 - 5.29.12

CHECKED BY RK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					GR	SA	SI
209.5								20	40	60	80	100								
209.4	ASPHALT: 65 mm THICK		1	SS	15		209											45	48	(7)
	GRANULAR FILL: GRAVEL AND SAND TO SAND SOME GRAVEL, TRACE SILT BROWN, COMPACT, MOIST TO WET		2	SS	12															
208.1							208													
1.4	SILT: MOTTLED SILT, SOME SAND, TRACE CLAY, TRACE GRAVEL CHANGING TO SILT SOME CLAY TRACE SAND AT 2.9 m DEPTH BROWNISH GREY TO GREY, FIRM, WET		3	SS	12													1	9	78 12
			4	SS	6		207													
			5	SS	5		206													
							205													
			6	SS	3		204											0	1	84 15
			7	TW	PH		203													
202.5							202													
7.0	CLAYEY SILT: CLAYEY SILT, TRACE SAND, DILATANT LAYERS GREY, STIFF, WTP		8	SS	WOH		201													
			9	SS	WOH		200													
							199													
			10	SS	WOH		198											0	1	78 21
			11	SS	WOH		197													
							196													
			12	SS	WOH		195													

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH12-2

2 OF 2

METRIC

LOCATION ENGLEHART PATROL YARD N 5 296 882.7; E 584 316.5

ORIGINATED BY DCL

BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT

COMPILED BY JW

DATUM GEODETIC DATE 5.29.12 - 5.29.12

CHECKED BY RK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W <sub>p</sub>	W	W <sub>L</sub>		
							20	40	60	80	100						
193.8			13	SS	7		194										
15.7	CONTINUOUS DYNAMIC CONE PENETRATION TEST BELOW 15.7 m DEPTH. NO SOIL SAMPLING COMPLETED.						193										
							192										
							191										
							190										
							189										
							188										
							187										
							186										
							185										
							184										
183.6	END OF BOREHOLE																
25.9																	

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH12-3

1 OF 3

METRIC

LOCATION ENGLEHART PATROL YARD N 5 296 909.5; E 584 314.6

ORIGINATED BY DCL

BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT

COMPILED BY JW

DATUM GEODETIC DATE 5.29.12 - 5.30.12

CHECKED BY RK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)							
								○ UNCONFINED + FIELD VANE								○							
								● QUICK TRIAXIAL × LAB VANE															
209.7									20	40	60	80	100										
208.6	ASPHALT: 90 mm THICK		1	SS	16																		
	GRANULAR FILL: GRAVELLY SAND TO SAND, SOME GRAVEL FILL BROWN, COMPACT, MOIST TO WET		2	SS	13																		
			3	SS	13																		
207.6																							
2.1	SILT: SILT, SOME FINE SAND, TRACE CLAY, DILATANT, LOW PLASTICITY GREY, STIFF TO FIRM, WTPL		4	SS	10											0 17 75 8							
			5	SS	4																		
			6	SS	3																		
			7	SS	3																		
204.0																							
5.7	CLAYEY SILT: CLAYEY SILT, TRACE SAND GREY, FIRM, WTPL		8	SS	WOH											4 6 70 20							
	OCCASIONAL LAYERS OF SILT, SOME CLAY, NO DISCERNABLE STRUCTURE		9	SS	WOH																		
			10	TW	PH																		
			11	SS	WOH											0 1 76 23							
	EVIDENCE OF VARVES / LAMINATION		12	SS	WOH																		
			13	SS	WOH																		
197.0																							
12.7	SILT AND CLAY: SILT AND CLAY, TRACE SAND GREY, FIRM, WTPL																						
	</																						

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

**METRIC**

ORIGINATED BY DCL

COMPILED BY JW

CHECKED BY           RK          

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



**METRIC**

ORIGINATED BY DCL

COMPILED BY JW

CHECKED BY                      RK

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH12-4

1 OF 2

METRIC

LOCATION ENGLEHART PATROL YARD N 5 296 908.8; E 584 293.3

ORIGINATED BY DCL

BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT

COMPILED BY JW

DATUM GEODETIC DATE 5.30.12 - 5.30.12

CHECKED BY RK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	● QUICK TRIAXIAL	+	×	FIELD VANE						LAB VANE	W <sub>P</sub>	W
209.7							20	40	60	80	100	10	20	30	kN/m <sup>3</sup>	GR	SA	SI	CL	
209.0	ASPHALT: 100 mm THICK		1	SS	14															
	GRANULAR FILL: GRAVELLY SAND TO SAND, SOME GRAVEL FILL BROWN, COMPACT, MOIST TO WET		2	SS	14															
			3	SS	13															
207.7																				
2.0	SILT: SILT, SOME CLAY, TRACE FINE SAND, DILATANT BROWN TO GREY, FIRM, WTPL		4	SS	6															
			5	SS	4															
206.0																				
3.7	CLAYEY SILT: CLAYEY SILT GREY, STIFF, WTPL		6	SS	4															
			7	TW	PH															
	VARVED, DILATANT LAYERS		8	SS	WOH															
			9	SS	WOH															
			10	SS	WOH															
	LAYERED / LAMINATED (VARVED)		11	SS	WOH															
			12	SS	WOH															
197.0																				
12.7	SILT AND CLAY: SILT AND CLAY, TRACE SAND GREY, STIFF, WTPL																			
			13	SS	WOH															

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH12-4

2 OF 2

METRIC

LOCATION ENGLEHART PATROL YARD N 5 296 908.8; E 584 293.3

ORIGINATED BY DCL

BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT

COMPILED BY JW

DATUM GEODETIC DATE 5.30.12 - 5.30.12

CHECKED BY RK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W <sub>p</sub> W W <sub>L</sub> 10 20 30						
194.0	<u>SILT AND CLAY:</u> SILT AND CLAY, TRACE SAND GREY, STIFF, WTPL ( <i>continued</i> )		14	SS	WOH												
15.7	END OF BOREHOLE  FIELD VANE TEST COMPLETED AT 16.2 m DEPTH.																

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

---

## Appendix B

Summary of Particle Size Distribution  
Results (Table B1)

Particle Size Distribution Analyses  
(Figures B1 to B4)

Plasticity Chart  
(Figures B5 to B7)

Consolidation Test Results  
(Figures B8 to B10)

---

**Table B1: Summary of Grain Size Distribution and Hydrometer Tests**

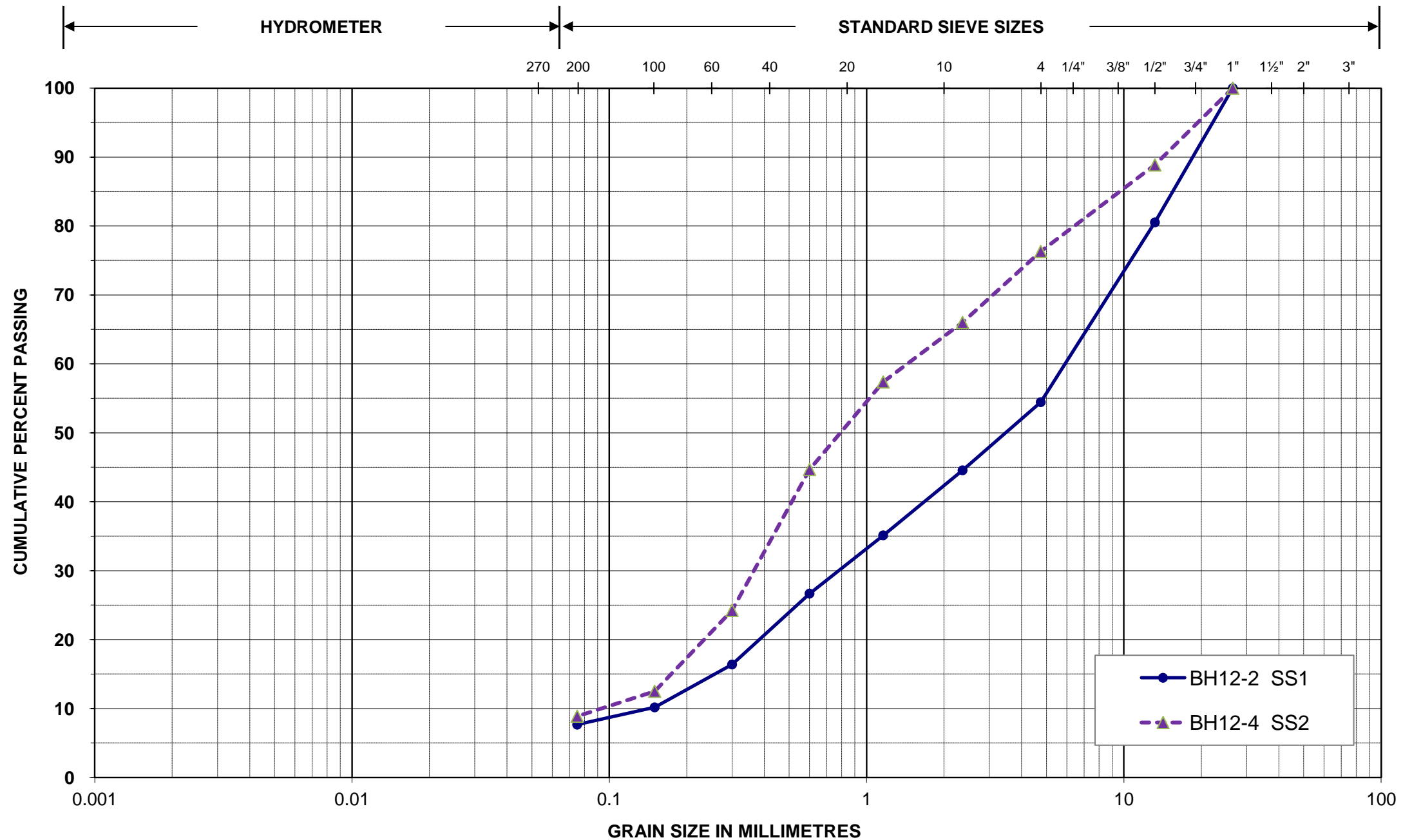
Borehole No.	Sample ID	Soil Description	Percentage Retained (%)			
			Gravel	Sand	Silt	Clay
BH12-1	SS4	Silt, some sand, some clay	0	16	70	14
BH12-1	SS7	Silt, some sand, some clay	0	10	76	14
BH12-1	SS10	Clayey silt, some sand	0	2	73	25
BH12-1	SS12	Clayey silt, trace sand	0	1	69	30
BH12-2	SS1	Sand and gravel, trace silt	45	48	7	
BH12-2	SS3	Silt, some clay, trace sand, trace gravel	1	9	78	12
BH12-2	SS6	Silt, some clay, trace sand	0	1	84	15
BH12-2	SS10	Clayey silt, trace sand	0	1	78	21
BH12-3	SS4	Silt, some sand, trace clay	0	17	75	8
BH12-3	SS8	Clayey silt, trace sand, trace gravel	4	6	70	20
BH12-3	SS11	Clayey silt, trace sand	0	1	76	23
BH12-3	SS14	Silt and clay, trace sand	0	1	57	42
BH12-4	SS2	Gravelly sand, trace silt	24	67	9	
BH12-4	SS5	Silt, some clay, trace sand	0	5	81	14
BH12-4	SS9	Clayey silt, trace sand	0	4	76	20
BH12-4	SS13	Silt and clay	0	0	63	37

**Terminology**                      **Proportion**

“trace” (e.g. trace sand)	< 10%
“some” (e.g. some sand)	10% to 20%
adjective (e.g. sandy)	20% to 35%
“and” (e.g. and sand)	35% to 50%
Noun (e.g. sand)	> 50%



# PARTICLE SIZE DISTRIBUTION



Unified Classification System

SILT AND CLAY	SAND	GRAVEL
---------------	------	--------

**Project Name:** MTO Agreement # 5011-E-0010 (Englehart)

**Project No.:** 121-17876-00

**Figure No.:** B1

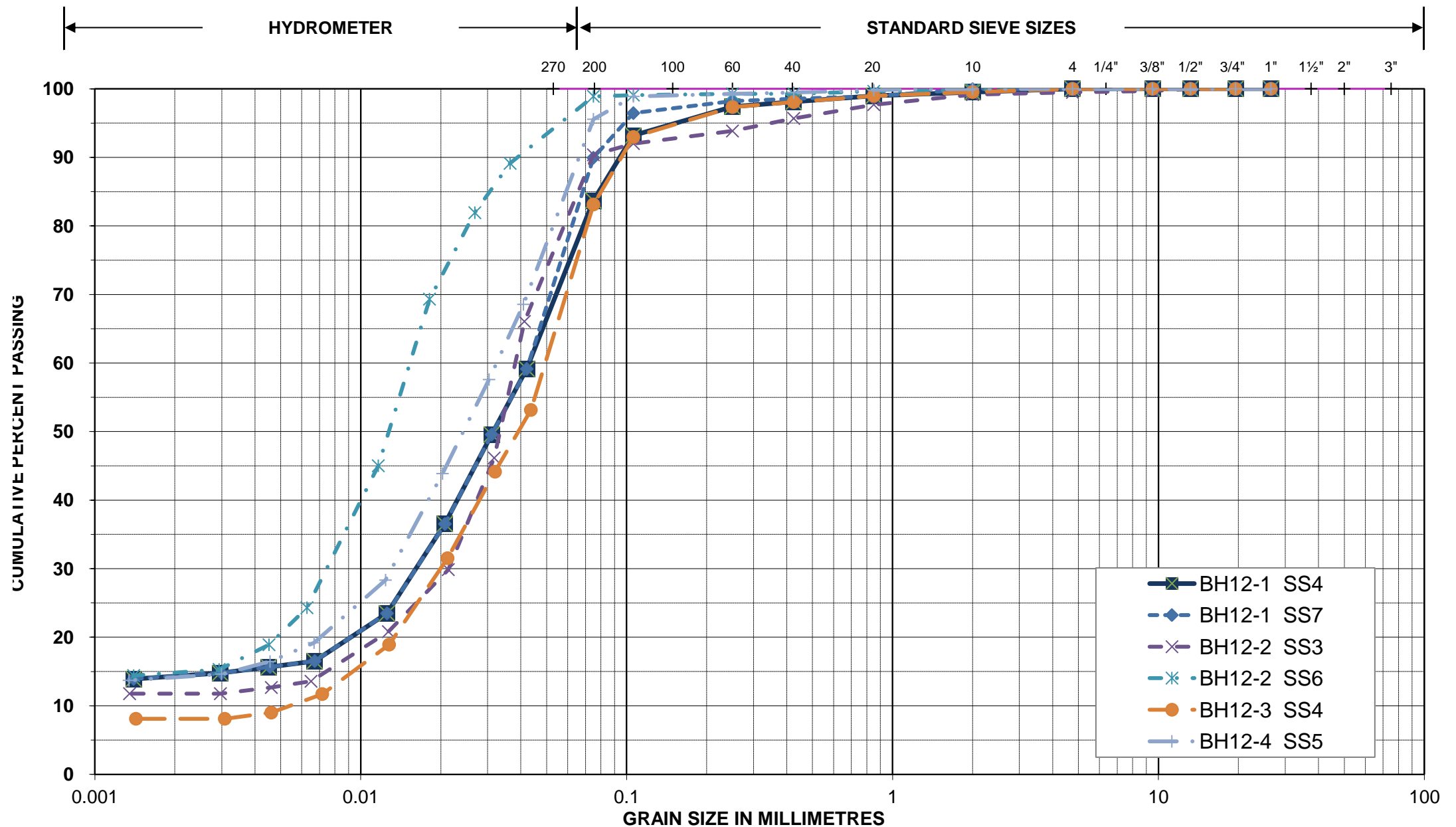
**Remarks:** Sand and gravel, trace silt



GENIVAR

# PARTICLE SIZE DISTRIBUTION

ASTM D422



Unified Classification System

SILT AND CLAY

SAND

GRAVEL

**Project Name:** MTO Agreement # 5011-E-0010 (Englehart)

**Project No.:** 121-17876-00

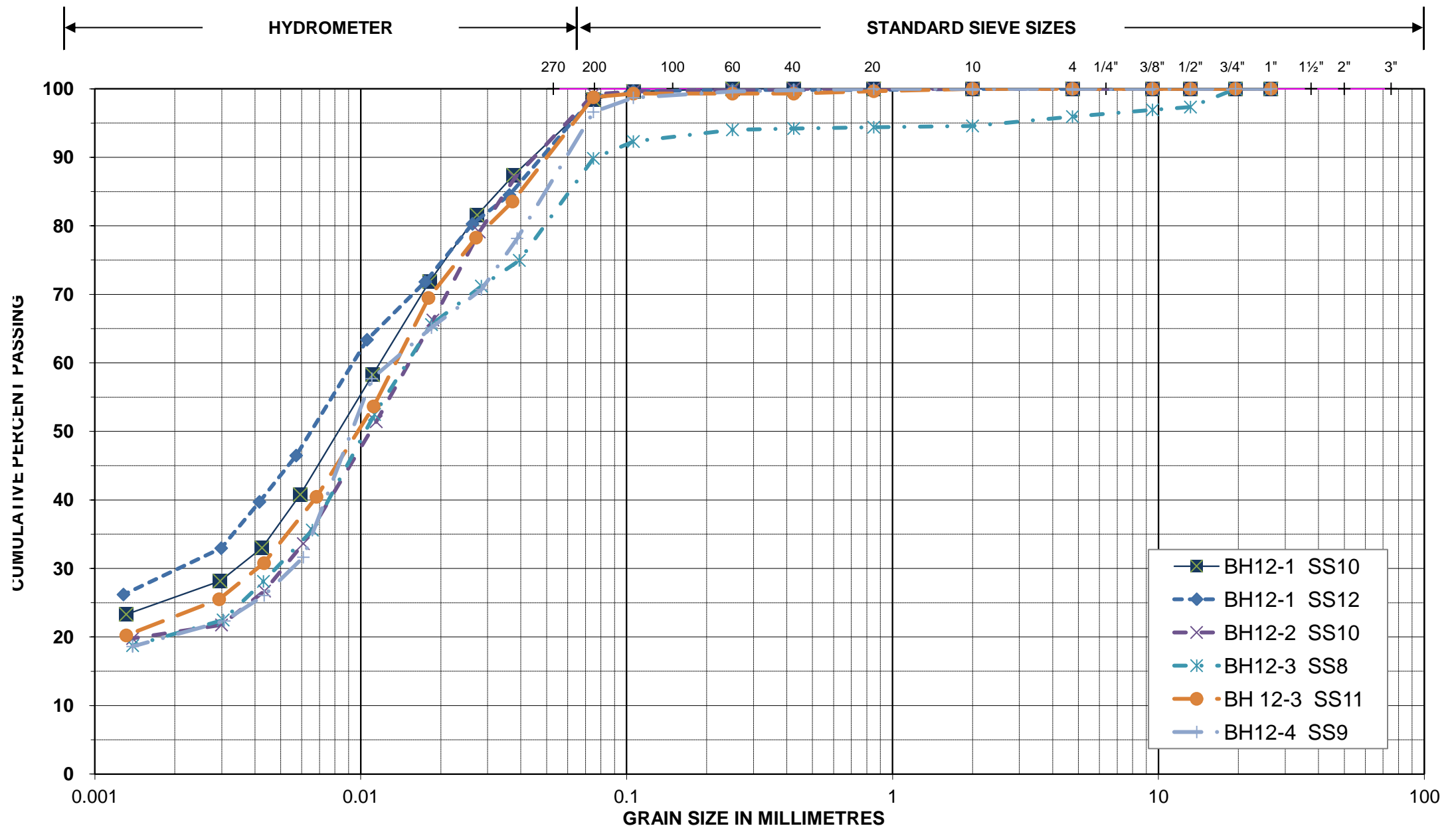
**Figure No.:** B2

**Remarks:** Silt, some clay, trace to some sand



GENIVAR

# PARTICLE SIZE DISTRIBUTION ASTM D422



**Project Name:** MTO Agreement # 5011-E-0010 (Englehart)

**Project No.:** 121-17876-00

**Figure No.:** B3

**Remarks:** Clayey silt, trace sand

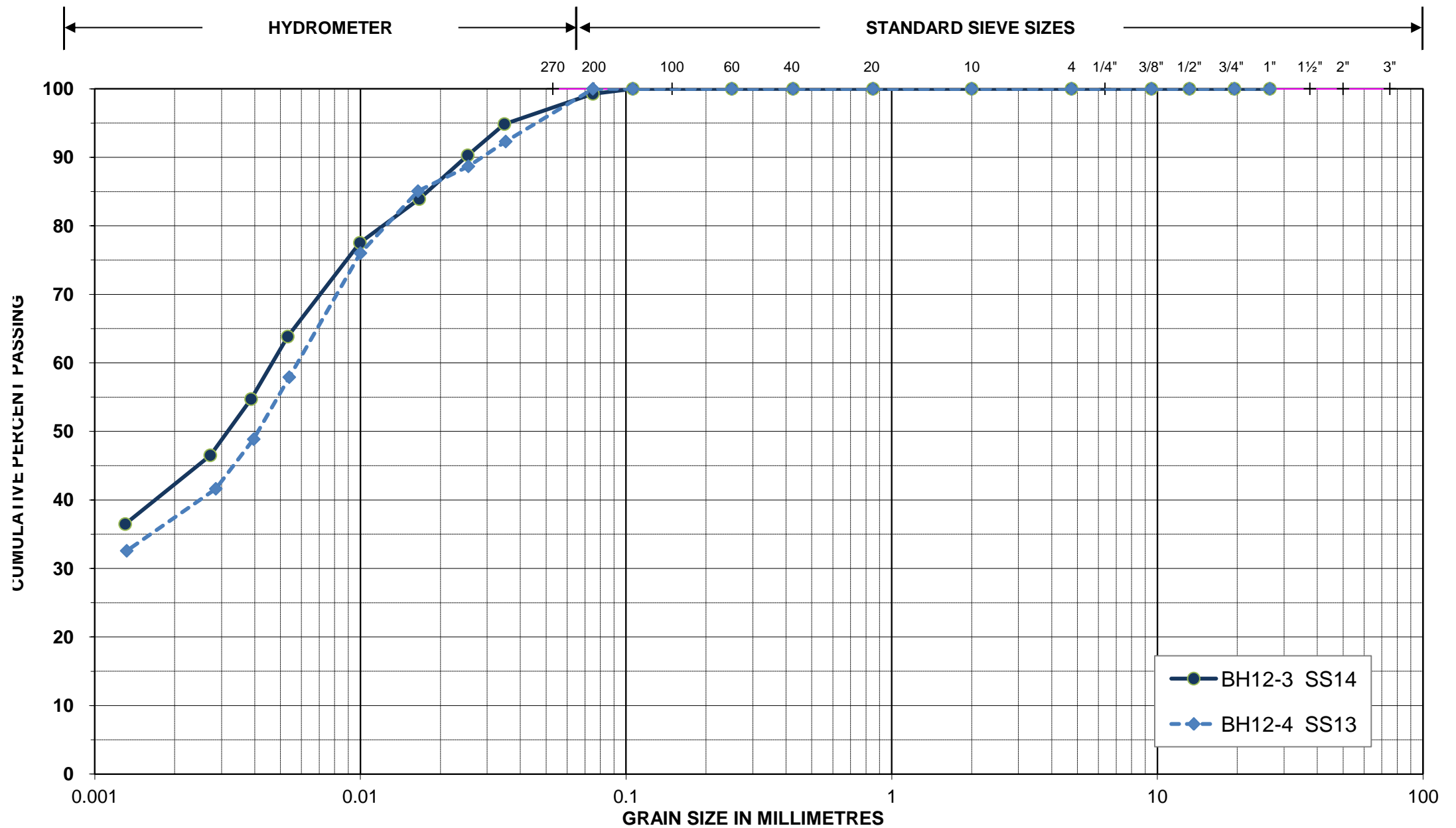




GENIVAR

# PARTICLE SIZE DISTRIBUTION

ASTM D422



Unified Classification System

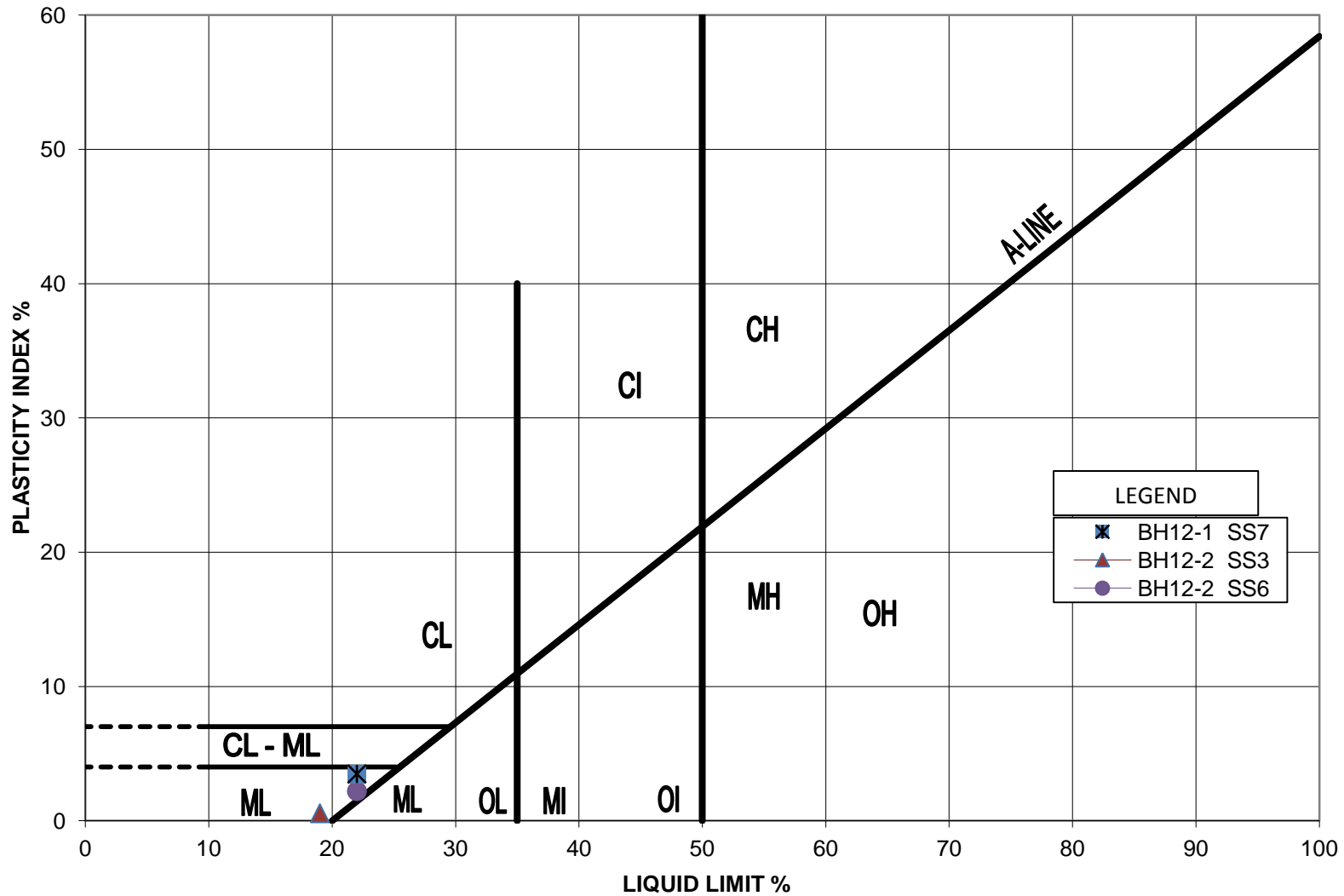
SILT AND CLAY	SAND	GRAVEL
---------------	------	--------

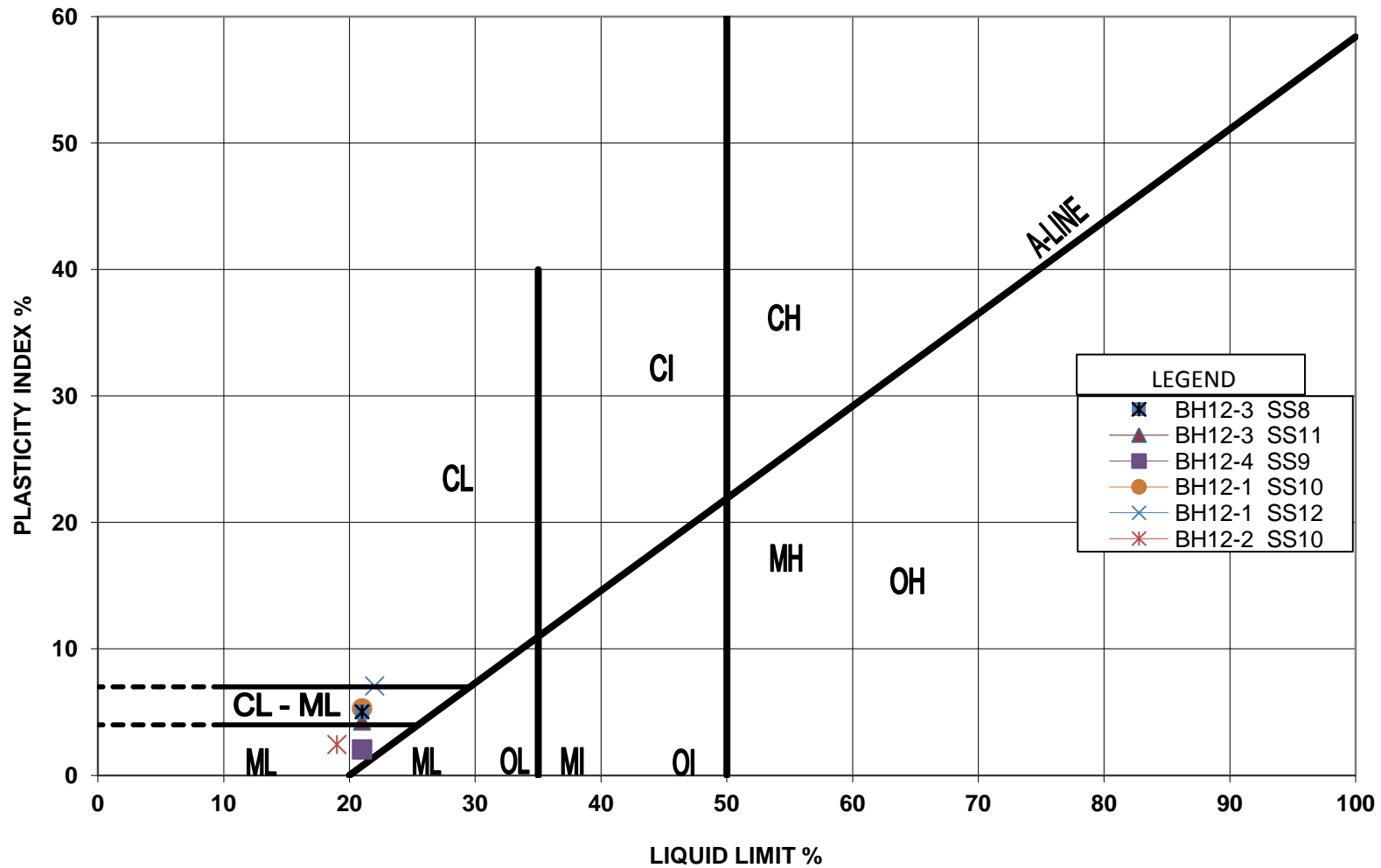
Project Name: MTO Agreement # 5011-E-0010 (Englehart)

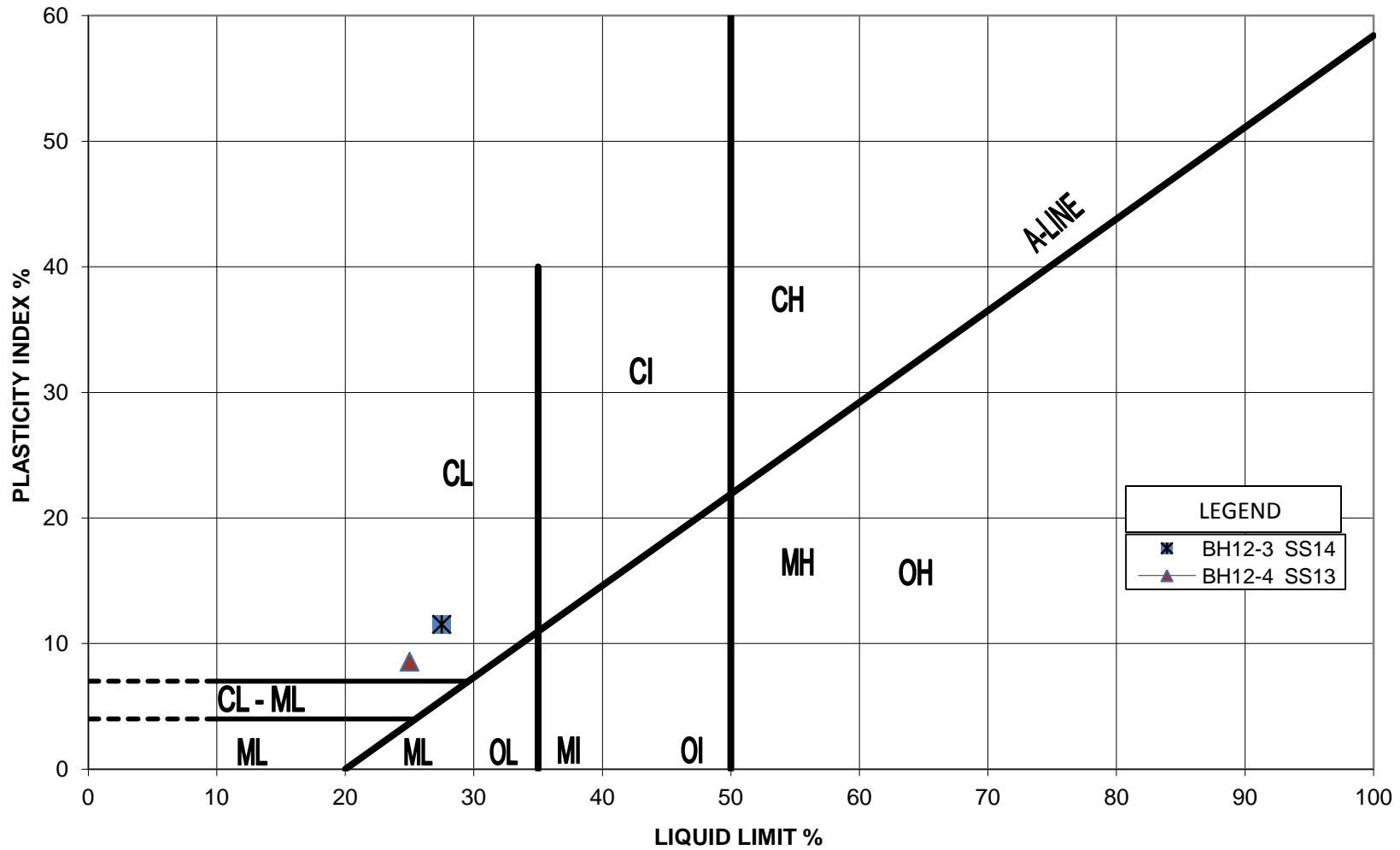
Project No.: 121-17876-00

Figure No.: B4

Remarks: Silt and clay







## PLASTICITY CHART SILT AND CLAY

**CONSOLIDATION TEST SUMMARY****FIGURE B8****SAMPLE IDENTIFICATION**

Project Number	12-1183-0067	Sample Number	S10
Borehole Number	12-3	Sample Depth, m	8.4-9.0

**TEST CONDITIONS**

Test Type	Standard	Load Duration, hr	24
Oedometer Number	1		
Date Started	6/07/2012		
Date Completed	6/14/2012		

**SAMPLE DIMENSIONS AND PROPERTIES - INITIAL**

Sample Height, cm	2.54	Unit Weight, kN/m <sup>3</sup>	18.79
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	14.39
Area, cm <sup>2</sup>	31.61	Specific Gravity, measured	2.75
Volume, cm <sup>3</sup>	80.22	Solids Height, cm	1.354
Water Content, %	30.52	Volume of Solids, cm <sup>3</sup>	42.81
Wet Mass, g	153.68	Volume of Voids, cm <sup>3</sup>	37.41
Dry Mass, g	117.74	Degree of Saturation, %	96.1

**TEST COMPUTATIONS**

Axial Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	c <sub>v</sub> cm <sup>2</sup> /s	m <sub>v</sub> m <sup>2</sup> /kN	k cm/s
0.00	2.538	0.874	2.538				
25.41	2.508	0.852	2.523	107	1.26E-02	4.59E-04	5.67E-07
51.75	2.495	0.842	2.502	282	4.71E-03	1.94E-04	8.97E-08
100.12	2.468	0.822	2.482	205	6.37E-03	2.25E-04	1.40E-07
198.79	2.416	0.783	2.442	623	2.03E-03	2.08E-04	4.14E-08
399.40	2.308	0.704	2.362	185	6.39E-03	2.12E-04	1.33E-07
796.28	2.236	0.650	2.272	101	1.08E-02	7.18E-05	7.62E-08
1590.81	2.169	0.601	2.202	126	8.16E-03	3.30E-05	2.64E-08
796.28	2.174	0.605	2.171				
399.40	2.184	0.612	2.179				
198.79	2.191	0.617	2.187				
25.37	2.217	0.636	2.204				

Note:

k calculated using cv based on t<sub>90</sub> values.

Consolidation loading and unloading schedule assigned by the client.

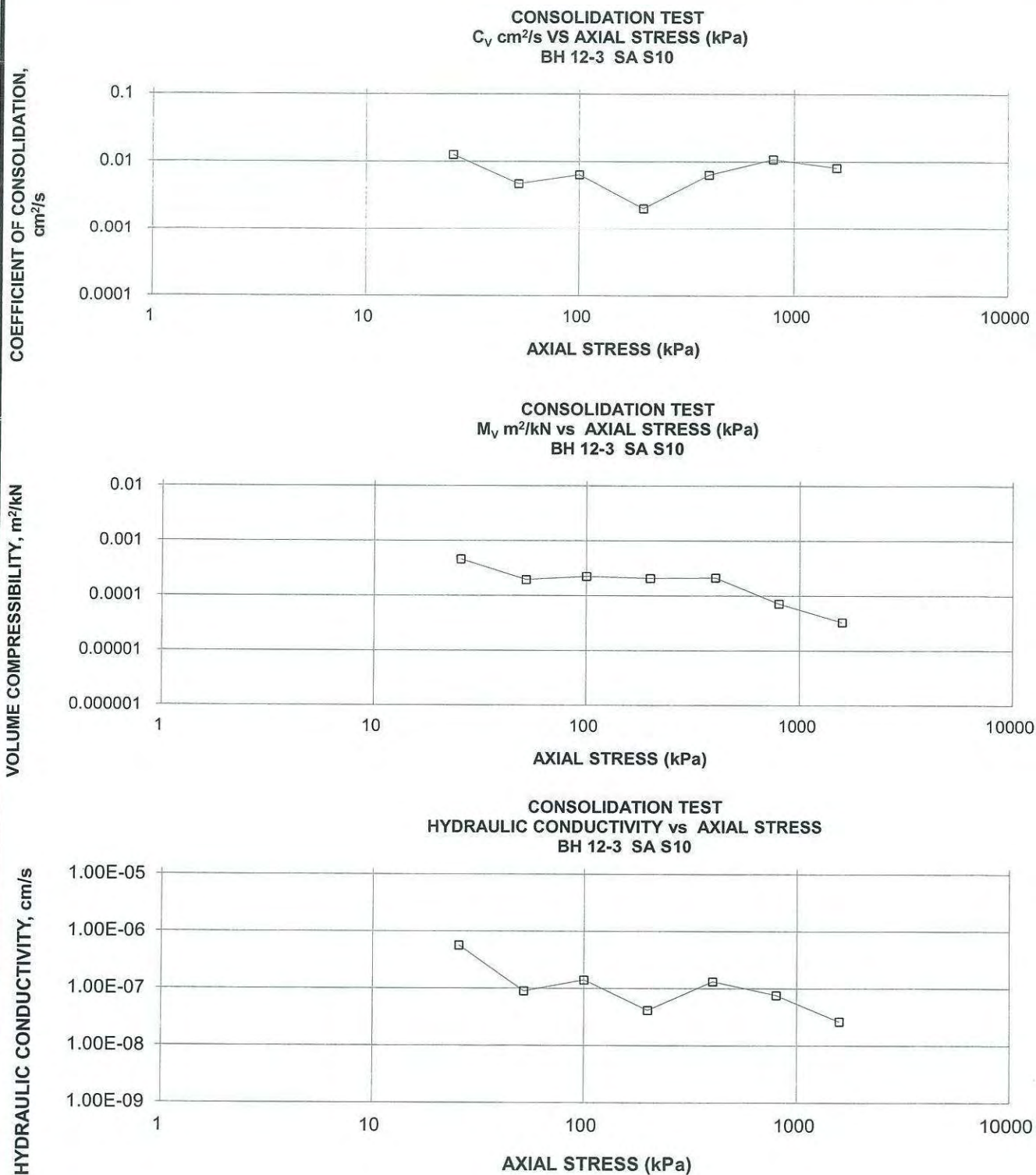
Specimen taken 12cm from bottom of the tube.

**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.22	Unit Weight, kN/m <sup>3</sup>	20.36
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m <sup>3</sup>	16.48
Area, cm <sup>2</sup>	31.61	Specific Gravity, measured	2.75
Volume, cm <sup>3</sup>	70.06	Solids Height, cm	1.354
Water Content, %	23.53	Volume of Solids, cm <sup>3</sup>	42.81
Wet Mass, g	145.44	Volume of Voids, cm <sup>3</sup>	27.25
Dry Mass, g	117.74		

# CONSOLIDATION TEST SUMMARY

FIGURE B9



Project No. 12-1183-0067

Prepared By: LFG

**Golder Associates**

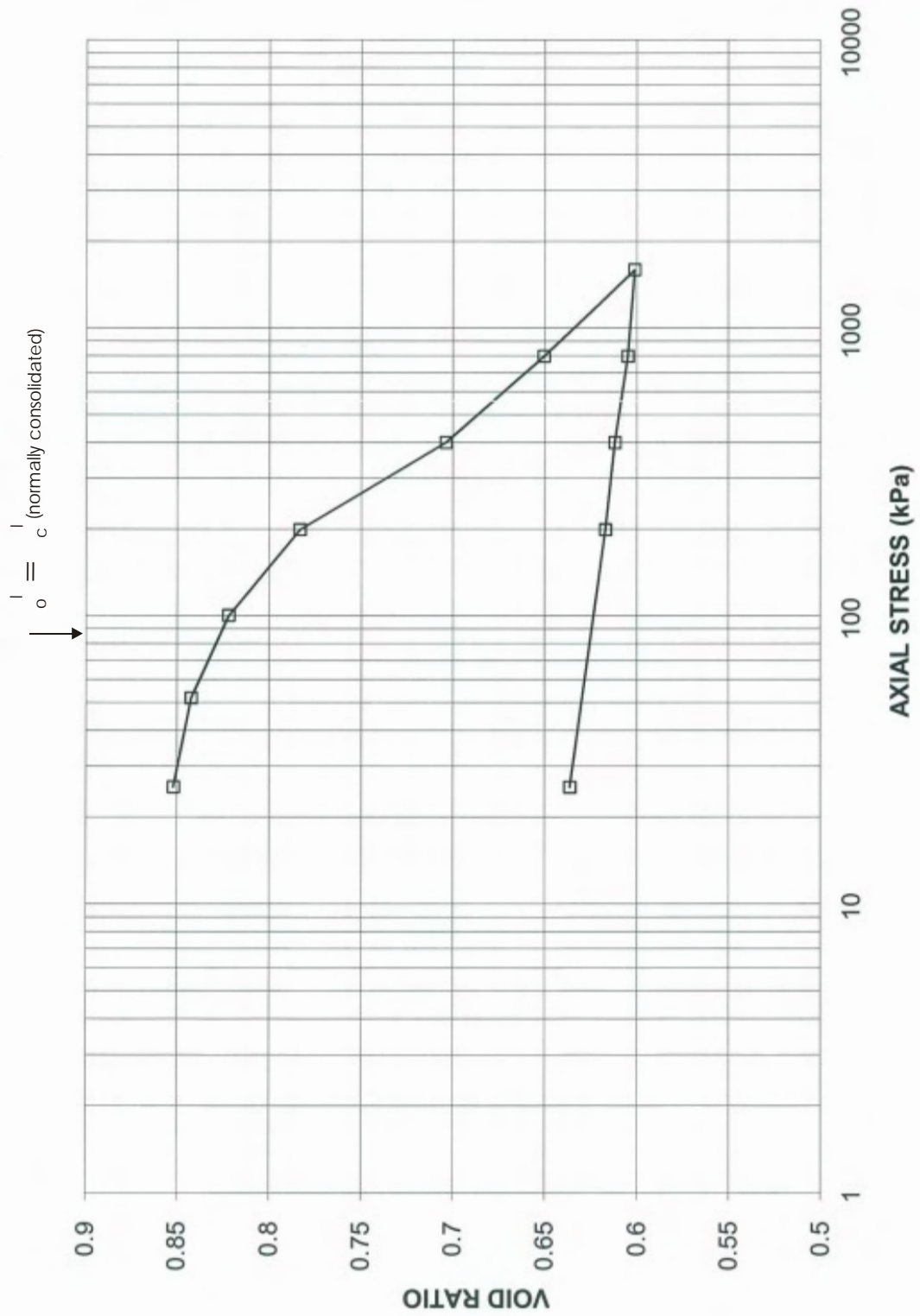
Checked By: *[Signature]*



CONSOLIDATION TEST  
VOID RATIO VS LOG AXIAL STRESS

FIGURE B10

CONSOLIDATION TEST  
VOID RATIO vs AXIAL STRESS  
BH 12-3 SA S10



---

## Appendix C

### Site Photographs

---



**MTO AGREEMENT #5011-E-0010  
ENGLEHART PATROL YARD**



Photograph 1: East side of 6-bay garage. Looking north.



Photograph 2: Existing sand/salt domes, dome on the right to be removed. Looking northwest.

**MTO AGREEMENT #5011-E-0010  
ENGLEHART PATROL YARD**



Photograph 3: Existing shed and above ground diesel storage tank. Looking east.



Photograph 4: Eastern-most sand/salt dome. Location of proposed structure. Looking north.



**MTO AGREEMENT #5011-E-0010  
ENGLEHART PATROL YARD**



Photograph 5: North side of existing domes. Looking east.



Photograph 6: Northern area of the site. Looking northwest.