MTO Agreement No. 5011-E-0010
WO No. 2011-11035
Proposed Sand/Salt Storage Facility
Detour Patrol Yard
Foundation Investigation and
Design Report

Geocres No. 42H-52

June 2013

Prepared for:
Ontario Ministry of Transportation
Northeastern Region
447 McKeown Avenue
North Bay, Ontario
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Project No. 121-17876-00

GENIVAR

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June 5, 2013

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.: 2011-11035
Proposed Sand/Salt Storage Facility Detour Patrol Yard
Foundation Investigation and Design Report (Geocres No. 42H-52)

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) Detour Patrol Yard in the Township of Tweed, 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.

J. Stephen Ash, P. Eng., P. Geo.
Director, Environment

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1. Introduction

GENIVAR Inc. (GENIVAR) was retained by the Ontario Ministry of Transportation Northeastern Region (MTO) to undertake a foundation investigation for the proposed construction of a sand/salt storage facility at the Detour Patrol Yard, located on Michel Lake Road approximately 150 m north from its intersection with Highway 652, in the Township of Tweed, 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 Detour 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 Detour Patrol Yard (site) is located on Michel Lake Road approximately 150 m north from its intersection with Highway 652, in the Township of Tweed, Ontario. A Site Plan is included as Drawing 1 and colour photographs of the site are included in Appendix C.

The site is sloped to the middle of the property, and then to a dry drainage ditch that slopes to the west. Access to the site is from Highway 652, and surrounding land uses are rural and uninhabited, with a low lying swampy area to the northeast. The site is surrounded with mixed deciduous and coniferous forest. No bedrock outcrops were visible on the site or immediate surrounding area.

The site is an operational MTO Patrol Yard, and currently contains the following structures:

- 4-bay garage;
- 1 large sand dome; and
- 1 well (dry).

The perimeter of the site is grassed, and there is a paved driveway from Highway 652 to the garage and extending to the sand dome.

2.2 Regional Geology

Two different map sources were consulted to determine the regional geology in the Detour area: i) Geology and Principal Minerals Map of Ontario published by the Ontario Department of Mines, and ii) Miscellaneous Data Release 160 of 'Northern Ontario Engineering Geology Terrain Study Data Base Map' published by the Ministry of Natural Resources (MNR).

Based on the mapping information, the site is located within a ground moraine, adjacent to an esker landform. Local soil deposits are comprised of peat and organic terrain underlain by stony till material, with sand and gravel material present within the eskers.

The glaciolacustrine sediments are underlain by Archean mafic to intermediate metavolcanic rocks. 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 Detour Patrol yard was obtained from the MTO Geocres Library in Downsview, Ontario. This patrol yard was the subject of a geotechnical investigation in 1981 when the site was first proposed as a bridge crossing over the South Floodwood River. The results of the geotechnical investigation are summarized in a technical letter, dated April 24, 1981, titled *"Foundation Investigation Report for Detour Lake Access Road Line 'A' – South Floodwood River Structure"* (Geocres 42H-18).

The geotechnical investigation consisted of sampling two (2) boreholes supplemented by the same number of dynamic cone penetration tests (DCPT). The soil stratigraphy at the site was found to be quite uniform and consisted of approximately 2.6 metres to 4.3 metres (m) of peat underlain by sandy silts and a granular till zone. Within the sandy silt layer, SPT N values ranged from 6 to 39 blows per 30 centimetres (cm), while SPT N values for the granular till zone exceeded 100 blows per 100 cm. No bedrock was noted to be encountered in the original investigation. The groundwater table was reported to be close to the ground surface at an elevation of 500 metres above sea level (mASL).

4. Investigation Procedures

4.1 Subsurface Investigation

A borehole investigation was performed at the subject site between June 19 and June 20, 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 the perimeter of the proposed 37.8 m diameter storage dome, as required by the Terms of Reference for the assignment.

MTO minimum requirements for the borehole investigation list 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 boreholes BH12-1 to BH12-4, augering was terminated at a depth of 15.9 m, in firm to stiff silt. Below the final depth of augering, Dynamic Cone Penetration Tests (DCPTs) were driven to refusal, which occurred at depth of 21.0 m to 22.0 m below ground surface.

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 converted to MTO standard coordinates (Northings and Eastings). Borehole elevations were surveyed to a temporary benchmark; a steel pin anchored in the existing pavement located just north of the proposed structure was assigned a relative elevation of 100.00 metres. Borehole elevations, coordinates and benchmark location are shown on Drawing 1. Borehole logs are included in Appendix A.

Drilling and soil sampling were 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 kilogram drop hammer falling 750 mm (ASTM D1586 procedure). Refusal depth for the purposes of this investigation is 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 a depth of 5.0 m, and at 1.5 m intervals thereafter to the termination depth, which was less than 20 m, 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 labeled 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 were taken with 70 mm diameter thin-walled Shelby tubes, which were pushed into the bottom of the borehole using the hydraulic ram of the drill rig. The Shelby tube samples were preserved for transport and storage, inspection and laboratory testing. In situ undrained shear strength (c_u) of the soil was measured using an ASTM tapered field vane and standardized procedures.

DCPTs were completed below a depth of 15.9 m in boreholes BH12-1 to BH12-4. 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. All boreholes were backfilled with drill cuttings mixed with bentonite hole plug and completed with the drill rig. The top portion of the boreholes was sealed with emulsified asphalt. 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 Relative Elevations

Borehole No.	Drilling Depth Below Existing Ground Surface (mbgs)/ Relative Elevation* (m)	Dynamic Cone Penetration Test Depth (m)
BH12-1	15.9/ 84.7	15.9 to 22.3
BH12-2	15.9/ 84.6	15.9 to 22.3
BH12-3	15.9/ 84.9	15.9 to 21.0
BH12-4	15.9/ 84.9	15.9 to 21.0

*Relative to temporary benchmark (see Drawing 1)

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 – Detour Patrol Yard

Test	ASTM Standard	Number of Samples
Natural Moisture Content	ASTM D2216	48
Particle Size Analysis	ASTM D422	13
Atterberg Limits	ASTM D4318	7

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. Laboratory testing results are presented on the borehole logs and 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 in Drawing 1 while the soil strata are provided in two cross sections presented on Drawing 2. Detailed borehole logs are provided in Appendix A, and laboratory test results with the summary tables are included in Appendix B.

5.1 Soil Profile Summary

The boreholes encountered a thin layer of asphalt overlying loose to compact to loose granular fill. Very stiff to firm clayey sandy silt was encountered beneath the fill layer, which in turn was underlain by firm to very stiff clays and silts extending to the borehole termination depths of 15.9 m below ground surface (mbgs). DCPTs were advanced 21.0 mbgs to 22.3 mbgs and results are described in Section 5.1.6. Descriptions of the individual soil units are provided in the following subsections.

5.1.1 Asphalt Pavement

A 40 mm to 50 mm thick surficial layer of asphaltic concrete (hot laid mix) was encountered at the surface of boreholes BH12-1, BH12-2 and BH12-4.

5.1.2 Granular Fill

At the surface of borehole BH12-3 and below the asphalt pavement in boreholes BH12-1, BH12-2 and BH12-4, a granular fill layer (pavement base/subbase) was encountered, consisting of 0.15 m to 0.20 m of gravelly sand, underlain by sand to silty sand with traces of gravel. This layer extended to the depths and relative elevations shown below:

Borehole No.	Depth to Bottom of Fill Layer, mbgs (Relative Elevation, m)
BH12-1	1.5 (99.1)
BH12-2	1.5 (99.0)
BH12-3	1.5 (99.3)
BH12-4	1.2 (99.6)

Laboratory particle size distribution analysis for one (1) sample from the fill layer was 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) - 4 %
- Sand (0.075 mm to 4.75 mm size) - 87 %
- Silt and Clay (less than 0.075 mm size) - 9 %

Standard Penetration Test (SPT) results (N Values) recorded in the fill layer ranged between 8 and 12 blows per 305 mm of penetration, indicating loose to compact to very dense relative density.

Laboratory determined moisture contents ranged between 3 % and 12 % for samples of the fill, indicating moist material.

5.1.3 Clayey Sandy Silt

Clayey sandy silt with traces of fine gravel was encountered below the fill layer and extending to depths (mbgs) and elevations (relative) shown below:

Borehole No.	Depth to Bottom of Clayey Sandy Silt Layer, mbgs (Relative Elevation, m)
BH12-1	7.5 (93.1)
BH12-2	7.5 (93.0)
BH12-3	6.5 (94.3)
BH12-4	6.5 (94.3)

Thus, the thickness of the clayey sandy silt layer varied from 6.0 m at boreholes BH12-1 and BH12-2 to 5.0 m at borehole BH12-3 to 5.3 m at borehole BH12-4.

Laboratory particle size distribution analyses for six (6) samples from the clayey sandy 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 5 %
- Sand (0.075 mm to 4.75 mm size) - 18 % to 28 %
- Silt (0.002 mm to 0.075 mm size) - 39 % to 45 %
- Clay (less than 0.002 mm size) - 20 % to 39 %

SPT results (N values) recorded in the clayey sandy silt layer ranged from 6 to 23 blows per 305 mm of penetration. Undrained shear strengths, as measured by field vane methods, ranged from 50 kPa to greater than 100 kPa. Based on these results, the consistency of the deposit is described as firm to very stiff. Sensitivity ranged from 2.2 to 4.5 (medium sensitivity to sensitive clay soil). Undrained shear strength was also approximated from pocket penetrometer readings taken in split spoon samples of this layer, and values ranged from 60 kPa to 175 kPa.

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

Liquid Limit (w_L): 22 % to 31 %
Plastic Limit (w_P): 14 %
Plasticity Index (I_P): 8 % to 17 %

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

The natural moisture content of samples recovered from this layer ranged from 10 % to 22 % based on laboratory testing, indicating about plastic limit (APL) to wetter than plastic limit (WTPL) soil.

5.1.4 Clay with some Silt

A layer of clay with some silt was encountered beneath the clayey sandy silt in boreholes BH12-1 to BH12-4. The clay layer is 6.2 m to 7.2 m thick and extends to the depths (mbgs) and elevations (relative) shown below:

Borehole No.	Depth to Bottom of Clay Layer, mbgs (Relative Elevation, m)
BH12-1	13.7 (86.9)
BH12-2	13.7 (86.8)
BH12-3	13.0 (87.8)
BH12-4	13.7 (87.1)

Laboratory particle size distribution analyses for four (4) samples of the clay/silt layer were completed, and results are summarized below and shown on Figure B3 of Appendix B:

- Gravel (greater than 4.75 mm size) - 0 %
- Sand (0.075 mm to 4.75 mm size) - 1 % to 2 %
- Silt (0.002 mm to 0.075 mm size) - 15 % to 27 %
- Clay (less than 0.002 mm size) - 72 % to 83 %

SPT results (N values) recorded for the clay layer ranged from 2 to 7 blows per 305 mm of penetration. Undrained shear strength, as measured by field vane tests, ranged from 43 kPa to 100 kPa. Based on the field results, the consistency of the clay layer is described as firm to stiff. Sensitivity ranged between 2.0 to 4.5 (low to medium sensitivity).

Atterberg Limits tests for four (4) samples from the deposit yielded the following index values:

- Liquid Limit (w_L) - 41 % to 53 %
- Plastic Limit (w_P) - 18 % to 23 %
- Plasticity Index (I_P) - 23 % to 32 %

From the USCS plasticity chart included as Figure B6 in Appendix B, the sample may be classified as inorganic clay of medium plasticity (CI) to clay of high plasticity (CH).

Laboratory determined moisture content ranged between 17 % and 41 % for the clay samples, indicating APL to WTPL material with moisture content generally below the liquid limit.

5.1.5 Silt

Underlying the clay layer, a slightly plastic silt layer with a trace to some clay and a trace to some sand was encountered in boreholes BH12-1 to BH12-4, extending to the end of the boreholes at 15.9 mbgs (relative elevations 84.6 m in borehole BH12-2 to 84.9 m in borehole BH12-3).

Laboratory particle size distribution analyses for two (2) samples of the material were completed; 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) - 1 % to 21 %
- Silt (0.002 mm to 0.075 mm size) - 72 % to 88 %
- Clay (less than 0.002 mm size) - 7 % to 11 %

Standard Penetration Test results (N Values) recorded in this deposit ranged between 4 and 9 blows per 305 mm of penetration, indicating firm to stiff consistency.

5.1.6 Dynamic Cone Penetration Testing

Dynamic cone penetration testing (DCPT) was performed below the borehole termination depths of 15.9 mbgs at boreholes BH12-1 through BH12-4. The DCPTs extended to depths of 23.2 mbgs at BH12-1 and BH12-2 to 21.0 mbgs at boreholes BH12-3 and BH12-4. Refusal, as defined by MTO as 100 blows per 305 mm of penetration, was encountered at the termination depths. The DCPT results indicate that stiff to very stiff and/or compact to dense soil is present between 20.0 and 21.0 mbgs. DCPT's were terminated hard or very dense soil between relative elevations 78.3 m and 80.0 m.

5.2 Groundwater Conditions

Groundwater conditions were observed in the open boreholes upon completion of drilling. Results are summarized in Table 5.1.

Table 5-1: Summary of Groundwater Levels

Location	Measured Groundwater Depth, mbgs (Relative Elevation, m)	Date Measured
BH12-1	9.9 (90.7)	19 June 2012
BH12-2	12.2 (88.3)	19 June 2012
BH12-3	11.6 (89.2)	20 June 2012
BH12-4	10.0 (90.8)	20 June 2012

Note: mbgs = metres below ground surface; Elevation relative to temporary benchmark (see Drawing 1)

Based on the moisture condition and colour 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 between 7.0 and 9.0 m below ground surface within or near the top of the clay unit.

It should be noted that groundwater levels may fluctuate seasonally and in response to climatic conditions. Due to the presence of fine-grained soils beneath the site, a potential for development of perched groundwater exists after wet seasons and periods of rainfall. It is possible that groundwater will become perched in the clayey sandy silt unit, above the clay unit, in wet seasonal periods (i.e. Spring).

6. Geotechnical Design Considerations

The proposed sand/salt storage facility at Detour Patrol Yard will replace an existing salt dome, and will have a circular footprint of approximately 37.8 m in diameter. 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.

Four (4) boreholes (BH12-1 to BH12-4) were drilled to assess the subsurface conditions at the proposed storage facility. The boreholes encountered a thin layer of asphalt overlying loose to compact granular fill, overlying a very stiff to firm clayey sandy silt layer. The boreholes were terminated in firm to stiff silt at a depth of 15.9 m below ground surface (mbgs). DCPTs were advanced 21.0 mbgs to 22.3 mbgs and results are indicate that stiff to very stiff and/or compact to dense soil is present between 20.0 and 21.0 mbgs.

The groundwater depth was estimated to be between 7.0 and 9.0 m below ground surface, at relative elevations between 91.5 m and 93.5 m.

6.1 “Red Flag” Conditions

A relatively thick clayey sandy to clay and silt 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. Settlement analyses assume that a 6.0 m thick clayey sandy silt layer is present, underlain by a 6.0 m thick layer of clay some silt, underlain by 8.0 m of silt some clay. The analyses consider three scenarios for loadings imposed by sand and salt stockpiles. Settlement potential and mitigation measures 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 relatively deep groundwater table, generally within 7 m to 9 m below the ground surface, should not be concern or present construction challenges for foundation construction. Wet silt layers at shallow depths (due to groundwater for example) may be prone to disturbance by construction equipment and workers, and protective measures are required to maintain adequate stability and foundation bearing capacity during construction. Mitigation measures for groundwater are provided in Section 6.7.

6.2 Mitigation of Settlement Potential

As described, the proposed storage structure at the Detour Patrol Yard is underlain by a very stiff to firm clayey sandy silt, firm to stiff clay and silt deposits, estimated to be about 20 m thick. It is understood that the existing storage dome 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 and the new loading footprint may be slightly different than the existing condition. We understand that the new dome footprint will be larger and there is a possibility that the new structure could move slightly. No borehole information is available inside the old dome. Therefore, mitigation of the settlement potential should be considered.

The theoretical settlement potential was predicted using the Skempton (1944) empirical correlation for the compression index: C_C=0.009 (LL- 10) and C_s=10% of C_C. The settlement analysis assumes a 6.0 m thick of upper clayey sandy silt layer underlain by a 6.0 m thick layer of clay some silt, underlain by a 8.0 m thick layer of silt some clay.

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” (3.6 m), with the stockpile periodically replenished throughout the winter. Assumed total weight = 9800 kN (1000 Tonnes).
- Scenario No. 2: Winter sand stacked to the maximum allowable height of the “push wall” at the rear of the facility, occupying ¾ of building’s footprint, with an additional 4900 kN (500 Tonnes) salt stockpile within the front ¼ of the building.
- Scenario No. 3: Storage facility loaded to full capacity. This scenario would consist of winter sand stacked to the maximum height of the “push wall”, with the stockpile area covering the entire footprint of the building.

The estimated effective stress increases (Δp) and the total and differential settlements for each loading are summarized as follows:

Scenario No.	Effective Stress Increase (Δp), kN/m^2	Total Settlement (mm)	Differential Settlement (mm)
Scenario No.1	3.6	20	15
Scenario No.2	24.0	70	25
Scenario No.3	30.0	80	35

Consolidation settlement for the worst case scenario, assuming normally consolidated cohesive soil, is estimated at 70 mm. In addition, there is 10 mm of immediate settlement potential associated with the 8 m thick silt layer below the clay layer. Thus, the total settlement potential under the proposed stockpile/structural loading is estimated at 80 mm. Differential settlement potential is estimated at 35 mm. It is expected that the existing stockpile has provided preconsolidation of the clay subsoils but the magnitude is not known. Also, the loading footprint may change as noted previously. Despite this, we suspect it would not be unreasonable to reduce the total and differential settlement potential due to stockpile loading by 30 % to account for the existing preloading condition from the stockpile provided the new structure is placed over the same area.

Other ground improvement options for the clay layer such as dynamic compaction, rammed aggregate and soil mixing for example, are not considered appropriate for the proposed dome structure, owing to economic factors and options for other foundation types if clay consolidation is a serious concern. Also, it is likely possible to construct the new building with no settlement mitigation, the building is constructed on the same footprint as the existing dome as planned, and preloading from the existing stockpile can be taken into account.

If the proposed building can tolerate up to 60 mm of settlement (20 mm differential), or if the structure can be equipped with adjustable supports that MTO can maintain, then settlement mitigation (such as preloading) may not be necessary. Similarly, if reduced stockpile loadings can be used and building settlements can be monitored and adjusted as required, then mitigation measures may not be required.

If building design tolerances for deflection are lower than stated, or if the new structure footprint is moved beyond the existing footprint, settlement potential due mainly to stockpile loading can be mitigated with a preloading program using a fill surcharge and possibly a vertical wick drainage system. Wick drains, if used, would need to extend to a depth of 20 m to 22 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 and confirm ground movements. 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 ten (10) months.

6.3 Non-standard Special Provisions (NSSP's)

The following Non-standard Special Provisions (NSSP's) are presented to address "Red Flag" conditions described above.

NSSP 1. Due to the presence of compressible materials below the proposed structure, a preloading program is recommended (as described in Section 6.2 of the geotechnical report) to capture approximately 80 mm of estimated total settlement (35 mm differential) under the maximum working load conditions.

NSSP 2. The Contractor should be notified that the soils underneath the proposed foundation include clayey sandy silt to clayey silt materials that are susceptible to sloughing and loosening in wet excavations. The Contractor shall ensure that appropriate construction procedures and equipment are used to maintain the open an adequately cleaned to allow for construction for construction in the dry.

NSSP 3. The Contractor shall ensure that excavation shoring systems are utilized as required by OHSA. Shoring systems shall be designed by a Professional Engineer Specialized in this work.

6.4 Structure Foundation Design Options

Based on the results of this investigation, and in consideration of the settlement mitigation requirements, foundation options are available in view of the following factors:

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

Comments for consideration of foundation design alternatives are provided in Table 6-1.

Table 6-1: Foundation Design Alternatives

Foundation Type	Advantages/ Disadvantages	Reliability	Risks/ Consequences	Recommendations
Strip Footing on Native Clayey sandy silt Layer	Low cost, lower foundation capacity versus deep foundation. Higher settlement potential.	Good, provided that construction practices minimize soil disturbance.	Minor risk of groundwater seepage and subgrade disturbance and subexcavation; pumping may be required depending on seasonal conditions; shoring will be necessary	Recommended, provided good construction practices are used. Preloading is required for site maximum loading condition to remove 80 mm total settlement potential. Foundation must be below frost or insulated.

Slab-on-Grade	Medium cost, medium geotechnical resistance, insulation required, larger foundation settlement versus deep foundation.	Good. Insulation required and must extend beyond structure.	Removal of shallow deleterious material and/or existing soil improvement is required. Larger excavation/disturbed area required for insulation component.	Not Recommended due to economic and constructability reasons. Preloading is required for site maximum loading verification to remove 80 mm total settlement potential.
Drilled and Cast-in-Place Concrete Foundation	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. Liners may be required. Additional drilling required to prove bedrock.	Not Recommended due to economic and constructability reasons. Preloading still required to address settlement potential under stockpile loading.
Steel H Piles	High bearing resistance, low settlement potential subject to down drag forces, 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.	Not Recommended due to economic and constructability reasons. Preloading still required to address settlement potential under stockpile loading.

6.5 Frost Penetration Depth

The recommended design frost protection depth for the site area is 2.5 m (Source: MTO Pavement Design and Rehabilitation Manual). Therefore, a permanent soil cover of about 2.5 m or its thermal equivalent of high density 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.

6.6 Preferred Foundation Option

Based on the results of this investigation, analysis of foundation options and assuming a preloading program will be undertaken (or that the building can be designed to tolerate differential settlements) the proposed sand/salt storage facility can be supported on spread/strip footings, founded in the undisturbed silt/sandy silt layer, with a recommended founding level at 2.5 m depth (elevation 98.0 m to 98.3 m) for frost protection purposes.

The following geotechnical resistances are appropriate for a minimum 0.9 m footing width.

- Factored Geotechnical Resistance at Ultimate Limit State (ULS) = 220 kPa
- Geotechnical Resistance at Serviceability Limit State (SLS) = 160 kPa

Geotechnical Resistance at Serviceability Limit State (SLS) is based on maximum total and differential settlements of 25 mm and 20 mm, respectively, due to foundation loading, and assumes settlement potential due to stockpile loading will be addressed as discussed in Section 6.2.

Based on limited testing, existing granular fill materials may be suitable for reuse, subject to Engineer approval onsite. The Geotechnical Engineer shall confirm materials are suitable to support design loadings and that all disturbed or loose soils are properly removed from below all footing areas. It should be noted that silty materials at the anticipated founding level can be easily disturbed by foot traffic. Thus, the base should be covered with a minimum 50 mm thick mud slab immediately after inspection and approval.

6.7 Resistance to Lateral Loads

Resistance to lateral forces/sliding between the concrete footings and subsoils should be calculated in accordance with Section 6.7.5 of the CHBDC. The adhesion (Ca) which develops for cast-in-place concrete footings constructed on undisturbed clayey sandy silt may be taken as 70 percent of the untrained shear strength (c_u). Thus, the average value of the adhesion for design should be taken as 70 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 Backfill and Lateral Earth Pressure

Backfill behind foundation/retaining walls should consist of non-frost susceptible, free-draining backfill materials (i.e. Granular ‘A’ or Granular ‘B’ Type I or II, with no more than 8 % passing the 0.75 mm sieve as per requirement of OPSS 1010 and its Amendment No. 110S13).

Computation of earth pressures acting against walls should be in accordance with the CHBDC. For design purposes, the properties outlined in Table 6-2 can be assumed for backfill.

Table 6-2: Backfill Properties

Property	Compacted Granular ‘A’ or Granular ‘B’ Type II	Compacted Granular ‘B’ Type I
Angle of Internal Friction ϕ (unfactored)	35°	32°
Unit Weight γ	22 kN/m^3	21 kN/m^3
Coefficients of Lateral Earth Pressure		
K_a	0.27	0.31
K_b	0.35	0.41
K_o	0.43	0.47
K*	0.45	0.57

Notes:

- K_a is the coefficient of active earth pressure
- K_b is the backfill earth pressure coefficient for an unrestrained structure, including compaction effects
- K_o is the coefficient of earth pressure at rest
- K* is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

Earth pressure coefficients are based on the assumption that the backfill behind retaining structures is free-draining granular material and adequate drainage is provided.

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:

$$P = K(\gamma h + q)$$

where:

P = lateral earth pressure (kPa) acting at depth h
K = earth pressure coefficient
γ = unit weight of backfill (kN/m^3)
h = depth to point of interest in metres
q = equivalent value of surcharge on the ground surface in kPa

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 6-3: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_O	K_p	γ (kN/m^3)
Granular Fill	0.33	0.5	3.0	19.0
Stiff Clay Silt	0.35	0.55	2.8	18.5
Firm to Stiff Clay	0.33	0.5	3.0	18.0

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), and in the absence of shear wave velocity tests, it is recommended that Site Class 'D' (stiff soil) be applied for structural design at this site.

Seismic information for the Detour 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	Detour	Source
Site Class	D	2006 Ontario Building Code Table 4.1.8.4.A
$S_a(0.2)$	0.159	2005 National Building Code Seismic Hazard Calculation
$S_a(1.0)$	0.044	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. Owing to the presence of mainly fine-grained cohesive soils and relatively deep groundwater relative to the foundation, dynamic and static liquefaction at the foundation soils are not expected to be a concern at this site.

6.10 Dewatering and Drainage

It is anticipated that groundwater exists below relative elevation 91 m (see Drawing 1). Therefore, the bottom of the foundation excavation should not encounter significant seepage and dewatering should not be required to stabilize the soil during construction.

The predominant soils encountered in the boreholes range in texture from upper granular fill underlain by clayey sand silt. Seepage from intermittently saturated granular layers may occur, depending on seasonal conditions at the time of construction. The clayey silt soils generally exhibit characteristics of low permeability, and seepage from this type of soil into foundation excavations should be relatively slow.

If groundwater encountered during the excavation, it can be lowered by about 0.5 m by pumping from strategically placed filtered sumps and using gravity drainage. For more extensive drawdown, vacuum well points and/or deeper purge wells could be used. It is recommended that the Contractor be requested to submit dewatering schemes to the MTO Project Manager for approval, prior to construction. Dewatering procedures should follow the requirements and specifications of OPSS 517. The Contractor should obtain a Permit to Take Water if he expects dewatering rates in excess of 50,000 L/day.

Since the structure foundation will be backfilled with granular material, it is recommended that 100 mm diameter geotextile wrapped subdrains be installed at exterior footing level. The subdrains should follow the footing perimeter and be connected to a frost free outlet for gravity drainage.

6.11 Excavations and General Construction Consideration

Construction excavations are required for foundations 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 OPSS 539 – Protection Systems and OPSS 902 – Excavations and Backfilling to Structure. The soils at the site may be classified as shown below, in accordance with the OHSA.

Table 6-4: Soil Classification for Excavations

Soil Type	Above Groundwater Level	Below Groundwater Level
Fill material	Type 3	Type 4 (not expected)
Stiff clayey silt	Type 2	Type 3
Firm clay some silt	Type 3	Type 4
Silt	Type 3	Type 4

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 to grade. 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). All excavation and grading procedures should follow the MTO’s requirements and specifications, and management of excess material should follow the requirements of OPSS 180.

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.

Regular inspections by qualified geotechnical engineering personnel must be conducted for any excavation in the bedrock to confirm that conditions are safe and consistent with the requirements of the OHSA.

Since the subject site was used for many years to store road salt, and will be used in the future for the same purpose, it is expected that the new foundation will be exposed to chloride, sodium and sulfate attack. To reduce damage potential and rate of deterioration, we recommend to use high sulfate-resistant cement (Type HS as per CSA A.23) in the concrete mix design with water-cement ratio should not exceed 0.45.

7. Miscellaneous Information

The following GENIVAR personnel and subcontractors responsible for completion of this foundation 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	Beverly Leno, C.E.T., rcji GENIVAR Inc.	Peterborough, ON	705-743-6850
Laboratory Low Complexity	Kelly Whitney, C.E.T. GENIVAR Inc.	Peterborough, ON	705-743-6850
Report Preparation	Raid Khamis, P. Eng, PMP. 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	Jason Balsdon, M.A.Sc., P. Eng. GENIVAR Inc.	Newmarket, ON	905-853-3303

8. Closure

The data presented in this foundation investigation 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.

J. Stephen Ash, P. Eng., P. Geo.
Director, Environment

LICENSED PROFESSIONAL ENGINEER
J. S. Ash
June 5/13
PROVINCE OF ONTARIO

Reviewed by:

Jason Balsdon, M.A.Sc., P. Eng.
Director, Environment

REGISTERED PROFESSIONAL ENGINEER
J. T. Balsdon
June 5/13
PROVINCE OF ONTARIO

Drawings

Drawing 1 – Borehole Location Plan

Drawing 2 – Soil Strata

METRIC

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

Agreement No.: 5011-E-0010
WO No.: 2011-11035

BOREHOLE LOCATION PLAN

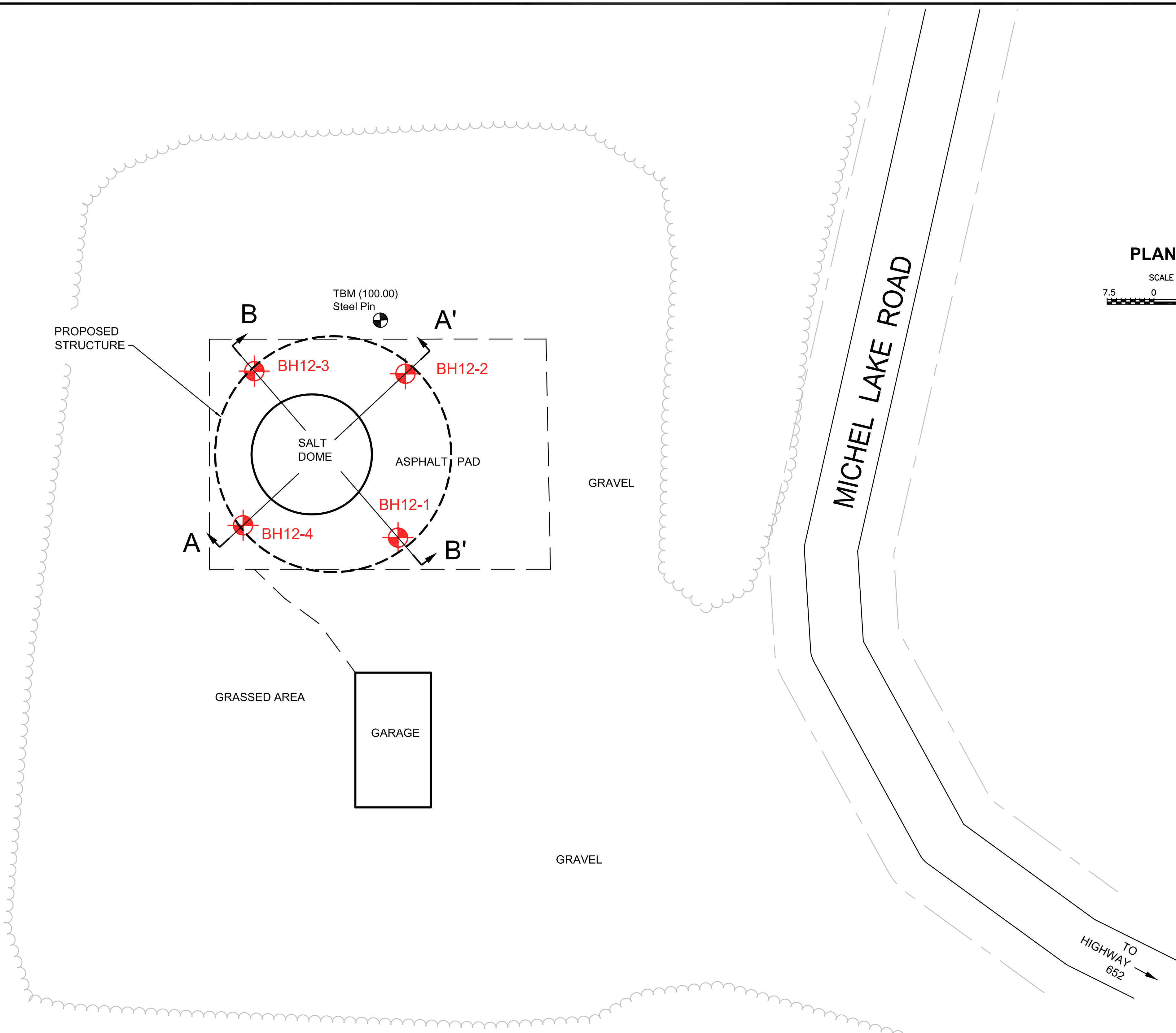
PROPOSED SAND/SALT STORAGE FACILITY
DETOUR PATROL YARD
HIGHWAY 652

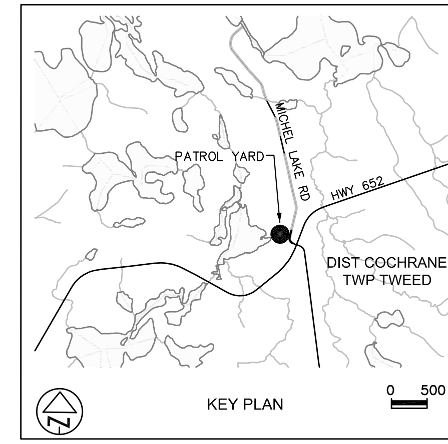
Client: MTO - Northeastern Region

DRAWING 1

PLAN VIEW

SCALE 1:750





KEY PLAN

LEGEND

- Borehole and Cone
- Tempory Benchmark (Assumed 100.00 m)
- Proposed Sand/Salt Storage Facility
- A — A' Line of Cross Section (See Figure 2)

BH No	ELEVATION (Relative m)	COORDINATES (NAD 83 Zone17) NORTHING	EASTING
12–1	100.579	5479240.6	541944.9
12–2	100.540	5479266.9	541946.1
12–3	100.797	5479226.3	541921.9
12–4	100.780	5479242.6	541920.1

– NOTE –

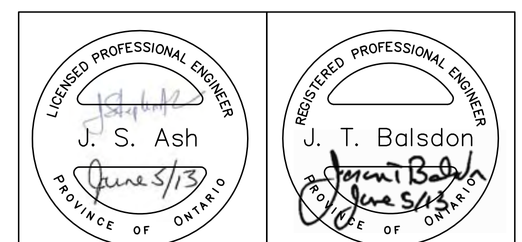
THE ACTUAL SOIL STRATIFICATION HAS BEEN VERIFIED FROM DATA OBTAINED AT THE BOREHOLE LOCATIONS ONLY. THE INFERRED CONTACTS SHOWN ARE BASED ON GEOLOGICAL EVIDENCE AND THESE MAY VARY FROM THOSE SHOWN BETWEEN BORINGS.

NOTES:

1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
2. COORDINATES AT BOREHOLE LOCATIONS WERE RECORDED BY HANDHELD GPS.
3. BOREHOLE ELEVATIONS WERE SURVEYED RELATIVE TO TEMPORARY BENCHMARK STEEL PIN NORTH OF BH12–2 (RELATIVE EL. 100.00 m).

PROJECT: 121–17876–00 111–06

GENIVAR



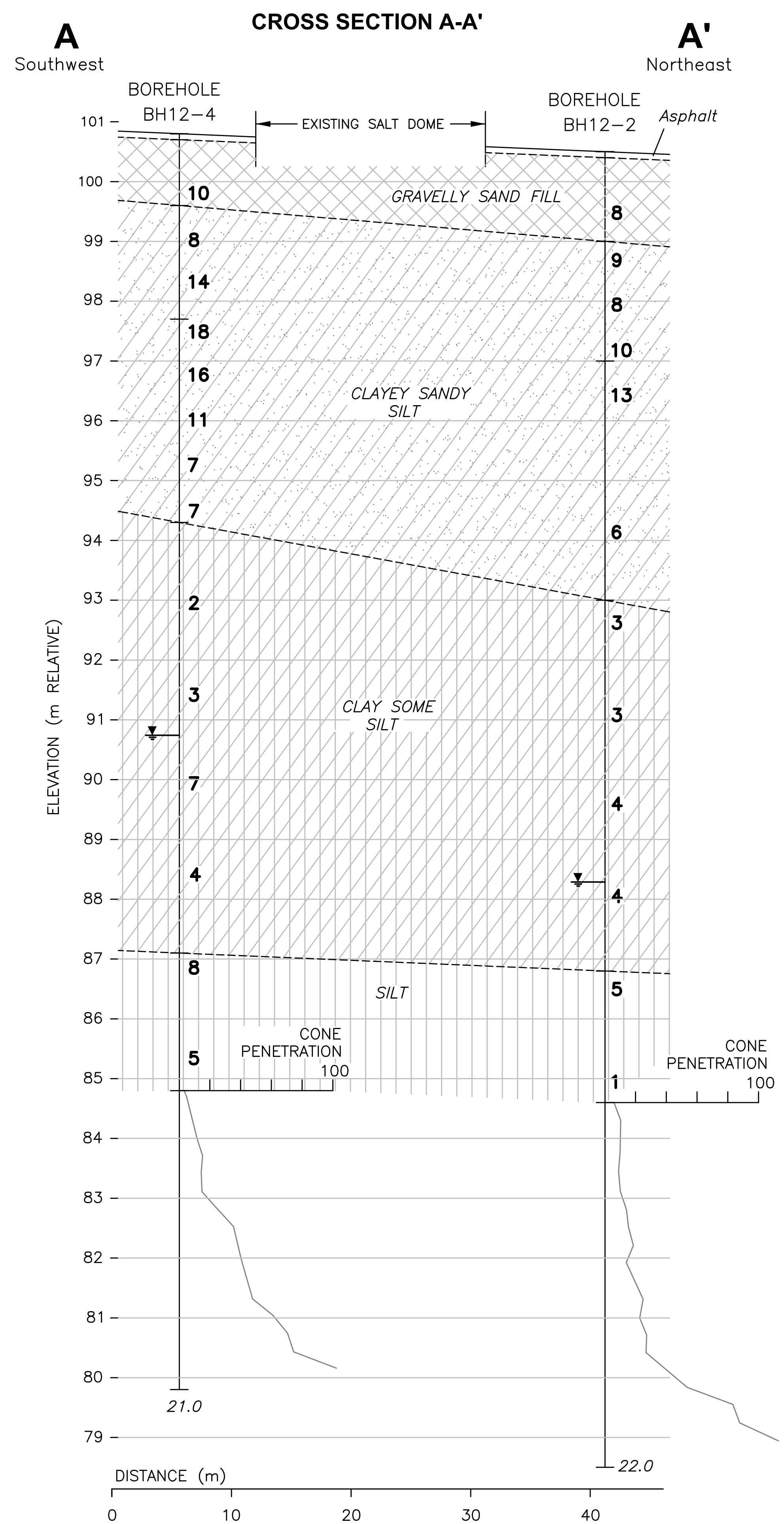
SITE PLAN MAPPING REF. NO.:
SITE PLAN SKETCH, NOT TO SCALE.

REVISIONS DATE	BY	DESCRIPTION

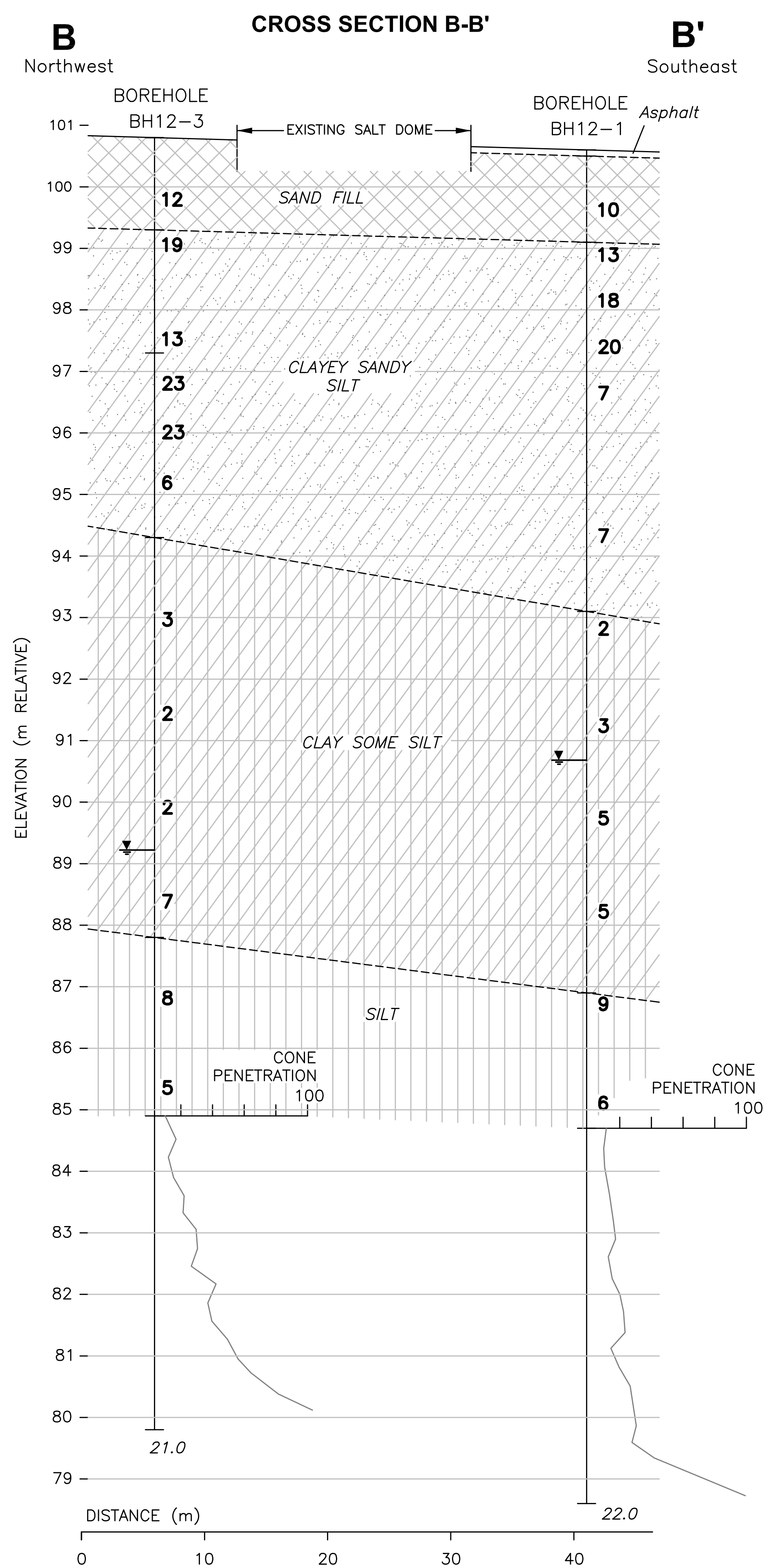
GEOCRES No. 42H–52

HWY No 652			DIST COCHRANE
SUBM'D --	CHECKED JSA	DATE MARCH 2013	SITE --
DRAWN PLB	CHECKED --	APPROVED --	DWG --

CROSS SECTION A-A'



CROSS SECTION B-B'



METRIC

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

Agreement No.: 5011-E-0010
WO No.: 2011-11035

SOIL STRATA

PROPOSED SAND/SALT STORAGE FACILITY
DETOUR PATROL YARD
HIGHWAY 652

Client: MTO - Northeastern Region

DRAWING
2

NOTES:

1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
2. COORDINATES AT BOREHOLE LOCATIONS WERE RECORDED BY HANDHELD GPS.
3. BOREHOLE ELEVATIONS WERE SURVEYED RELATIVE TO TEMPORARY BENCHMARK STEEL PIN NORTH OF BH12-2 (RELATIVE EL. 100.00 m).

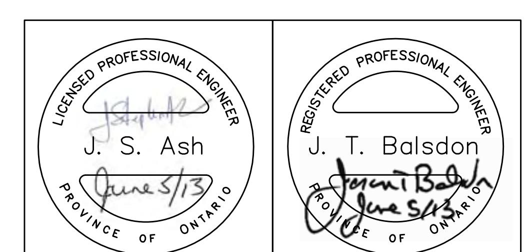
LEGEND

N	Blows/0.3m (Std. Pen Test, 475 J / blow)
CONE	Blow/0.3m (60° Cone, 475 J / blow)
▼	Water Level At Time Of Investigation

BH No	ELEVATION (Relative m)	COORDINATES (NAD 83 Zone17) NORTHING	EASTING
12-1	100.579	5479240.6	541944.9
12-2	100.540	5479266.9	541946.1
12-3	100.797	5479226.3	541921.9
12-4	100.780	5479242.6	541920.1

PROJECT: 121-17876-00 111-06

GENIVAR



– 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. 42H-52

HWY No 652			DIST COCHRANE
SUBM'D --	CHECKED JSA	DATE MARCH 2013	SITE --
DRAWN PLB	CHECKED --	APPROVED --	DWG --

SITE PLAN MAPPING REF. NO.:
SITE PLAN SKETCH, NOT TO SCALE.

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.

Soil Classification*		Terminology	Proportion
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:

COHESIONLESS SOIL		COHESIVE SOIL		
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.

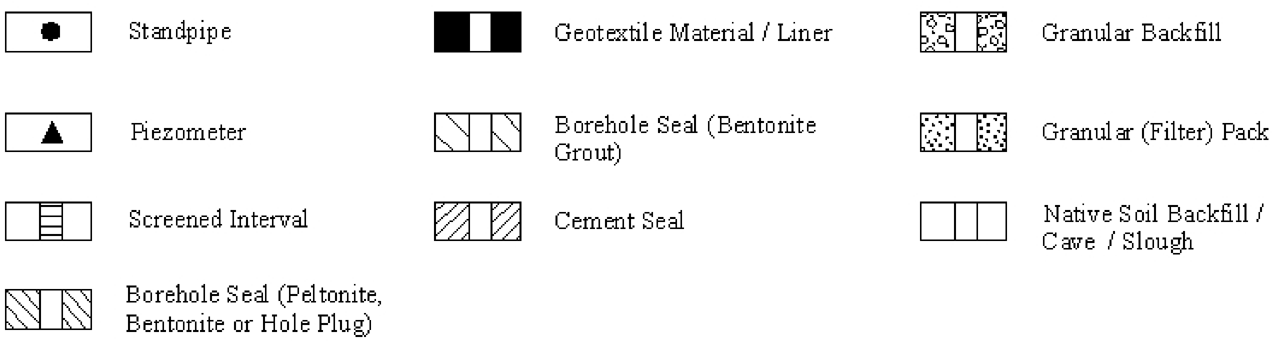
COHESIONLESS SOILS	COHESIVE SOILS		
Dry	DTPL	-	Drier Than Plastic Limit
Moist	APL	-	About Plastic Limit
Wet	WTPL	-	Wetter Than Plastic Limit
Saturated	MWTPL	-	Much Wetter Than Plastic Limit

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.



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	TCR	=	Total Core Recovery

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

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, 60º 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 $\frac{xBlows}{mm}$

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

W_P - Plastic Limit of a fine-grained soil expressed as a percentage as determined from the Atterberg Limit Test.

W_L - 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

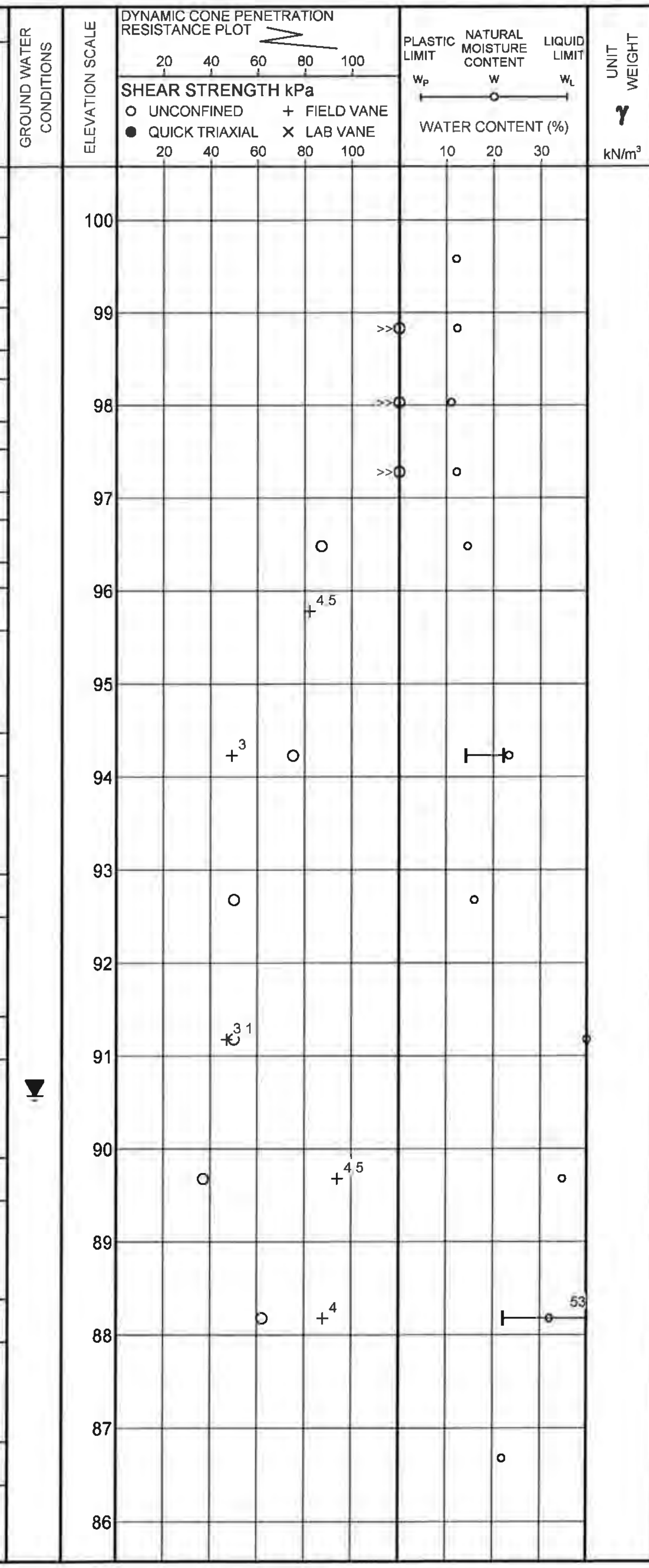
METRIC

LOCATION: DETOUR PATROL YARD N 5479240.6 ; E 541944.9
ORIGINATED BY: DCL

BOREHOLE TYPE: CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT
COMPILED BY: DCL/BDL

DATUM: GEODETIC
DATE: 6.19.12 - 6.19.12
CHECKED BY: RK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
100.6					
100.6 0.0	**40 mm ASPHALT:**				
	GRANULAR FILL: GRAVELLY SAND TO SAND SOME SILT FILL BROWN, COMPACT, MOIST	1	AUGER		
		2	SS	10	
99.1 1.5	**CLAYEY SANDY SILT :** CLAYEY SANDY SILT BROWN, VERY STIFF TO FIRM, APL TO WTPL	3	SS	13	
		4	SS	18	1 27 45 27
		5	SS	20	
		6	SS	7	
		7	VANE		
		8	SS	7	1 28 39 32
93.1 7.5	**CLAY SOME SILT :** CLAY SOME SILT, TRACE SAND GREY, FIRM TO STIFF, WTPL	9	SS	2	
		10	SS	3	
		11	SS	5	
		12	SS	5	0 1 19 80
86.9 13.7	**SILT:** SILT, SOME SAND, TRACE CLAY GREY, STIFF TO FIRM , DILATENT WET	13	SS	9	



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$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

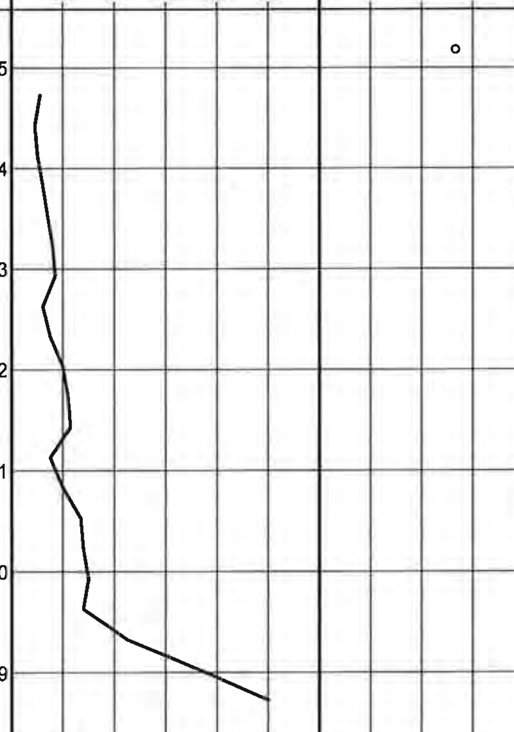
ONTARIO MOT DETOUR BH LOGS.GPJ ONTARIO MOT.GDT 2/13/13

RECORD OF BOREHOLE No BH12-1

METRIC

LOCATION	DETOUR PATROL YARD N 5479240.6 ; E 541944.9	ORIGINATED BY	DCL
BOREHOLE TYPE	CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT	COMPILED BY	DCL/BDL
DATUM	GEODETIC DATE 6 19 12 - 6 19 12	CHECKED BY	RK

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT: SILT, SOME SAND, TRACE CLAY GREY, STIFF TO FIRM , DILATENT, WET *(continued)*		14	SS	6		85				
84.7 15.9	DYNAMIC CONE PENETRATION TEST BELOW 15.9 m DEPTH NO SOIL SAMPLING COMPLETED						84 83 82 81 80 79				
78.3 22.3	END OF BOREHOLE										



$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH12-2

METRIC

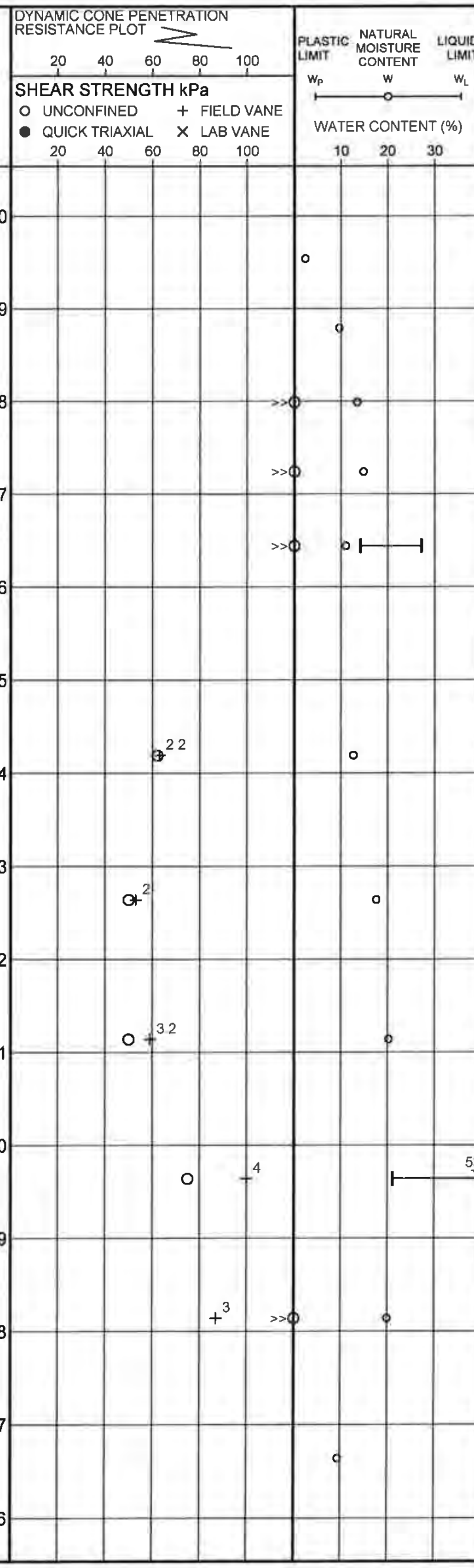
LOCATION	DETOUR PATROL YARD N 5479266.9 ; E 541946.1
BOREHOLE TYPE	CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT
DATUM	GEODETIC
DATE	6.19.12 - 6.19.12
ORIGINATED BY	DCL
COMPILED BY	DLC/BDL
CHECKED BY	RK

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
100.5	
100.5 / 0.0	**40 mm ASPHALT:**
	GRANULAR FILL: GRAVELLY SAND SOME SILT FILL, BROWN, COMPACT, MOIST
99.0 / 1.5	**CLAYEY SANDY SILT:** CLAYEY SANDY SILT BROWN, STIFF, MOIST
93.0 / 7.5	**CLAY SOME SILT:** CLAY SOME SILT, TRACE SAND GREY, STIFF, WTPL
86.8 / 13.7	**SILT:** SILT, SOME SAND, TRACE CLAY GREY, FIRM, DILATENT, WET

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AUGER		
2	SS	8	4 87 (9)
3	SS	9	
4	SS	8	
5	SS	10	
6	SS	13	0 18 45 37
7	ST		
8	SS	6	
9	SS	3	
10	SS	3	
11	SS	4	
12	SS	4	0 1 17 82
13	SS	5	



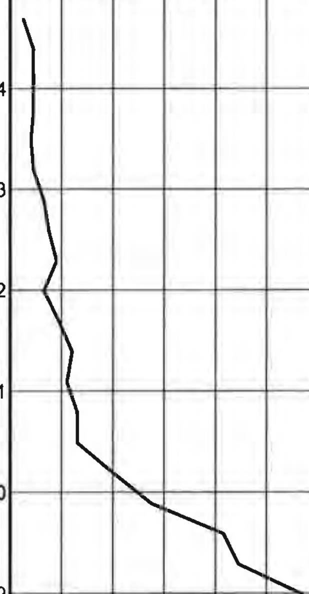
Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

ONTARIO MOT DETOUR BH LOGS.GPJ ONTARIO MOT.GDT 2/13/13

LOCATION	DETOUR PATROL YARD N 5479266.9 ; E 541946.1	ORIGINATED BY	DCL
BOREHOLE TYPE	CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT	COMPILED BY	DLC/BDL
DATUM	GEODETIC DATE 6.19.12 - 6.19.12	CHECKED BY	RK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SILT : SILT, SOME SAND, TRACE CLAY GREY, FIRM, DILATENT, WET *(continued)*		14	SS	4		85				
84.6 / 15.9	DYNAMIC CONE PENETRATION TEST BELOW 15.9 m DEPTH NO SOIL SAMPLING COMPLETED						84				
							83				
							82				
							81				
							80				
							79				
78.3	END OF BOREHOLE										
22.3	END OF BOREHOLE										



$+^3, \times^3$: Numbers refer to Sensitivity O$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH12-3

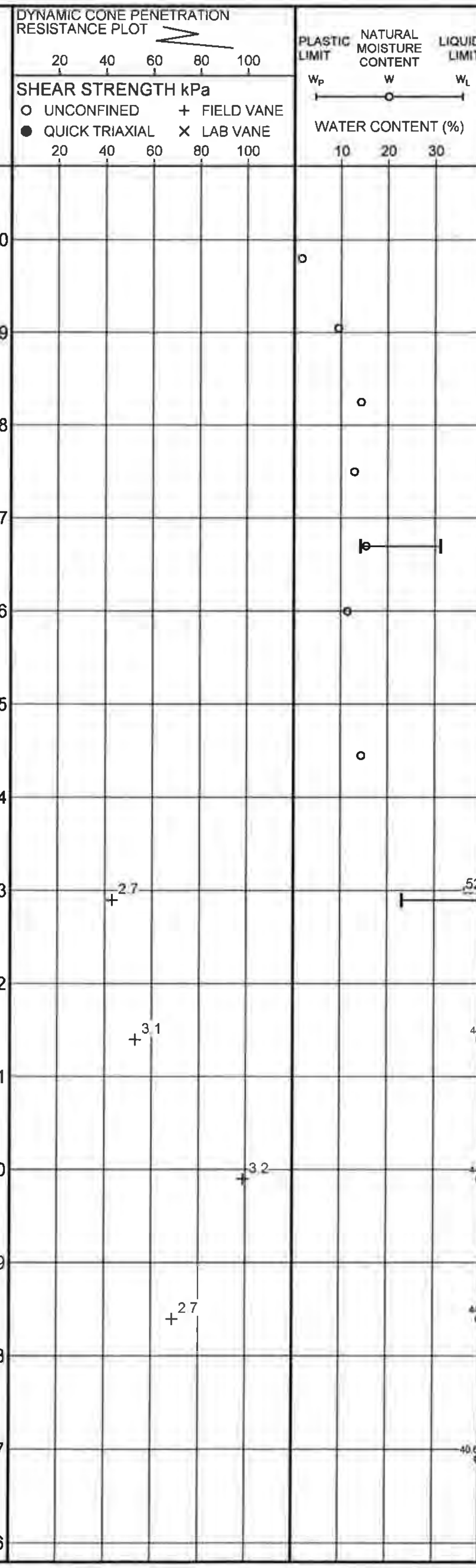
METRIC

LOCATION: DETOUR PATROL YARD N 5479226.3 ; E 541921.9 — ORIGINATED BY: DCL

BOREHOLE TYPE: CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT — COMPILED BY: DCL/BDL

DATUM: GEODETIC — DATE: 6.20.12 - 6.20.12 — CHECKED BY: RK

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
100.8 / 0.0	**GRANULAR FILL:** SAND TO SANDY SILT FILL, SOME GRAVEL BROWN, COMPACT, MOIST	1	AUGER			
		2	SS	12	100	
99.3 / 1.5	**CLAYEY SANDY SILT :** CLAYEY SANDY SILT BROWN, STIFF TO VERY STIFF, MOIST	3	SS	19	99	1 37 42 20
		4	SS	8	98	
		5	SS	13	97	
		6	SS	23		1 21 39 39
		7	SS	23	96	
					95	
		8	SS	6		
94.3 / 6.5	**CLAY SOME SILT :** CLAY SOME SILT, TRACE SAND GREY, FIRM TO STIFF, WET				94	
		9	SS	3	93	0 2 15 83
					92	
		10	SS	2	91	
		11	SS	2	90	
					89	
		12	SS	7	88	
87.8 / 13.0	**SILT :** SILT, SOME CLAY, TRACE SAND GREY, STIFF TO FIRM, DILATENT, WET					
		13	SS	8	87	0 1 88 11
					86	



UNIT WEIGHT γ kN/m³

Continued Next Page

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

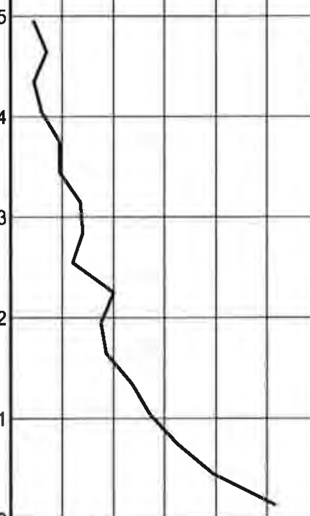
RECORD OF BOREHOLE No BH12-3

METRIC

LOCATION: DETOUR PATROL YARD N 5479226.3 ; E 541921.9 — ORIGINATED BY: DCL

BOREHOLE TYPE: CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT — COMPILED BY: DCL/BDL

DATUM: GEODETIC — DATE: 6.20.12 - 6.20.12 — CHECKED BY: RK

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT : SILT, SOME CLAY, TRACE SAND GREY, STIFF TO FIRM , DILATENT, WET *(continued)*		14	SS	5						
84.9 15.9	DYNAMIC CONE PENETRATION TEST BELOW 15.9 m DEPTH NO SOIL SAMPLING COMPLETED.						85, 84, 83, 82, 81, 80				
79.8 21.0	END OF BOREHOLE										

$+^3, \times^3$: Numbers refer to Sensitivity — $O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH12-4

METRIC

LOCATION: DETOUR PATROL YARD N 5479242 6 ; E 541920 1

ORIGINATED BY: DCL

BOREHOLE TYPE: CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT

COMPILED BY: DCL/BDL

DATUM: GEODETIC

DATE: 6 20 12 - 6 20 12

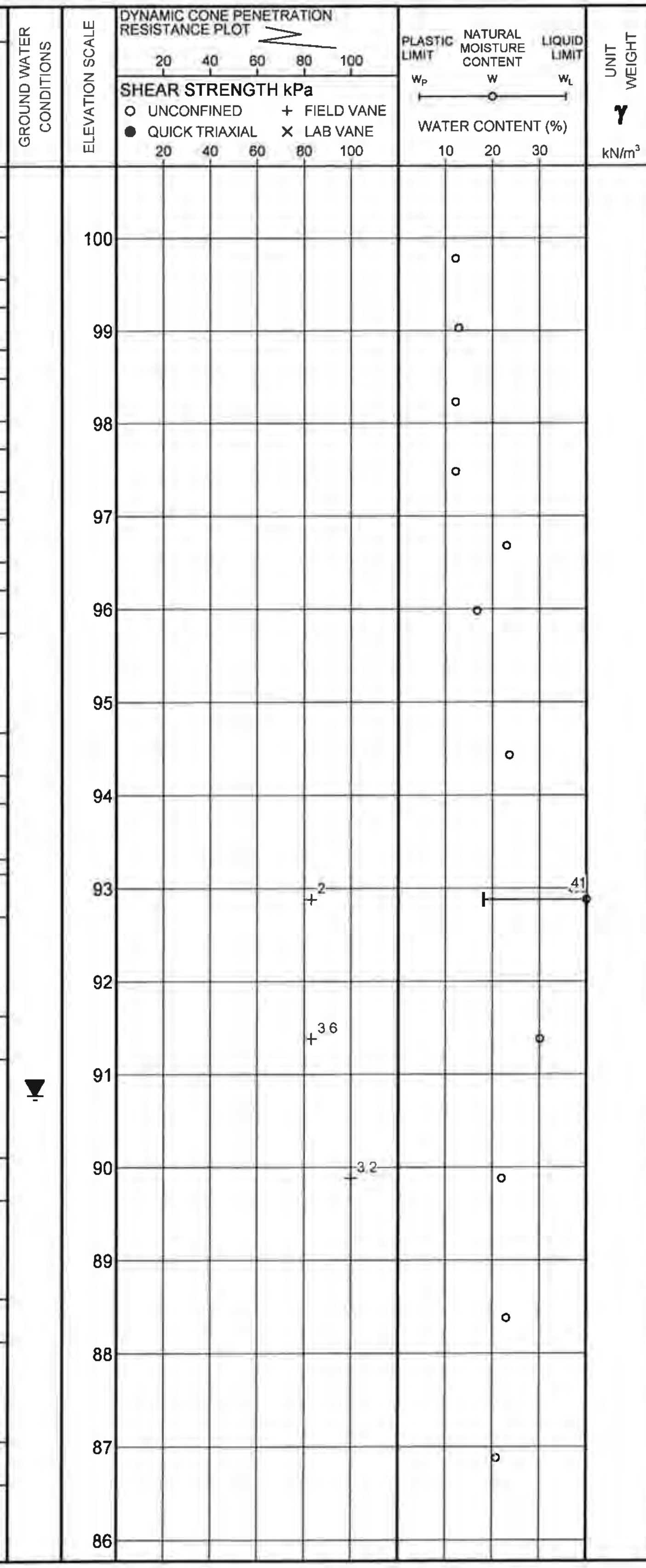
CHECKED BY: RK

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
100.8	
100.7 / 0.1	**50 mm ASPHALT:**
	GRANULAR FILL: GRAVELLY SAND TO SANDY SILT FILL BROWN, COMPACT, MOIST
99.6 / 1 2	**CLAYEY SANDY SILT :** CLAYEY SANDY SILT BROWN, VERY STIFF TO STIFF, APL TO WTPL
94.3 / 6 5	**CLAY SOME SILT :** CLAY SOME SILT, TRACE SAND GREY, STIFF, MOIST TO WET
87 1 / 13 7	**SILT :** SILT, SOME SAND, TRACE CLAY GREY, STIFF TO FIRM , DILATENT, WET

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AUGER		
2	SS	10	
3	SS	8	
4	SS	14	5 27 42 26
5	SS	18	
6	SS	16	
7	SS	11	
8	SS	7	
9	ST		
10	SS	2	0 1 27 72
11	SS	3	
12	SS	7	
13	SS	4	
14	SS	8	



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$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No BH12-4

METRIC

LOCATION: DETOUR PATROL YARD N 5479242.6 ; E 541920.1

ORIGINATED BY: DCL

BOREHOLE TYPE: CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT

COMPILED BY: DCL/BDL

DATUM: GEODETIC

DATE: 6.20.12 - 6.20.12

CHECKED BY: RK

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	NATURAL MOISTURE CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	SILT : SILT, SOME SAND, TRACE CLAY GREY, STIFF TO FIRM , DILATENT, WET *(continued)*	15	SS	5	~23 (plotted)	0	21	72	7
84.9 / 15.9	DYNAMIC CONE PENETRATION TEST BELOW 15.9 m DEPTH NO SOIL SAMPLING COMPLETED								
79.8 / 21.0	END OF BOREHOLE								

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ 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 and B6)

Table B1: Summary of Grain Size Distribution

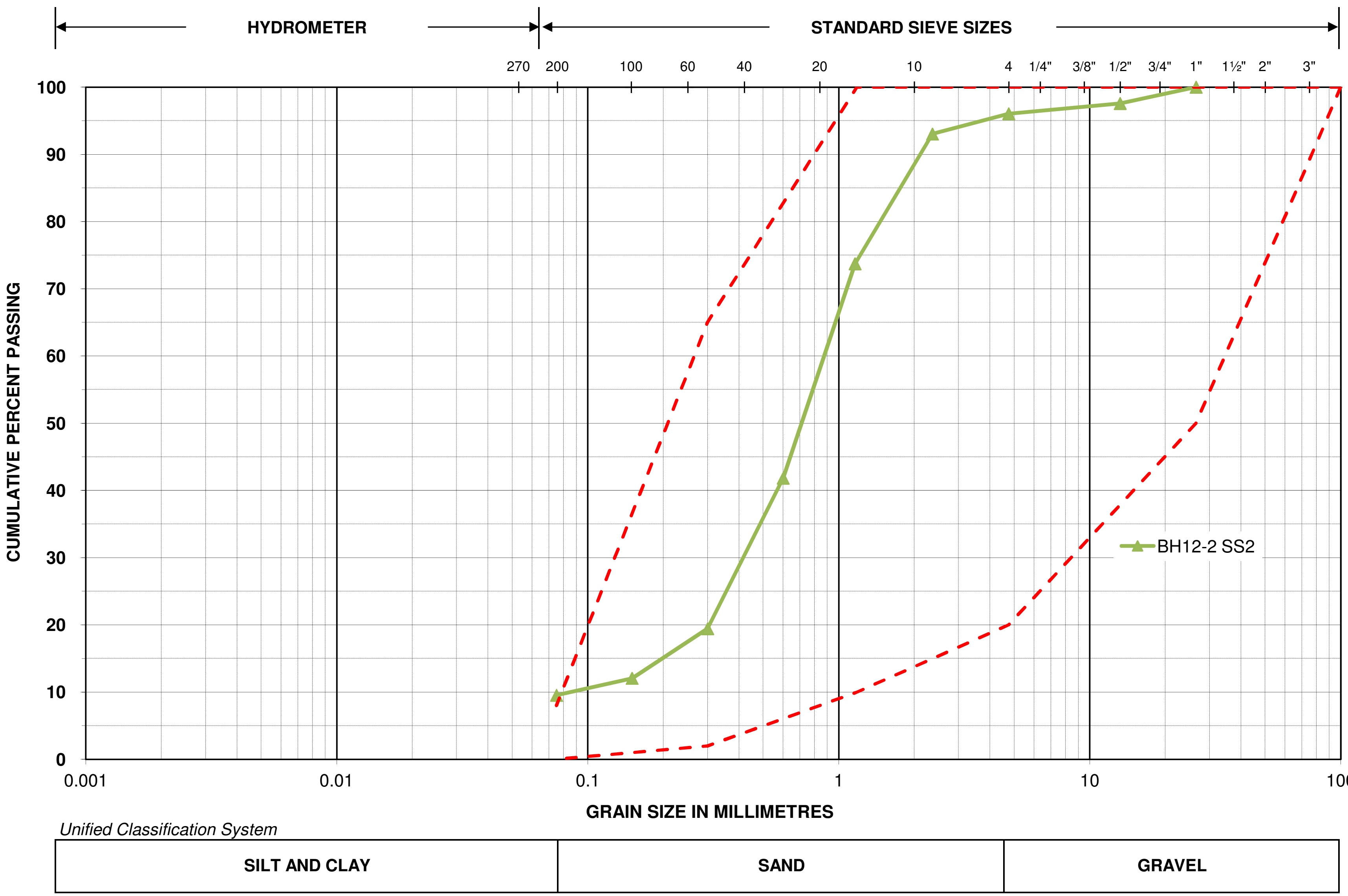
Borehole No.	Sample ID	Soil Description	Percentage Retained (%)			
			Gravel	Sand	Silt	Clay
BH12-1	SS4	Clayey sandy silt, trace gravel	1	27	45	27
BH12-1	SS8	Clayey sandy silt, trace gravel	1	28	39	32
BH12-1	SS12	Clay, some silt, trace sand	0	1	19	80
BH12-2	SS2	Sand, trace silt, trace gravel	4	87	9	
BH12-2	SS6	Silt and clay, some sand	0	18	45	37
BH12-2	SS11	Silty clay, trace sand	0	1	17	82
BH12-3	SS3	Clayey silt and sand, trace gravel	1	37	42	20
BH12-3	SS6	Sandy clay and silt, trace gravel	1	21	39	39
BH12-3	SS9	Clay, some silt, trace sand	0	2	15	83
BH12-3	SS13	Silt, some clay, trace sand	0	1	88	11
BH12-4	SS4	Sandy clayey silt, trace gravel	5	27	42	26
BH12-4	SS10	Silty clay, trace sand	0	1	27	72
BH12-4	SS15	Sandy silt, trace clay	0	21	72	7

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%

NOTE:
Division of Particle Sizes (USCS except clay based on MIT division)

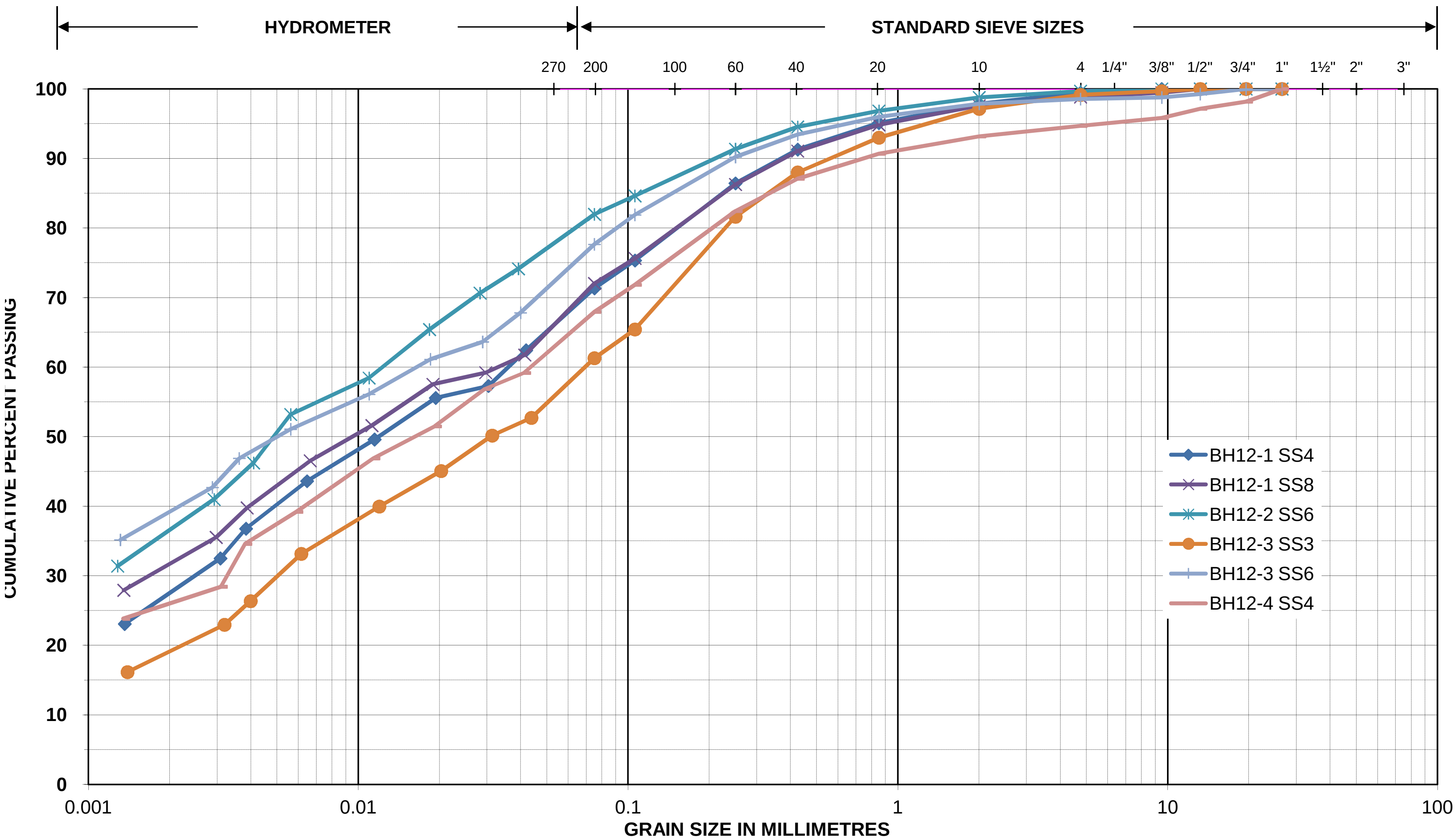
- Gravel > 4.75 mm
- Sand 0.075 mm to 4.75 mm
- Silt 0.002 mm to 0.075 mm
- Clay < 0.002 mm

PARTICLE SIZE DISTRIBUTION



Project Name:	MTO Agreement 5011-E-0010 - Detour Patrol Yard	**Project No.:**	121-17876-00	**Figure No.:**	**B1**
Remarks:	Sand, trace silt, trace gravel (Fill Layer)				

PARTICLE SIZE DISTRIBUTION ASTM D422



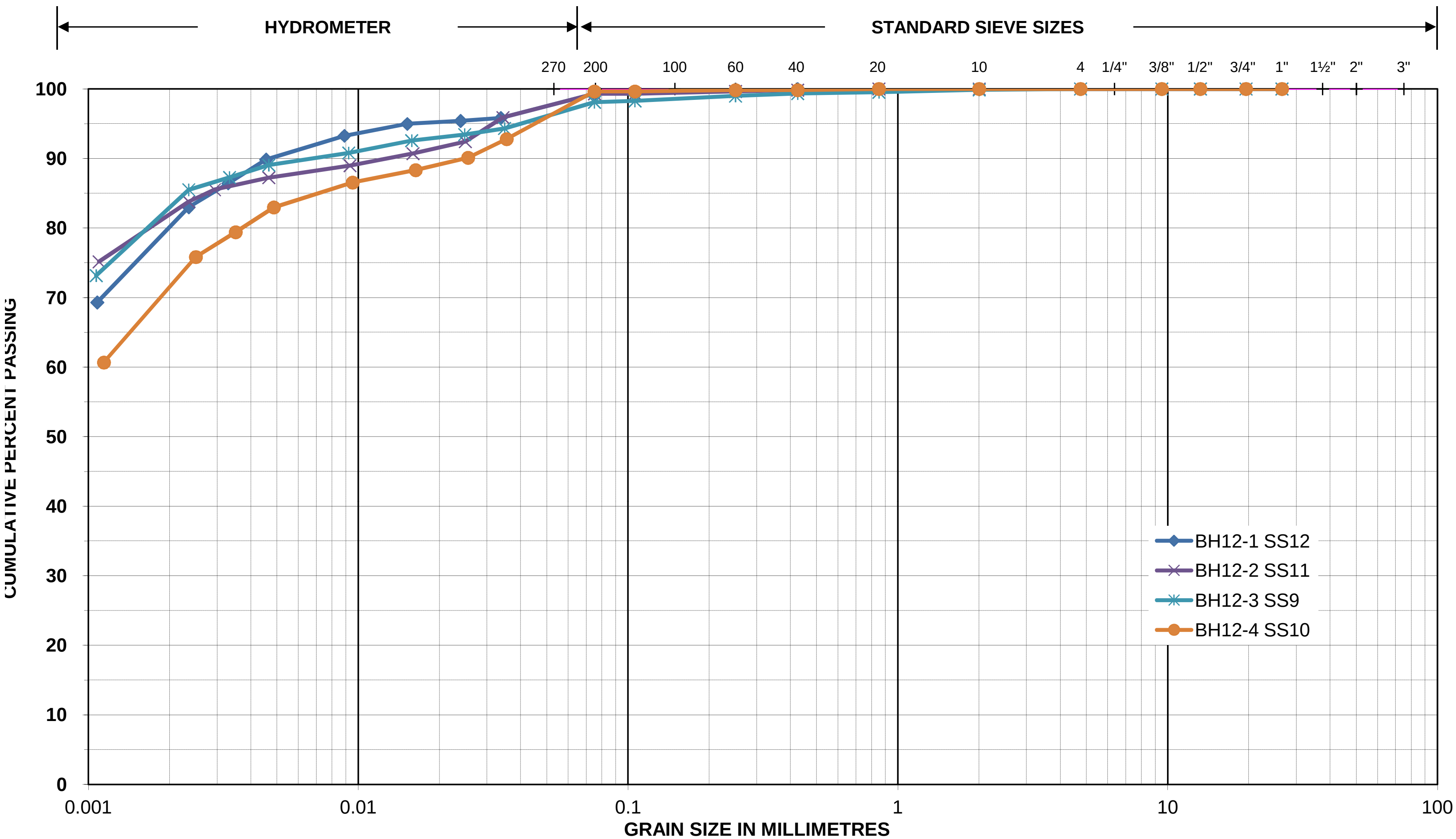
Unified Classification System

SILT AND CLAY	SAND	GRAVEL

Project Name:	MTO Agreement #5011-E-0010 - Detour Patrol Yard	**Project No.:**	121-17876-00	**Figure No.:**	B2
Remarks:	Clayey sandy silt, trace gravel				

GENIVAR

PARTICLE SIZE DISTRIBUTION ASTM D422



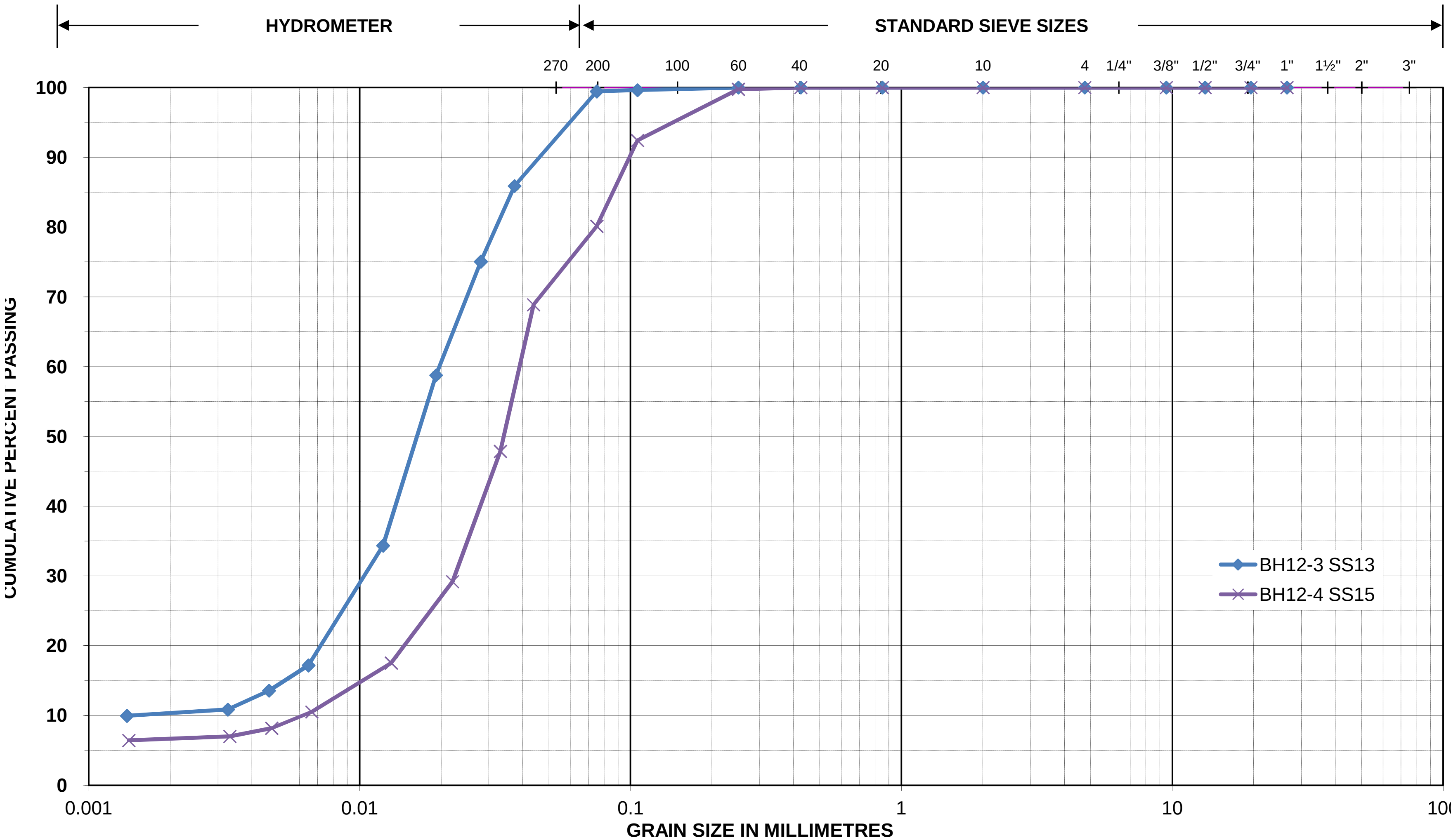
Unified Classification System

SILT AND CLAY	SAND	GRAVEL

Project Name:	MTO Agreement #5011-E-0010 - Detour Patrol Yard	**Project No.:**	121-17876-00	**Figure No.:**	B3
Remarks:	Clay, some silt, trace sand				

GENIVAR

PARTICLE SIZE DISTRIBUTION ASTM D422

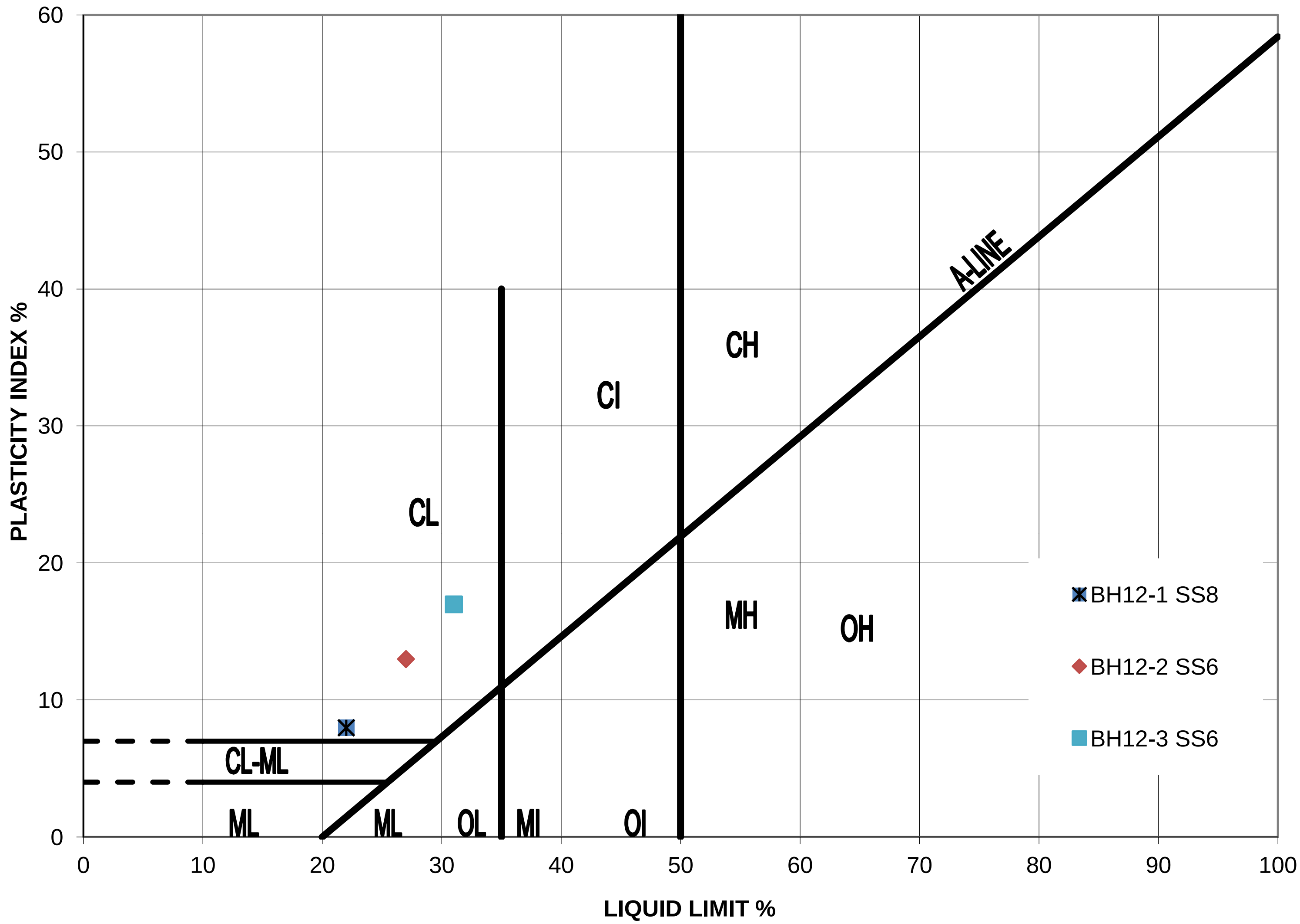


Unified Classification System

SILT AND CLAY	SAND	GRAVEL

Project Name:	MTO Agreement #5011-E-0010 - Detour Patrol Yard	**Project No.:**	121-17876-00	**Figure No.:**	**B4**
Remarks:	Silt, some to trace clay, trace sand				

GENIVAR



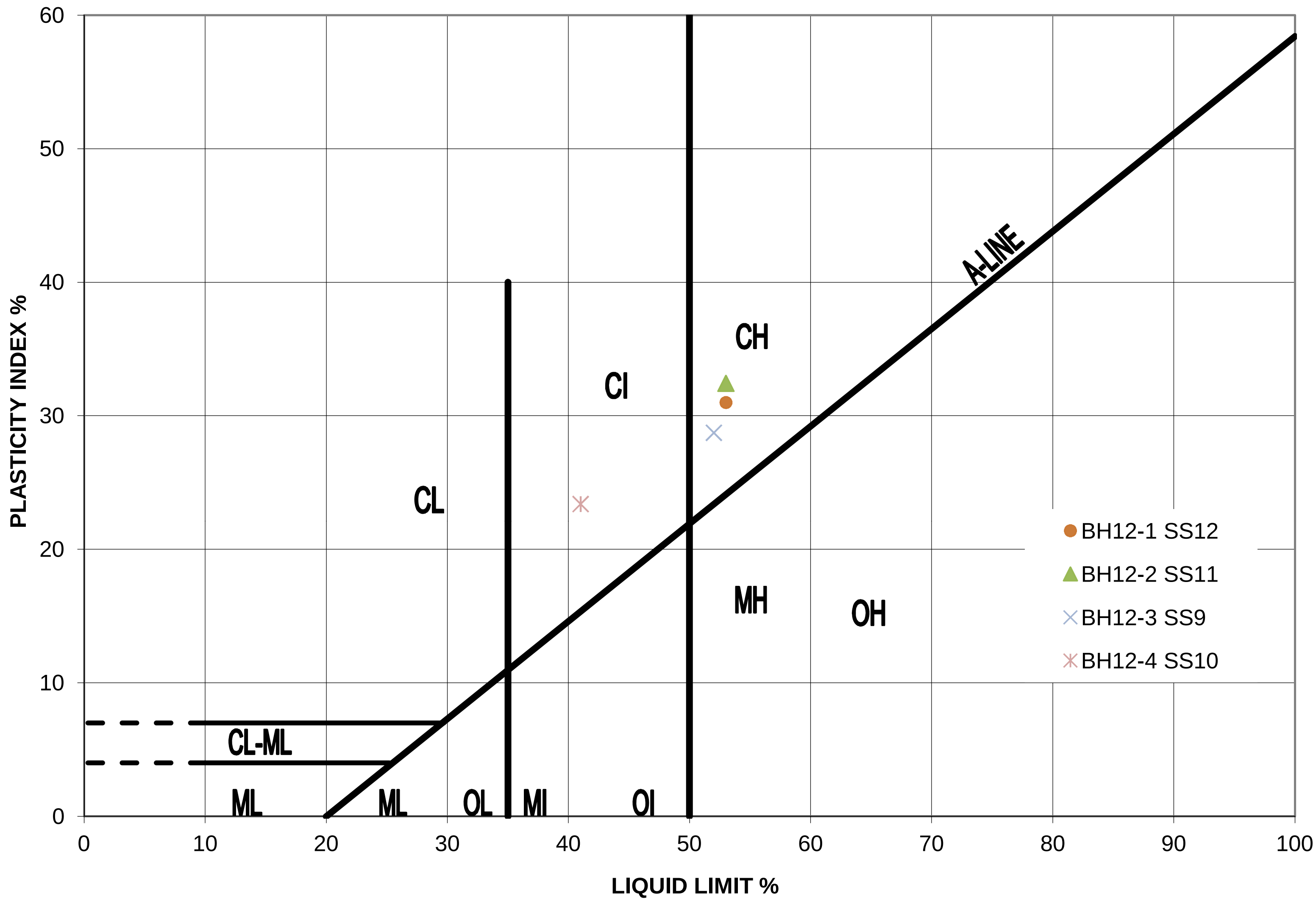
PLASTICITY CHART

(Clayey Sandy Silt)

MTO Agreement #5011-E-0010 (Detour)

Figure No.: B5

GENIVAR



PLASTICITY CHART

(Clay some Silt)

MTO Agreement #5011-E-0010 (Detour)

Figure No.: B6

Appendix C

Site Photographs

Photograph 1: Borehole BH12-1. Looking northwest.

Photograph 2: Borehole BH12-2. Looking southwest.

Photograph 3: Borehole BH12-3. Looking southeast.

Photograph 4: Borehole BH12-4. Looking southeast.

Photograph 5: Existing 4 bay garage and salt domes. Looking northwest.

Photograph 6: Existing salt dome. Location of proposed sand/salt shed. Looking southwest.