MTO Agreement No. 5011-E-0010
WO No. 2011-11032
Proposed Sand/Salt Storage Facility
Kenogami Patrol Yard
Foundation Investigation and
Design Report

Geocres No. 42A-93

March 2013

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

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March 20, 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.: 2011-11032
Proposed Sand/Salt Storage Facility – Kenogami Patrol Yard
Foundation Investigation and Design Report (Geocres No. 42A-93)

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) Kenogami Patrol Yard in Kenogami, 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 Kenogami Patrol Yard, located on Highway 11, 0.5 kilometres north of the junction of Highway 11 and Highway 66 in Kenogami, 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 Kenogami 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 Kenogami Patrol Yard (site) is located 0.5 kilometres north of the junction of Highway 11 and Highway 66 in the Township of Eby, Ontario. A Site Plan is included as Drawing 1 and colour photographs of the site are included in Appendix C.

The site is level along the east and west sides, and gently slopes toward the centre to a ditch that drains along the north perimeter of the site to a swampy area in the east. There is ponded water to the east of the existing garage at the time of the investigation, and an elevated laydown area to the south. Access to the site is from Highway 11 and the surrounding land uses is rural (forested area consisting of mixed deciduous and coniferous trees). No exposed bedrock was visible onsite.

The site is an operational MTO Patrol Yard, and is currently occupied by a number of structures, including:

- 8-bay garage / office;
- 1 large sand dome;
- 1 small salt dome;
- 1 well;
- 1 horseshoe pit;
- 1 tile bed;
- 2 site trailers;
- 1 oil / water separator; and
- 1 above ground diesel fuel storage tank.

There is a paved driveway from Highway 11 to the garages and extending back to the sand / salt domes.

2.2 Regional Geology

Two different map sources were consulted to determine the regional geology in the Kenogami area: i) Geology and Map of Ontario published by the Ministry of Northern Development and Mines (Map 2543 east Central Sheet) ii) Miscellaneous Release Data 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 between a bedrock knob and a bedrock ridge. Local soil deposits are comprised of stony till underlain by ridged bedrock terrain. The local bedrock is Precambrian age and reportedly consists of mafic intrusive and clastic metasedimentary rocks. Conglomerate metasedimentary rock was encountered in two of the boreholes in the current site investigation, thereby confirming the actual bedrock types below the site and proposed structure.

3. Historic Report Review

Two (2) previous geotechnical reports for the Kenogami Patrol yard were obtained from the MTO Geocres Library in Downsview, Ontario. The first report, entitled '*Foundation Investigation Report for W.P. 24-82-02, Site 47-009, Blanche River Bridge in Kenogami, Highway 11, District 14, New Liskeard'* (Geocres 42A-34) was completed in 1982 as part of a foundation investigation for the proposed bridge replacement near the site. The second report*, entitled 'Final Foundation Investigation Report – Culvert Station 15+675 – TWP. of Eby, GWP 162-98-00 MEL SITE A"* (Geocres 42A-80) was completed in 2010 as part of a subsurface investigation for the replacement of a single 610 millimetre (mm) diameter culvert.

The geotechnical investigation conducted in 1982 was completed at the Blanche River Highway 11 crossing. Work was comprised of sampling seven (7) boreholes supplemented by 13 dynamic cone penetration tests (DCPT). The soil stratigraphy at the site consisted of a 1.7 m to 5.4 m thick silty sand and silt with occasional silty clay, underlain by bedrock. An average SPT N value of 2 per 300 mm was recorded in the top 1.5 m of the soil profile, increasing to 60 below. Bedrock was cored at all seven (7) locations at elevations between 295.9 metres above sea level (mASL) and 300.2 mASL, and was described as a slate like material with igneous and metamorphic rock fragments.

The geotechnical investigation conducted in 2010 consisted of sampling three (3) boreholes supplemented by the same number of DCPT. The soil stratigraphy at the site consisted of approximately 0.3 m to 0.8 m of peat, underlain by silt, silty clay, sand, and embankment fill. SPT N values between 6 and 44 blows per 300 mm were recorded on the silt layer, while SPT N values between 62 blows per 300 mm and 88 blows per 250 mm were recorded in the sand layer. No bedrock was encountered in the boreholes. Groundwater was observed in the boreholes at the time of the investigation, and elevations were recorded as 306.3 mASL and 306.4 mASL.

4. Investigation Procedures

4.1 Subsurface Investigation

A borehole investigation was performed at the subject site between September 17 and September 18, 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 by the Terms of Reference for the assignment.

MTO minimum requirements for the borehole investigation outlined a maximum drilling depth of 10.0 m, unless refusal was encountered at shallower depth, or justification for deeper drilling was authorized by the MTO Project Manager. Augering in boreholes BH12-1 to BH12-4 was terminated at depths ranging between 3.7 m to 7.9 m, on presumed bedrock or very dense glacial till material. Bedrock was core sampled at boreholes BH12-2 and BH12-3. Dynamic Cone Penetration Tests (DCPT's) were driven to refusal at borehole BH12-4, which occurred at 5.6 m below ground surface (mbgs).

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 subsequently converted to MTO standard coordinates (Northings and Eastings). Borehole elevations were surveyed to a temporary benchmark: an anchor nail set in the asphalt located east of borehole BH12-3 was used as a temporary benchmark with an elevation of 100.00 m. Borehole elevations and coordinates are shown on Drawing 1, and are provided on the borehole logs 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 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, 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 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.

DCPT's were completed below 3.7 m depth in borehole 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.

NQ-size coring equipment (47.6 mm diameter) was used to obtain 2.3 m and 3.2 m long bedrock core samples at boreholes BH12-2 and BH12-3, respectively. Core recovery and rock quality index properties were determined by field inspection. Core samples were placed in labelled core boxes for transport, future reference and storage.

All boreholes 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. 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) / Relative Elevation (m)	Dynamic Cone Penetration Test Depth (m)	Comment
BH12-1	7.9/ 92.1	-	-
BH12-2	5.1/ 94.8	-	Cored into bedrock from 5.1 m to 7.4 m below existing grade
BH12-3	5.7/ 94.2	-	Cored into bedrock from 5.7 m to 8.9 m below existing grade
BH12-4	3.7 / 96.2	3.7 m to 5.6 m	-

Note: Elevations are relative to benchmark described above on page 4-1.

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 – Kenogami Patrol Yard

Test	ASTM Standard	Number of Samples
Natural Moisture Content	ASTM D2216	22
Particle Size Analysis	ASTM D422	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 is 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 granular fill, overlying compact to dense glacial till consisting mainly of sand some silt to silty sand material. The till is underlain by bedrock, which was core sampled at boreholes BH12-2 and BH12-3. Dynamic Cone Penetration Testing (DCPT) advanced in borehole BH12-4 from a depth of 3.7 m to 5.6 m below the ground surface indicates the same very dense deposit (presumed till). Descriptions of the individual soil units are provided in the following subsections.

5.1.1 Asphalt Pavement

A 65 mm thick layer of asphaltic concrete (hot laid mix) was encountered from the surface at boreholes BH12-1 and BH12-2.

5.1.2 Granular Fill

Below the asphalt pavement at boreholes BH12-1 to BH12-2, and at the surface of boreholes BH12-3 and BH12-4, a granular fill layer was encountered consisting of gravelly sand to sand with some gravel, extending to depths of between 0.8 mbgs at borehole BH12-2 and 1.4 mbgs at boreholes BH12-1, BH12-3 and BH12-4.

A laboratory particle size distribution analysis for a sample of 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) - 20 %
- Sand (0.075 mm to 4.75 mm size) - 75 %
- Silt and Clay (less than 0.075 mm size) - 5 %

Standard Penetration Test results (N Values) recorded in the fill layer ranged between 5 and 15 blows per 305 mm of penetration, indicating loose to compact relative density.

Laboratory determined moisture contents ranged between 9 % and 12 % for samples of the fill, indicating moist material.

5.1.3 Till

Underlying the granular fill layer in boreholes BH12-1 to BH12-4, a glacial till material was encountered extending to depths (metres below ground surface; mbgs) and relative elevations shown below:

Borehole No.	Depth to Bottom of Till Layer (Relative Elevation)
BH12-1	7.9 mbgs (92.1 m)
BH12-2	5.1 mbgs (94.8 m)
BH12-3	5.7 mbgs (94.2 m)
BH12-4	3.7 mbgs (96.2 m)

The texture of the till layer was predominantly sand with some silt to silty sand, with a trace to some gravel and clay. Boreholes BH12-2 and BH12-3 were terminated on the bedrock surface. Boreholes BH12-1 and BH12-4 were terminated at depths 7.9 mbgs and 3.7 mbgs, respectively, due to auger refusal. A DCPT was advanced at BH12-4, extending to 5.6 mbgs.

Laboratory particle size distribution analyses for six (6) samples of the till material were completed, and results are summarized below and shown in Figures B2 and B3 of Appendix B:

- Gravel (greater than 4.75 mm size) - 6 % to 18 %
- Sand (0.075 mm to 4.75 mm size) - 51 % to 78 %
- Silt and Clay (less than 0.075 mm size) - 13 % to 41 %

Standard Penetration Test results (N Values) recorded in the till deposit ranged between 5 and 38 blows per 305 mm of penetration, indicating loose to dense (generally compact) relative density.

The DCPT performed at borehole BH12-4 extended to refusal, defined by MTO as 100 blows per 305 mm of penetration, at a depth of 5.6 mbgs (relative elevation 94.3 m) in very dense material.

5.1.4 Bedrock

Bedrock core samples were taken in boreholes BH12-2 and BH12-3, and were 2.3 m and 3.2 m long, respectively. Drilling at borehole BH12-2 was terminated at 7.4 mbgs (relative elevation 92.5 m) and BH12-3 was terminated at 8.9 mbgs depth (relative elevation 91.0 m). Photographs of the bedrock cores are included in Appendix C.

Descriptions of the bedrock are provided in Table 5-1 and in the borehole logs. Total Core Recovery (TCR) ranged from 92 % to 100 %. Rock Quality Designation (RQD) values for the core samples in borehole BH12-2 ranged from 45 % to 87 %, which is described as poor to good rock quality. The RQD values for borehole BH12-3 ranged between 0 % and 67%, described as very poor to fair rock quality.

Table 5-1: Rock Core (RC) Description, RQD, and Recovery Data

BH	RC #	Depth (m)	TCR (%)	RQD (%)	Depth (m)	Description
12-2	1 2	5.1 – 5.9 5.9 – 7.4	100 100	45 87	5.1 –7.4	CONGLOMERATE, grey with subangular to subrounded clasts in fine-grained matrix, occasional secondary quartz carbonate.
12-3	1 2 3	5.7 –6.5 6.5 – 7.5 7.5- 8.9	100 92 100	0 67 60	5.7 – 8.9	CONGLOMERATE, grey with subangular to subrounded clasts in fine-grained matrix, occasional secondary quartz carbonate

5.2 Groundwater Conditions

Groundwater conditions were observed in the open boreholes upon completion of drilling. Results are summarized in Table 5-2.

Table 5-2: Summary of Groundwater Levels

Location	Measured Groundwater Depth mbgs (relative elevation, m)	Date Measured
BH12-1	2.4 (97.6)	17 September 2012
BH12-2	2.5 (97.3)	18 September 2012
BH12-3	2.5 (97.4)	18 September 2012
BH12-4	2.5 (97.4)	18 September 2012

Note: mbgs = metres below ground surface.

Based on the water level measurements, moisture conditions, and changing color and/or staining 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 at 2.5 m below ground surface (estimated relative elevation 97.6 m to 97.3 m). It should be noted that groundwater levels may fluctuate seasonally and in response to climatic conditions.

6. Geotechnical Design Considerations

The proposed sand/salt storage facility at Kenogami Patrol Yard will replace an existing salt dome, and will have a rectangular footprint of approximate dimensions 18.3 m × 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 editions of the Canadian Highway Bridge Design Code (CHBDC) and 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. A compact sand to silty sand till layer was encountered beneath the fill, extending to the presumed surface of the bedrock at relative elevations ranged between 94.8 m to 92.1 m. Conglomerate bedrock was core sampled at boreholes BH12-2 and BH12-3.

Based on measurements at the time of the field investigation, the groundwater level was inferred to be at 2.5 mbgs (relative elevation 97.6 m to 97.3 m).

6.1 "Red Flag" Conditions and NSSP's

Groundwater within relative elevation 97.6 m to 97.3 m presents a possible challenge for foundation construction. Depending on the depth of the foundation and related service excavations, dewatering requirements may range from simple pumping from filtered sumps and ditching, to use of wellpoints. Protective measures are required to maintain adequate excavation stability and foundation bearing capacity; mitigation measures for groundwater are further discussed in Section 6.8.

The presence of cobbles and boulders should be anticipated within the sand to silty sand till deposit, and may cause difficulties during the excavation and/or possible installation of shoring units. If boulders extend below founding level and are dislodged by an excavator, the soils around the boulders will become disturbed. In that case, the boulders will need to be fully removed (and not pushed back into place) and voids should be filled with concrete.

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

NSSP 1. A high groundwater table, generally within 2.5 m of the ground surface presents construction challenges for foundation construction. Groundwater may have to be pumped from construction excavations. Wet soil 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.

NSSP 2. If boulders are encountered and/or removed during excavation and/or shoring procedures, the Contractor shall ensure that the integrity of the disturbed soil is restored so that there are no void, loose zones present. Unshrinkable concrete fill shall be used when necessary.

6.2 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
- 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 Sand to Silty Sand Till Layer	Lower cost, lower foundation capacity versus deep foundation, requires greater effort to control groundwater and prevent subgrade disturbance, larger foundation settlement versus deep foundation design.	Good, provided that construction practices are used to minimize soil disturbance.	Risk of groundwater and subgrade disturbance; subexcavation may be needed; pumping may be required depending on seasonal conditions; difficulties during construction if excavation will be within the water bearing till layer; shoring may be necessary	Recommended, provided good construction practices are used. Foundation must be below frost or insulated.
Slab-on-Grade	Medium cost, medium geotechnical resistance, insulation required, larger foundation settlement versus deep foundation.	Good.	Removal of shallow deleterious material and larger excavation/disturbed area required for insulation component.	Not Recommended due to economic and constructability reasons. Insulation required and must extend beyond structure.
Drilled and Cast-in-Place Concrete Foundation on Bedrock	High bearing resistance, negligible settlement, and protection of subgrade against cave in is required, high cost. Possibility of encountering cobbles and boulders during drilling.	Good	Must extend to bedrock. Liners may be required. Construction difficulties if boulders encountered during drilling	Not Recommended due to economic and constructability reasons.
Steel H Piles on Bedrock	High bearing resistance, negligible settlement, protection of subgrade against disturbance not as critical as for shallow foundations, high cost. Possibility of encountering cobbles and boulders during driving and thus needing to pre-drill pile locations, use lower pile capacity, and/or drive additional piles.	Good	Must extend to deeper competent material or bedrock. Vibrations and/or soil disturbance may be an issue for nearby structures.	Not Recommended due to economic and constructability reasons.

6.3 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 high density foam insulation is required for frost protection of foundations. In consideration of the depth of required soil cover for frost protection and the groundwater levels at the site, it is assumed that a significant grade raise around the structure area is not an option.

6.4 Preferred Foundation Option

Based on the results of this investigation, the proposed sand/salt storage facility should be supported on strip footings founded on sand to silty sand till layer, with a recommended highest founding level at 1.4 m below existing grade (relative elevation 98.3 m to 98.6 m). Permanent thermal insulation (e.g. DOW Styrofoam HI60 or equivalent) shall be installed for frost protection according to manufacturer's requirements. Insulation shall be at least 50 mm thick. The other option (not preferred) is to lower the foundation level below 2.2 m depth (elevation 97.5 m to 97.8 m) for frost protection purposes, but this option may encounter wet soils near the base of the excavation, depending on conditions at the time of construction.

The following geotechnical resistances are appropriate for insulated strip footings with 0.9 m minimum width, constructed in the undisturbed sand till layer at depth 1.4 m below the ground surface (elevation 98.3 to 98.6 m):

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

The above geotechnical design resistances may also be used if foundation grade is lowered to 2.2 m below the ground surface, at relative elevations between 97.5 m to 97.8 m, to provide the required frost protection. However, the Geotechnical Engineer must confirm that soil conditions are adequate, otherwise the shallower insulated design should be used.

The Geotechnical Resistance at Serviceability Limit State (SLS) value is based on maximum total and differential settlements of 25 mm and 20 mm, respectively.

Existing fill materials overlying the native till 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 wet till material may be disturbed by foot traffic. In this case, a minimum 50 mm thick mud mat should be installed immediately after subgrade inspection and approval.

6.5 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 coefficient of friction, tan δ, may be taken as 0.44 for cast in place concrete footings constructed on undisturbed compact sand till. In accordance with CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistances. Resistance to lateral loads could be increased by constructing a shear key at the bottom of footings. 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 excessively disturbed by construction activities.

6.6 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 5% passing the 0.75 mm sieve as per requirement of OPSS 1010 and its Amendment No. 110S13) and the provision of drain pipes and weep holes to prevent hydrostatic pressure build-up against the walls.

Computation of horizontal 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 that adequate drainage is provided.

Should temporary shoring be required to support excavations, shoring systems should be carried out in accordance with the OPSS 539 and 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.

Retaining and shoring walls below grade may 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
Compact to Dense Sand Till	0.30	0.45	3.4	19.5

6.7 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 ‘C’ (very dense soil and rock) be applied for structural design at this site.

Seismic information for the Kenogami 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	Kenogami	Source
Site Class	C	2006 Ontario Building Code Table 4.1.8.4.A
$S_a(0.2)$	0.232	2005 National Building Code Seismic Hazard Calculation
$S_a(1.0)$	0.056	2005 National Building Code Seismic Hazard Calculation
F_a	1.0	2006 Ontario Building Code Table 4.1.8.4.B
F_v	1.0	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 till soils, dynamic and static liquefaction are not expected to be a concern at this site.

6.8 Dewatering and Drainage

It is expected that the foundation excavations to approximately relative elevation 98.3 m to 98.6 m will not encounter significant groundwater. Above the groundwater level, localized lenses of perched groundwater may exist within the till layer may exist, but amounts should be minor.

If groundwater is encountered during construction, gravity drainage or pumping from filtered sumps located at the base of the excavations may be required to provide groundwater control during foundation excavations. Surface water runoff should be directed away from the excavations at all times. Dewatering procedures should follow the requirements and specifications of OPSS 517 and groundwater control requirements should be planned accordingly by the Contractor prior to construction.

Depending on the construction and dewatering procedures to be used, the Contractor should obtain a Permit to Take Water (PTTW) under Section 34 of the Ontario Water Resources Act if pumping rates will exceed 50,000 L/day. It is unlikely that a PTTW will be required for the recommended construction procedure.

6.9 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
Compact sand till	Type 3	Type 4

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 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 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 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	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
Laboratory Medium Complexity	Marijana Manojlovic, B.Sc. Golder Associates	Mississauga, ON	905-567-4444
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, P. Eng. GENIVAR Inc.	Newmarket, ON	905-853-3303

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.
Geotechnical Engineer

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

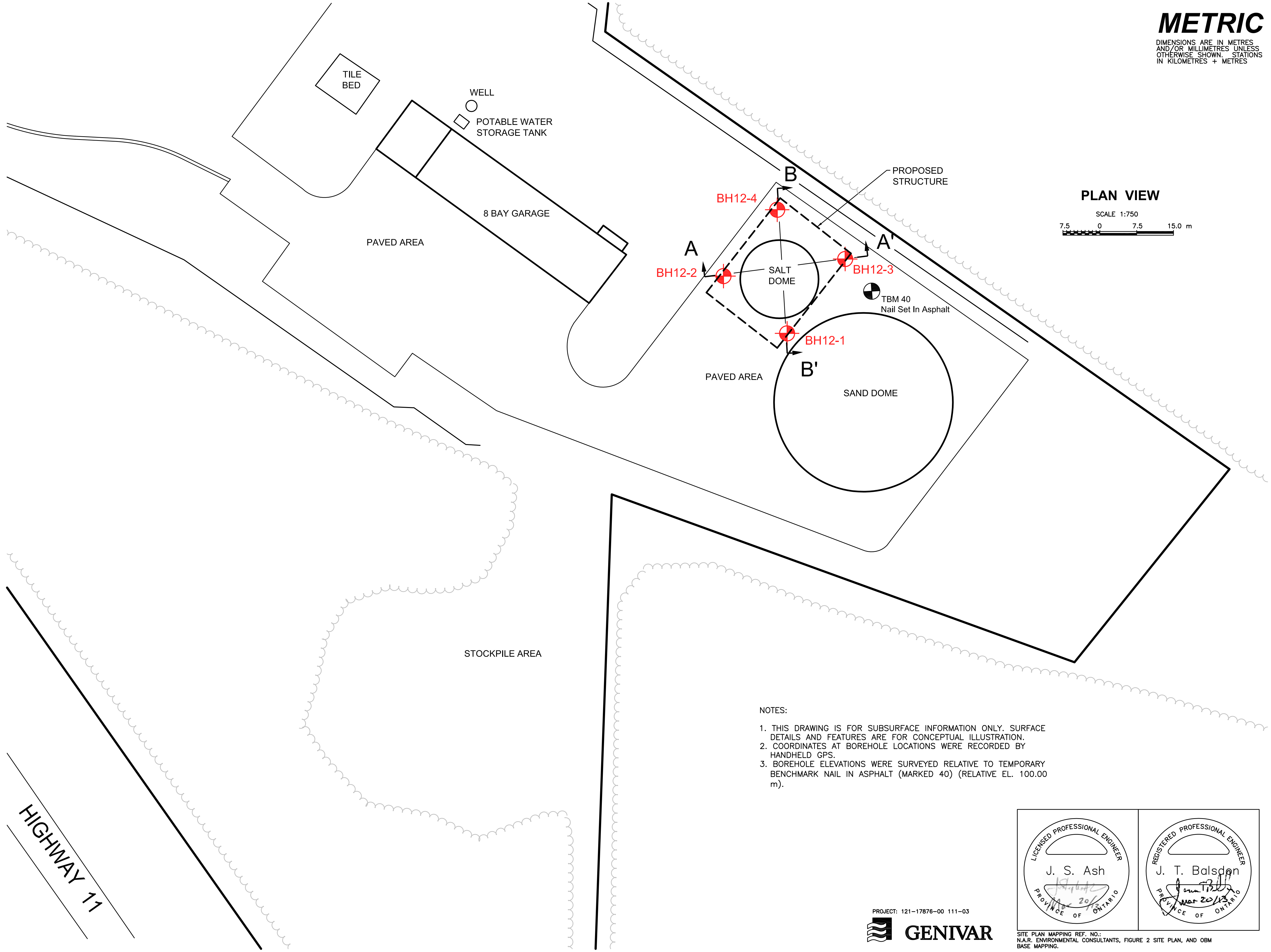
Reviewed by:

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

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

PLAN VIEW

SCALE 1:750

7.5 0 7.5 15.0 m

Agreement No.: 5011-E-0010

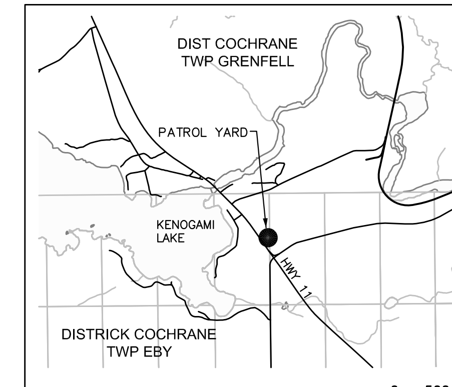
WO No.: 2011-11032

BOREHOLE LOCATION PLAN

PROPOSED SAND/SALT STORAGE FACILITY
KENOGAMI PATROL YARD
HIGHWAY 11

Client: MTO - Northeastern Region

DRAWING 1



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	99.995	5327387.5	560390.5
12–2	99.833	5327399.1	560377.6
12–3	99.985	5327402.6	560402.4
12–4	99.891	5327412.6	560388.5

– 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 NAIL IN ASPHALT (MARKED 40) (RELATIVE EL. 100.00 m).

PROJECT: 121–17876–00 111–03

GENIVAR

LICENSED PROFESSIONAL ENGINEER J. S. Ash PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER J. T. Balsdon PROVINCE OF ONTARIO

SITE PLAN MAPPING REF. NO.:
N.A.R. ENVIRONMENTAL CONSULTANTS, FIGURE 2 SITE PLAN, AND OBM BASE MAPPING.

REVISIONS	DATE	BY	DESCRIPTION

GEOCRES No. 42A–93

HWY No 11			DIST COCHRANE
SUBM'D --	CHECKED JSA	DATE MARCH 2013	SITE --
DRAWN PLB	CHECKED --	APPROVED --	DWG --

METRIC

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

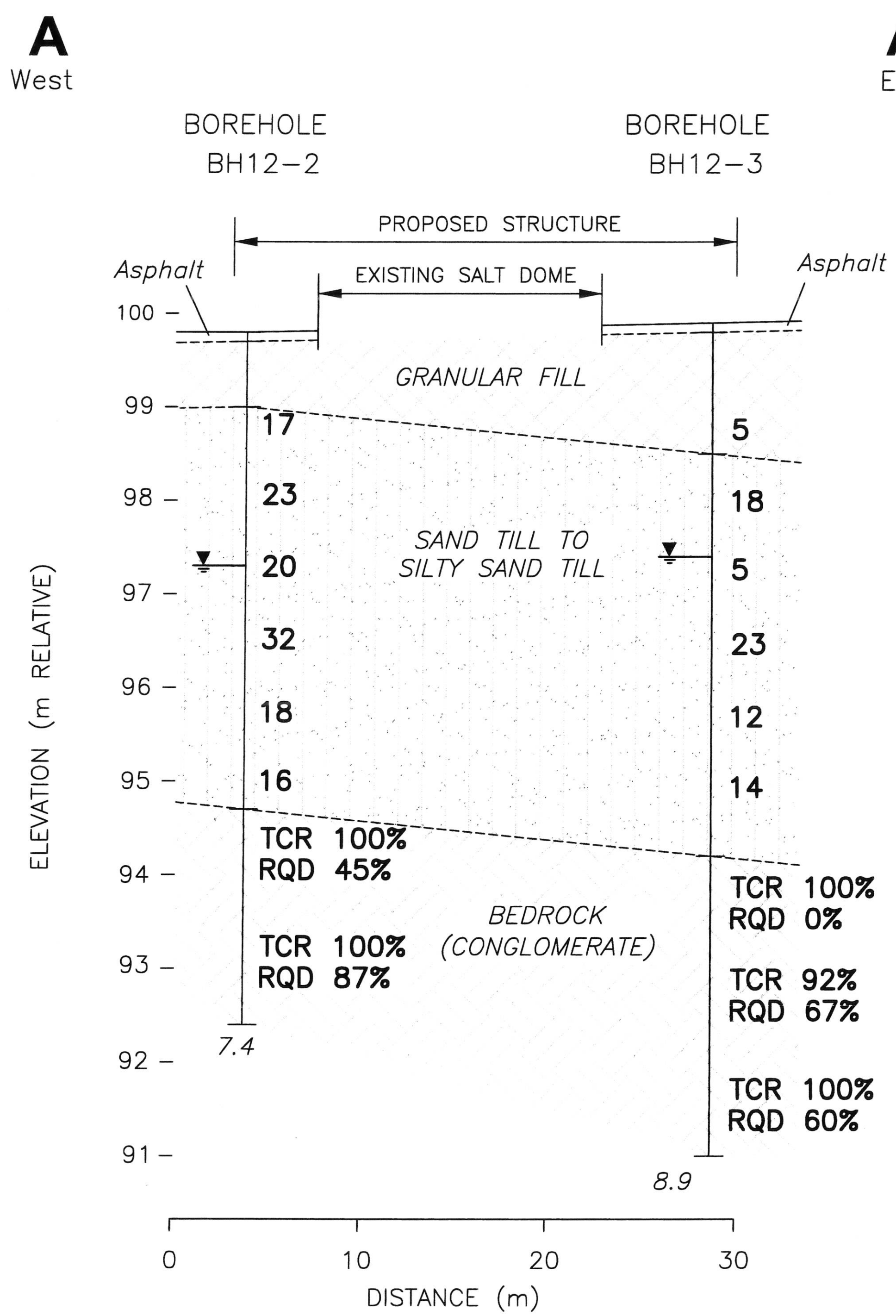
Agreement No.: 5011-E-0010
WO No.: 2011-11032

SOIL STRATA
PROPOSED SAND/SALT STORAGE FACILITY
KENOGAMI PATROL YARD
HIGHWAY 11

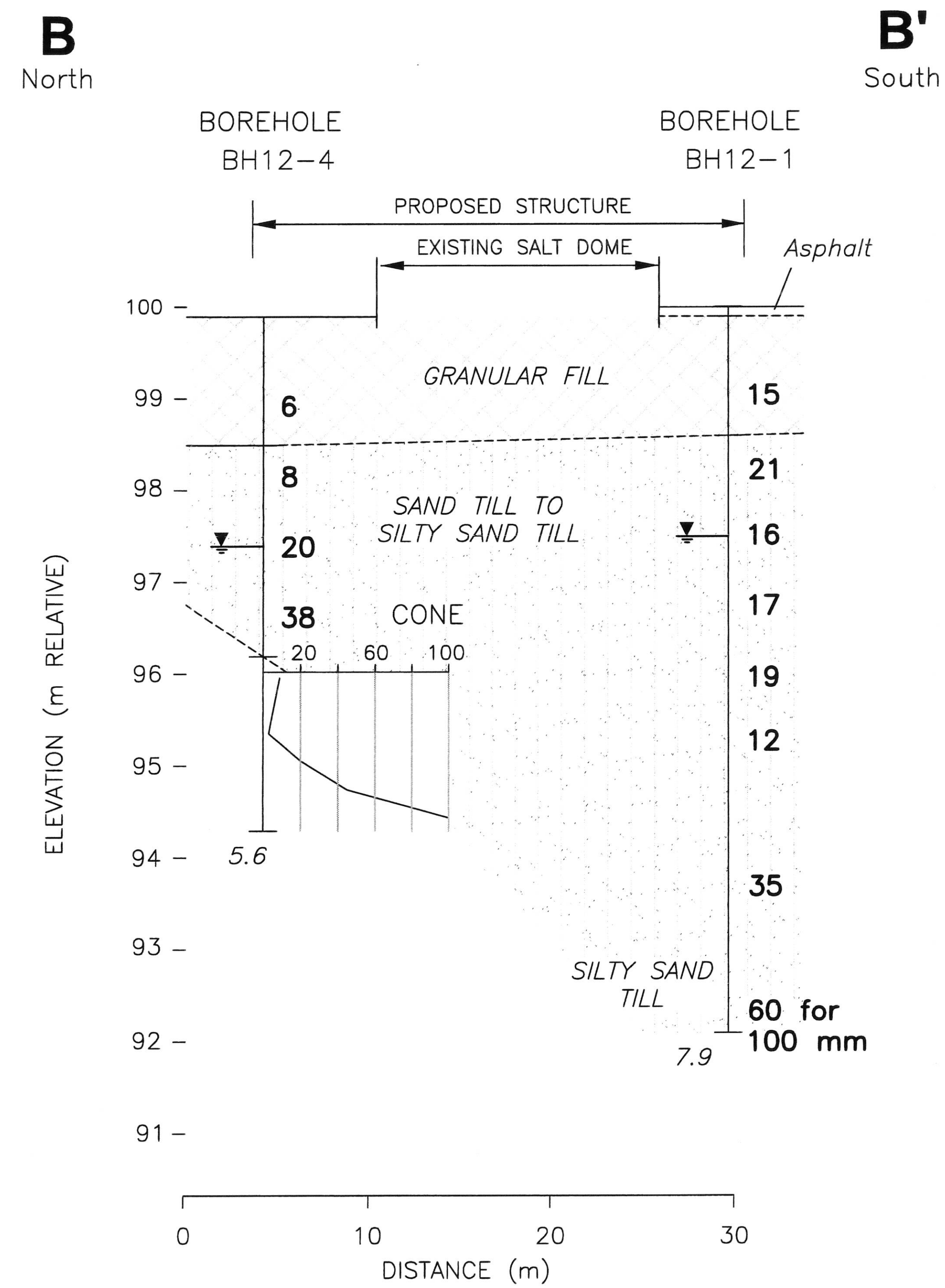
Client: MTO - Northeastern Region

DRAWING 2

CROSS SECTION A-A'



CROSS SECTION B-B'



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 NAIL IN ASPHALT (MARKED 40) (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	99.995	5327387.5	560390.5
12-2	99.833	5327399.1	560377.6
12-3	99.985	5327402.6	560402.4
12-4	99.891	5327412.6	560388.5

– 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.

LICENSED PROFESSIONAL ENGINEER
J. S. Ash
Feb 20/13
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
J. T. Balsdon
Feb 20/13
PROVINCE OF ONTARIO

PROJECT: 121-17876-00 111-03

GENIVAR

SITE PLAN MAPPING REF. NO.:
N.A.R. ENVIRONMENTAL CONSULTANTS, FIGURE 2 SITE PLAN, AND OBM BASE MAPPING.

REVISIONS DATE	BY	DESCRIPTION

GEOCRES No. 42A-93

HWY No 11			DIST COCHRANE
SUBM'D --	CHECKED JSA	DATE FEBRUARY 2013	SITE --
DRAWN PLB	CHECKED --	APPROVED --	DWG --

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.

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	TRC =	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, 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 $\frac{xBlows}{mm}$

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: KENOGAMI PATROL YARD N 5327387.5; E 560390.5

ORIGINATED BY: DCL

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

COMPILED BY: JW

DATUM: GEODETIC

DATE: 9.17.12 - 9.17.12

CHECKED BY: RK

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
100.0	
99.9 / 0.1	**ASPHALT:** 65 mm THICK
	GRANULAR FILL: GRAVELLY SAND BROWN, COMPACT, MOIST
98.6 / 1.4	**SAND TILL:** FINE SAND SOME SILT TO SILTY SAND, TRACE TO SOME GRAVEL, TRACE CLAY BROWN, COMPACT TO DENSE, SATURATED.
92.1 / 7.9	END OF BOREHOLE AUGER REFUSAL ON PRESUMED BEDROCK

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AS		
2	SS	15	
3	SS	21	
4	SS	16	
5	SS	17	
6	SS	19	8 51 38 3
7	SS	12	
8	SS	35	
9	SS	60 for 150 mm	7 62 (31)

GROUND WATER CONDITIONS

ELEVATION SCALE: 99, 98, 97, 96, 95, 94, 93

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

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

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

RECORD OF BOREHOLE No BH12-2

METRIC

LOCATION: KENOGAMI PATROL YARD N5327399.1; E 560377.6 — ORIGINATED BY: DCL

BOREHOLE TYPE: CONTINUOUS FLIGHT HOLLOW STEM AUGERS WITH SPT AND DCPT — COMPILED BY: JW

DATUM: GEODETIC — DATE: 9.18.12 - 9.18.12 — CHECKED BY: RK

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
99.8						
99.7 / 0.1	**ASPHALT:** 65 mm THICK					
	GRANULAR FILL: GRAVELLY SAND BROWN, MOIST		AS			
99.1 / 0.8	**SAND TILL:** FINE SAND, SOME SILT TO SILTY SAND, TRACE TO SOME GRAVEL, TRACE CLAY BROWN TO GREY COMPACT TO DENSE, MOIST TO SATURATED - SATURATED BELOW 2.5 m DEPTH	1	SS	17		
		2	SS	23		18 69 (13)
		3	SS	20	▼	
		4	SS	32		
	- BECOMING GREY	5	SS	18		11 68 (21)
		6	SS	16		
94.8 / 5.1	**BEDROCK:** GREY CLASTIC METASEDIMENTARY ROCK (CONGLOMERATE) WITH GREY,PINK REDDISH SUBANGULAR TO SUBROUNDED CLASTS UP TO 3 CM IN FINE GRAINED MATRIX. FRACTURES AT 60 DEGREES TO CORE AXIS, WHITE QUARTZ-CARBONATE VEINS RUNNING ALONG CORE AXIS.	1	RC	TCR = 100%		RQD = 45%
		2	RC	TCR = 100%		RQD = 87%
92.5 / 7.4	END OF BOREHOLE					

ELEVATION SCALE: 99, 98, 97, 96, 95, 94, 93

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/m^3

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

RECORD OF BOREHOLE No BH12-3

METRIC

LOCATION: KENOGAMI PATROL YARD N 532742.6; E 560402.4
ORIGINATED BY: DCL

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

DATUM: GEODETIC
DATE: 9.18.12 - 9.18.12
CHECKED BY: RK

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
99.9 / 0.0	**SAND FILL:** SAND, SOME GRAVEL BROWN, LOOSE, MOIST		AS			
		1	SS	5		
98.5 / 1.4	**SAND TILL:** FINE SAND, TRACE TO SOME SILT, TRACE GRAVEL, TRACE CLAY BROWN TO GREY, LOOSE TO COMPACT, SATURATED	2	SS	18		6 78 (16)
		3	SS	5	▼	
	- BECOMING GREY	4	SS	23		
		5	SS	12		
		6	SS	14		
94.2 / 5.7	**BEDROCK:** GREY CLASTIC METASEDIMENTARY ROCK (CONGLOMERATE) WITH GREY,PINK REDDISH SUBANGULAR TO SUBROUNDED CLASTS UP TO 2 TO 3 CM IN FINE GRAINED TO APHANITIC MATRIX FRACTURES AT 60 TO 70 DEGREES TO CORE AXIS.	1	RC	TCR = 100%		RQD = 0%
		2	RC	TCR = 92%		RQD = 67%
		3	RC	TCR = 100%		RQD = 60%
91.0 / 8.9	END OF BOREHOLE					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (99, 98, 97, 96, 95, 94, 93, 92); 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

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

RECORD OF BOREHOLE No BH12-4

METRIC

LOCATION: KENOGAMI PATROL YARD N 5327412.6; E 560388.5
ORIGINATED BY: DCL

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

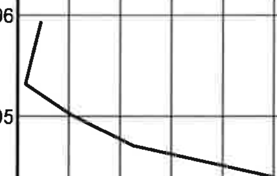
DATUM: GEODETIC
DATE: 9.18.12 - 9.18.12
CHECKED BY: RK

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
99.9 / 0.0	**GRANULAR FILL:** GRAVELLY SAND, TRACE SILT BROWN, LOOSE, MOIST	1	AS						
		2	SS	6		99			20 75 (5)
98.5 / 1.4	**SAND TILL:** FINE SAND, SOME SILT TO SILTY SAND, TRACE TO SOME GRAVEL, TRACE CLAY BROWN TO GREY, LOOSE TO DENSE, SATURATED	3	SS	8		98			
		4	SS	20	▼	97			
		5	SS	38					13 65 (22)
96.2 / 3.7	CONTINUOUS DYNAMIC CONE PENETRATION TEST BELOW 3.7 m DEPTH. NO SOIL SAMPLING COMPLETED.					96, 95			
94.3 / 5.6	END OF DCPT ON PRESUMED BEDROCK								

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



+³, ×³: Numbers refer to Sensitivity

O ³% STRAIN AT FAILURE

Appendix B

Summary of Particle Size Distribution Results (Table B1)

Particle Size Distribution Analyses (Figures B1 to B3)

Table B1: Summary of Grain Size Distribution

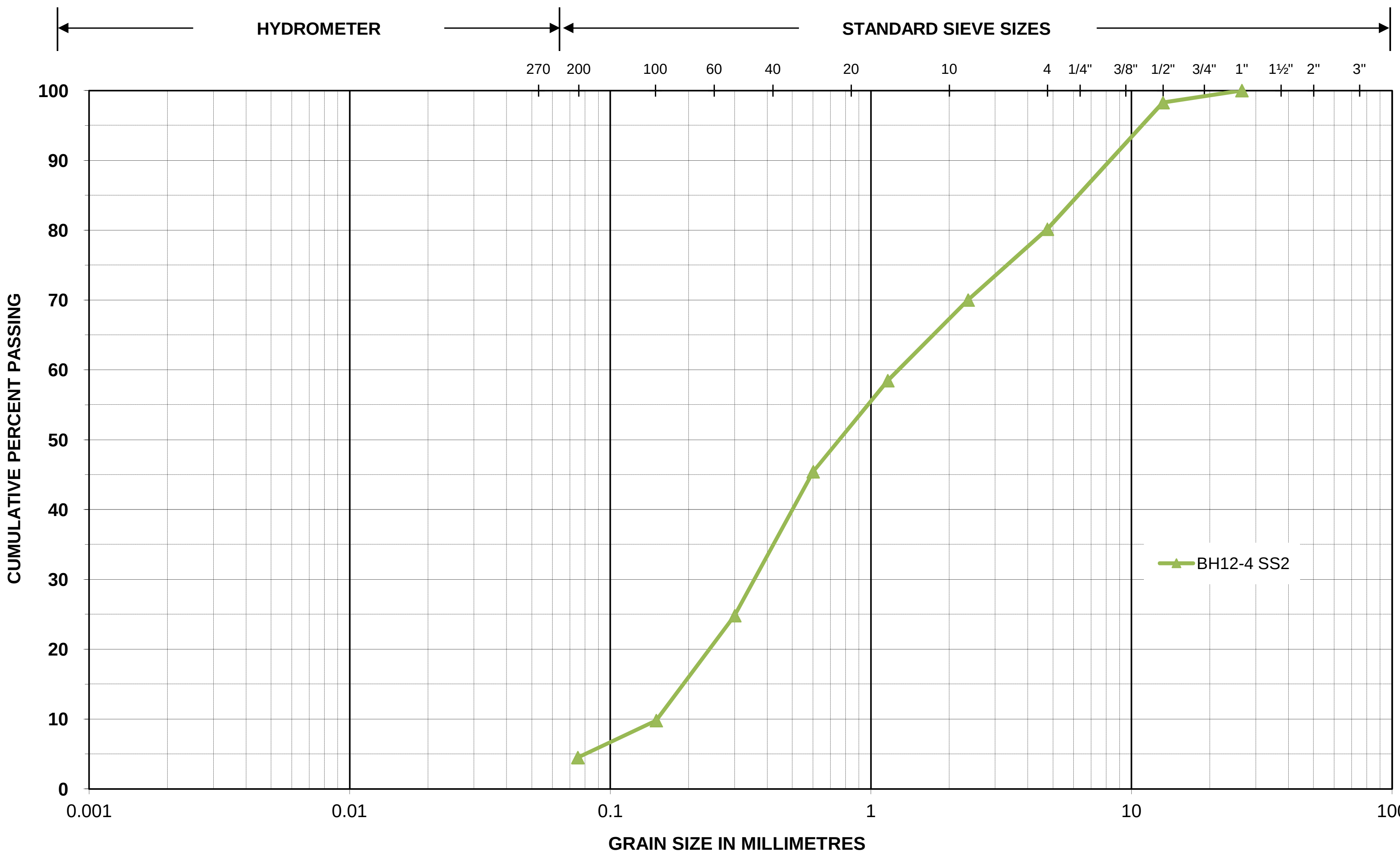
<table>
<tr><th rowspan="2">Borehole No.</th><th rowspan="2">Sample ID</th><th rowspan="2">Soil Description</th><th colspan="4">Percentage Retained (%)</th></tr>
<tr><th>Gravel</th><th>Sand</th><th>Silt</th><th>Clay</th></tr>
<tr><td>BH12-1</td><td>SS6</td><td>Silty sand, trace gravel, trace clay</td><td>8</td><td>51</td><td>38</td><td>3</td></tr>
<tr><td>BH12-1</td><td>SS9</td><td>Sand and silt, trace gravel</td><td>7</td><td>62</td><td colspan="2">31</td></tr>
<tr><td>BH12-2</td><td>SS2</td><td>Silty sand, some gravel</td><td>18</td><td>69</td><td colspan="2">13</td></tr>
<tr><td>BH12-2</td><td>SS5</td><td>Silty sand, some gravel</td><td>11</td><td>68</td><td colspan="2">21</td></tr>
<tr><td>BH12-3</td><td>SS2</td><td>Sand, some silt, trace gravel</td><td>6</td><td>78</td><td colspan="2">16</td></tr>
<tr><td>BH12-4</td><td>SS2</td><td>Gravelly sand, trace silt</td><td>20</td><td>75</td><td colspan="2">5</td></tr>
<tr><td>BH12-4</td><td>SS5</td><td>Silty sand, some gravel</td><td>12</td><td>64</td><td colspan="2">22</td></tr>
</table>

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

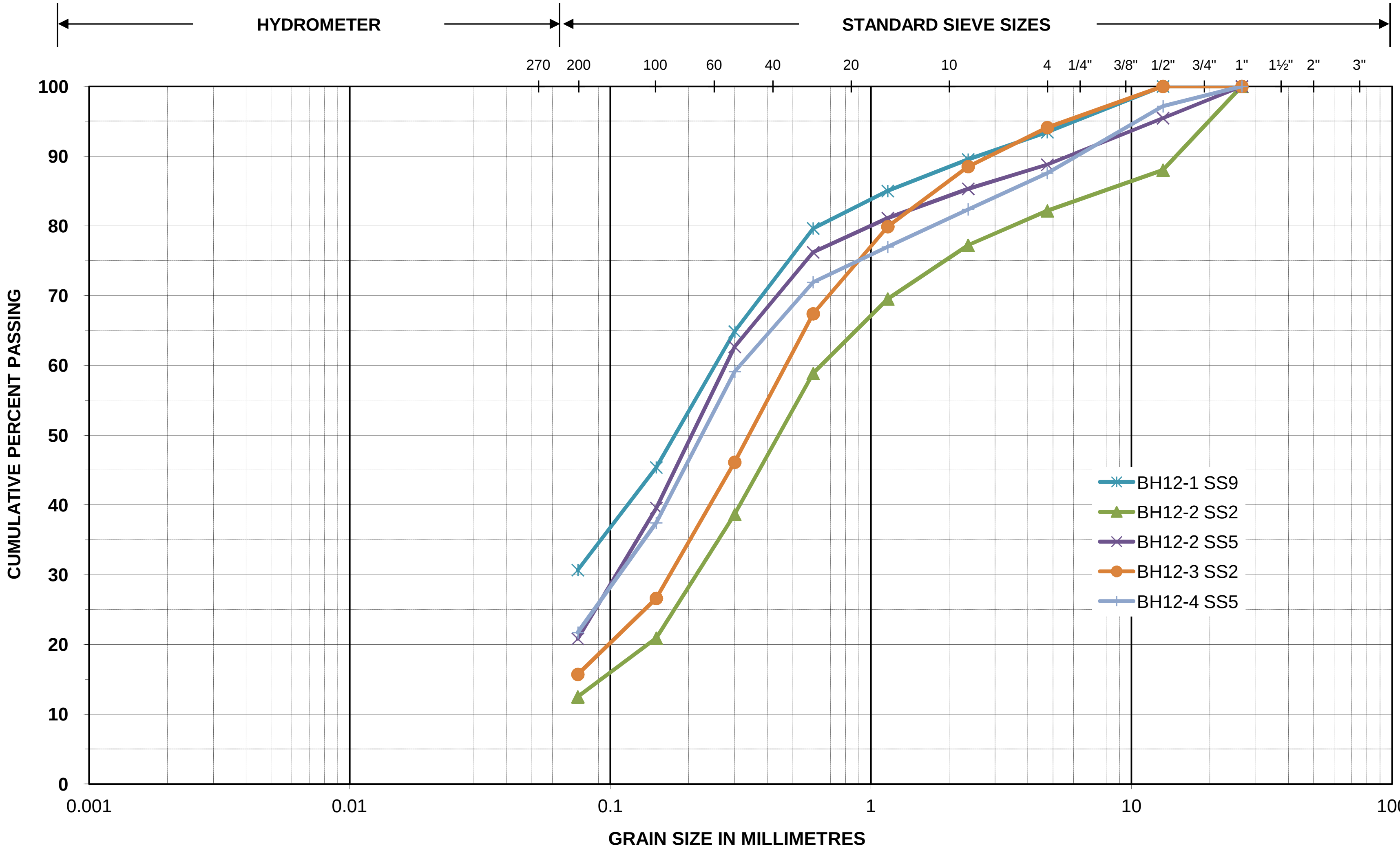


Unified Classification System

SILT AND CLAY	SAND	GRAVEL

Project Name:	MTO Agreement # 5011-E-0010 - Kenogami	**Project No.:**	121-17876-00	**Figure No.:**	**B1**
Remarks:	Gravelly sand, trace silt				

PARTICLE SIZE DISTRIBUTION



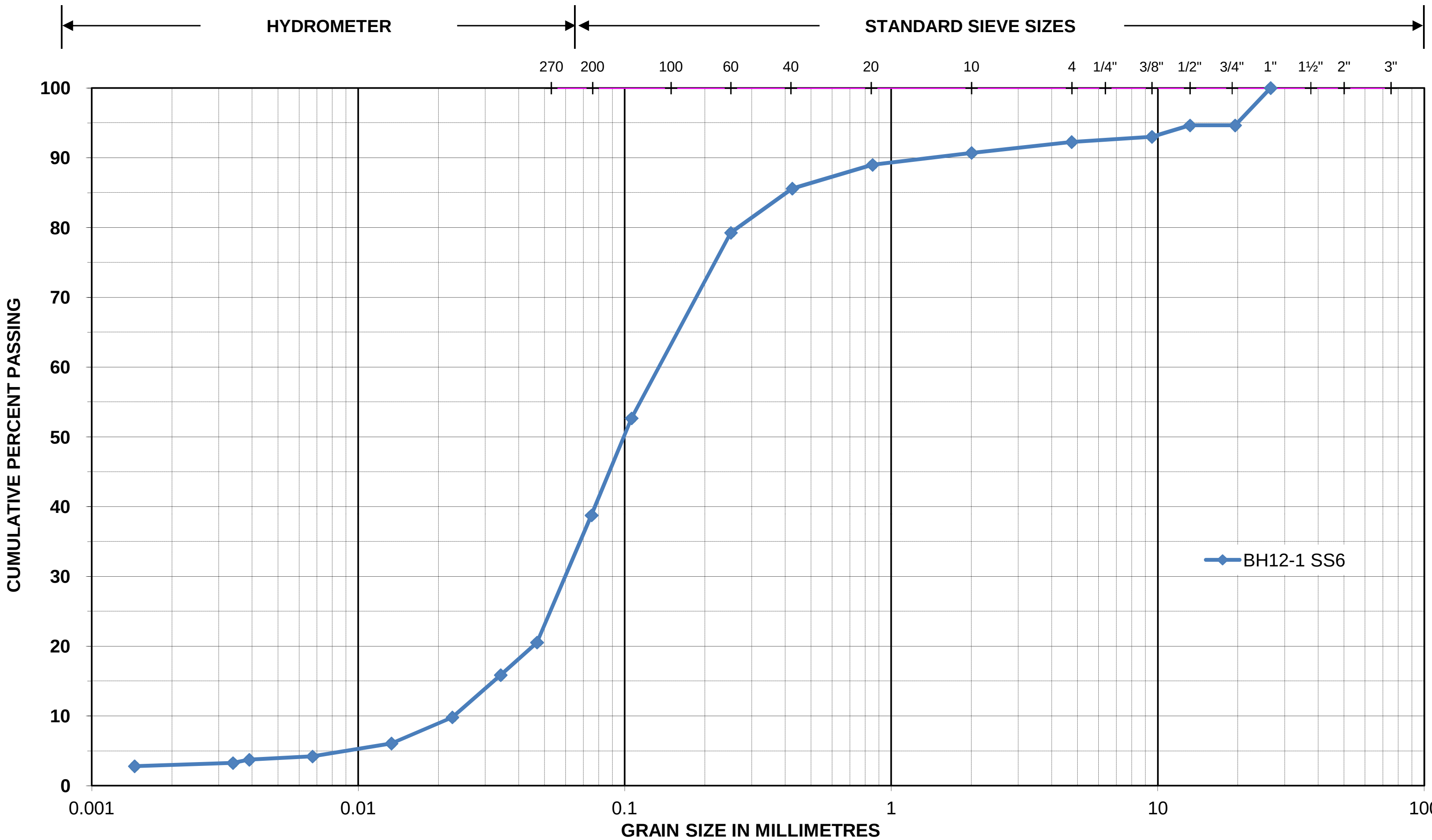
Unified Classification System

SILT AND CLAY	SAND	GRAVEL

Project Name:	MTO Agreement # 5011-E-0010 - Kenogami	**Project No.:**	121-17876-00	**Figure No.:**	**B2**
Remarks:	Silty sand to sand and silt, some to trace gravel	**Remarks:**	Silty sand, some gravel		

PARTICLE SIZE DISTRIBUTION ASTM D422

GENIVAR



Unified Classification System

SILT AND CLAY	SAND	GRAVEL

Project Name:	MTO Agreement #5011-E-0010 Kenogami	**Project No.:**	121-17876-00	**Figure No.:**	**B3**
Remarks:	Silty sand, trace gravel, trace clay				

Appendix C

Site Photographs

Rock Core Photographs

MTO AGREEMENT #5011-E-0010
KENOGAMI PATROL YARD

Photograph 1: Borehole BH12-1. Looking northeast.

Photograph 2: Borehole BH12-2. Looking southeast.

Photograph 3: Borehole BH12-4. Looking east.

Photograph 4: Existing 8-bay garage and office. Facing west.

Photograph 5: Central drainage swale. Looking north.

Photograph 6: Existing salt (dome on left) and sand dome. Current salt dome is proposed location for sand/salt shed. Looking north.

MTO AGREEMENT #5011-E-0010
KENOGAMI PATROL YARD – ROCK CORE

Photograph 1: BH12-2 Rock Core (4.94 m to 7.33 m).

Photograph 2: BH12-2 Rock Core (4.94 m to 7.33 m).

MTO AGREEMENT #5011-E-0010
KENOGAMI PATROL YARD – ROCK CORE

Photograph 3: BH12-3 Rock Core (5.55 m to 8.85 m).

Photograph 4: BH12-3 Rock Core (5.55 m to 8.85 m).