MTO Agreement No. 5011-E-0010
WO No. 2011-11040
Proposed Sand/Salt Storage Facility
Priddle Rock Patrol Yard
Foundation Investigation and Design Report

Geocres No. 41G-13

January 2013

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

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

Project No. 121-17876-00

Project No. 121-17876-00

January 15, 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. 2011-11040
Proposed Sand/Salt Storage Facility – Priddle Rock Patrol Yard
Foundation Investigation and Design Report (Geocres No. 41G-13)

Dear Mr. Perron:

We are pleased to submit our final Foundation Investigation and Design Report for the proposed Sand/Salt Storage Facility at the Ontario Ministry of Transportation Northeastern Region (MTO) Priddle Rock Patrol Yard on Manitoulin Island, Ontario. A borehole and laboratory testing program was conducted to assess soil, bedrock, and groundwater conditions at the site and to 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 geotechnical investigation for the proposed construction of a sand/salt storage facility at the Priddle Rock Patrol Yard, located on Highway 540, 1.8 km east of the Highway 540 / Cooks Dock Road intersection on Manitoulin Island, 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 Priddle Rock 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 Priddle Rock Patrol Yard (site) is located on Highway 540 approximately 1.8 km east of the Highway 540 / Cooks Dock Road junction, on Manitoulin Island, Ontario. A Site Plan is included as Figure 1 and colour photographs of the site are included in Appendix C.

The site slopes to the west and is surrounded by trees. Fractured bedrock is generally at the surface. There is no standing water on the site and no drainage ditches surrounding the site. Some fractures in the bedrock were observed near the surface, indicating signs of Karst-like topography. Access is from Highway 540.

The site is an operational MTO Patrol Yard, and is currently occupied by a single sand shed and a single garage. There are two gravel piles located close to Highway 540, south of the entrance, and a sand pile further to the west which encroaches on the northeast corner of the proposed sand/salt structure footprint.

2.2 Regional Geology

The Physiography of Southern Ontario by Chapman and Putnam (1984) indicates that the project area is situated in the Manitoulin Island physiographic region, consisting mainly of limestone tablelands tilted slightly toward the southwest or south. Only a small portion of the island is covered with deep drift.

The Bedrock Geology of Ontario, Southern Sheet, Map 2544, issued by the Ministry of Northern Development and Mines in 1991 indicates that the bedrock in the general site area consists of grey interbedded sandstone, shale, dolostone, and siltstone of the middle and lower Silurian Age Amabel Formation.

Overburden thickness on this part of Manitoulin Island is generally thin. The depth to bedrock at the site was 150 mm to 300 mm. Bedrock outcrops are frequent.

3. Historic Report Review

No historic geotechnical reports were available from the MTO Geocres Library for the Priddle Rock Patrol yard or locations close to the yard. Previous work completed by GENIVAR on other parts of the island, as well the regional geology information taken from the sources quoted in Section 2, indicate that the site has shallow bedrock conditions.

4. Investigation Procedures

4.1 Subsurface Investigation

A borehole investigation was performed at the subject site on June 6, 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 as close to the four corners of the proposed storage structure as possible, as required in the Terms of Reference. Borehole BH12-4 was located just north of the proposed corner of the structure due to the presence of a sand stockpile on site.

MTO minimum requirements for the borehole investigation outlined a maximum drilling depth of 15.0 m, unless refusal was encountered at shallower depth, or justification for deeper drilling was authorized by the MTO Project Manager. As well, two (2) of the boreholes were to include rock cores for a minimum of 3.0 m depth below the rock surface, if bedrock was encountered within the shallow foundation zone.

In all four boreholes augering was terminated at 150 mm to 250 mm below ground surface on bedrock. Bedrock coring was completed in boreholes BH12-1 and BH12-3 to a depth of 3.0 m and 4.4 m, respectively.

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 (Northing and Easting). Borehole elevations were surveyed to a known benchmark. The anchor bolt in the wall of the southern sand/salt dome, with a reported geodetic elevation of 260.293 metres above sea level (mASL) was used. Borehole elevations and coordinates are shown on Drawing 1, and are provided on the borehole logs included in Appendix A.

Drilling and soil sampling was completed using a truck-mounted drill rig operating under the supervision of an experienced GENIVAR soils technician. The boreholes were advanced through the thin overburden using continuous flight augers. Grab samples of the soil were taken from the augers for reference. The samples were inspected and placed in labelled bags for transport, future reference, possible laboratory testing, and storage.

NQ-size coring equipment (47.6 mm diameter) was used to obtain two (2) bedrock core samples 3.0 m to 4.4 m long, as noted above. 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.

If groundwater was observed within the boreholes during drilling, prior to backfilling, it was noted in the field logs. The two (2) cored holes were backfilled with bentonite hole plug to the surface. The two (2) boreholes that were terminated on bedrock were backfilled with sand. The backfill material was compacted with the drill rig. As such, the boreholes are abandoned in accordance with O. Reg. 903 requirements, as amended. Table 4.1 below summarizes the borehole numbers and drilling depths and the surveyed elevations.

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

Borehole No.	Drilling Depth Below Existing Ground Surface (m) / Elevation (mASL)	Comment
BH12-1	3.0 / 217.5	Cored into bedrock
BH12-2	0.2 / 220.1	Met refusal on bedrock
BH12-3	4.4 / 216.3	Cored into bedrock
BH12-4	0.2 / 220.7	Met refusal on bedrock

4.2 Laboratory Testing

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.

One (1) Particle Size Analysis (ASTM D422) and one (1) Natural Moisture Content test was completed for a sample of the overburden material.

5. Subsurface Conditions

Subsurface conditions for the proposed structure were determined at four (4) borehole locations, designated as BH12-1 to BH12-4. Borehole locations and soil strata are shown in Drawing 1. Detailed borehole logs are provided in Appendix A, and laboratory test results are included in Appendix B.

5.1 Soil Profile Summary

All four (4) of the boreholes encountered a thin layer of overburden overlying bedrock. At boreholes BH12-1 and BH12-3 the bedrock was cored to a depth of 3.0 and 4.4 m below ground surface, respectively. Descriptions of the major subsurface units are provided in the following subsections.

5.1.1 Overburden

A 150 mm to 250 mm thick layer of overburden soil was encountered at each of the borehole locations. At borehole BH12-1 the overburden consisted of brown silty sand topsoil with some gravel. At the other three boreholes the overburden consisted of brown sand with a trace of gravel and a trace of silt.

A laboratory particle size distribution analysis for one (1) sample of the sand 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) - 3 %
- Sand (0.075 mm to 4.75 mm size) - 90 %
- Silt and Clay (less than 0.075 mm size) - 7 %

Laboratory determined moisture content was 5 % for the sand, indicating moist material.

5.1.2 Bedrock

Boreholes BH12-1 and BH12-3 were cored and terminated in dolostone bedrock. Borehole BH12-1 was terminated at 3.0 m depth below ground surface (elevation 217.5 mASL) while BH12-3 was terminated at 4.4 m depth (elevation 216. 3 mASL). Photographs of the bedrock core are included in Appendix C.

A description of the bedrock is provided in Table 5.1. Total Core Recovery (TCR) ranged from 68 % to 100 %. Rock Quality Designation (RQD) values for the core samples in borehole BH12-1 ranged from 56 % to 63 %, which is described as fair. The RQD values for borehole BH12-3 were 0% for the first two core runs, described as very poor, and 68 % for the final core segment, described as fair. The RQD values are consistent with the observed Karst-like fractures in the surface of the bedrock.

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

BH	RC #	Depth (m)	TCR (%)	RQD (%)	Depth (m)	Description
12-1	1	0.20 – 1.52	100	56	0.20 – 2.97	DOLOSTONE, grey, fine grained, medium bedded, medium hard, broken, slightly weathered
	2	1.52 – 2.97	100	63		
12-3	1	0.25 – 1.37	68	0	0.25 – 4.37	DOLOSTONE, grey, fine grained, medium bedded, medium hard, very broken to broken, slightly weathered
	2	1.37 – 2.85	69	0		
	3	2.85 – 4.37	100	68		

5.2 Groundwater Conditions

No groundwater was observed in the open boreholes during and upon completion of drilling. The overburden soil generally was moist. Based on these observations, the groundwater level within the footprint of the proposed structure, at the time of the field investigation, was estimated to be below 4.4 m below ground surface. Groundwater levels may fluctuate seasonally and in response to climatic conditions. Based on geological mapping, the degree of fluctuation should be less than 2 m.

6. Geotechnical Design Considerations

The proposed sand/salt storage facility at Priddle Rock Patrol Yard 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 and rock 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. In general, the boreholes encountered a 150 mm to 250 mm thick layer of sand with a trace of gravel soil overlying grey dolostone bedrock. The bedrock was confirmed by coring in borehole BH12-1 to 3.0 m depth below ground surface (elevation 217.5 mASL) and in borehole BH12-3 which was terminated at 4.4 m depth (elevation 216.3 mASL).

Groundwater was not encountered during drilling operations. It is inferred that the groundwater level within the footprint of the proposed structure, at the time of the field investigation, was more than 4.4 m below the ground surface.

6.1 “Red Flag” Conditions and NNSP’s

The presence of bedrock at the Priddle Rock site, at the surface and at shallow depths ranging from 150 mm to 250 mm below existing grade, may pose some challenges for construction of the foundation for the proposed sand/salt storage facility.

The following Non-Standard Special Provision (NNSP) is presented to address “Red Flag” conditions.

NNSP 1 The Contractor should be alerted of the subsurface conditions and that excavations for the foundation installation will be within weak to moderate strength dolostone bedrock. Any rock excavations for the new foundation should be carried out so as not to induce excessive vibrations which could damage the foundation of the nearby existing buildings.

6.2 Foundation Design

6.2.1 Shallow Foundation

Based on the results of this investigation, the proposed sand/salt storage facility can be supported on strip footings founded on dolostone bedrock, with a recommended highest founding level at 0.75 m below existing grade (elevation 219.7 mASL to 219.5 mASL) providing that permanent thermal insulation will be installed for frost protection. The other option is to lower the foundation level below 1.6 m depth (elevation 218.8 mASL to 218.6 mASL) for frost protection purposes.

It should be noted that in between and beyond borehole locations, the bedrock surface and the depth to suitable bedrock may vary considerably. In addition, the extent of the rock excavation will depend on the actual founding level. All loose, fractured, or highly weathered bedrock under the footing should be removed and replaced with concrete. Mass concrete (20 MPa minimum) may be placed to raise the grade to the founding level, where necessary.

The following geotechnical resistance can be used for the design of the strip footings with a minimum 0.9 m width, constructed on the bedrock:

- Factored Bearing Resistance at ULS, at 0.75 m depth= 1000 kPa
- Factored Bearing Resistance at ULS, at 1.6 m depth= 2000 kPa
- Bearing Resistance at SLS will not govern the design.

A deeper foundation elevation for the strip footing will provide higher geotechnical resistance but we believe it is not required for the proposed storage facility.

All footing excavations and bearing surfaces must be inspected, evaluated, and approved by a Geologist or Geotechnical Engineer who is familiar with the findings of this investigation.

6.2.2 Deep Foundation

Based on the subsurface conditions at the site, and serviceability and reliability of the proposed building, the use of deep foundations is not recommended.

6.3 Frost Penetration Depth

The recommended design frost protection depth for the site area is 1.6 m (Source: MTO Pavement Design and Rehabilitation Manual). Therefore, a permanent soil cover of about 1.6 m or its thermal equivalent of high density insulation is required for frost protection of foundations. In case of rockfill, only one-half of the rockfill thickness should be assumed to be effective in providing frost protection.

Any obvious cracks/joints at the surface of the excavated bedrock at the founding elevation should be grouted to prevent water penetration which may cause further cracking/jointing in the founding bedrock.

6.4 Resistance to Lateral Loads

Resistance to lateral forces/sliding between the concrete footings and clean bedrock surface should be calculated in accordance with Section 6.7.5 of the CHBDC. The ultimate angle of friction between the underside of the foundations and the clean, intact bedrock surface (or between concrete surface) can be taken as 32 degrees. The ultimate angle of friction can be increased to 35 degrees if the surface of the bedrock can be chiseled (i.e. roughened).

If additional horizontal resistance is required, or if the rock surface is not sufficiently level, dowelling or keying-in into the bedrock can be considered. Such measures would be required if the rock surface is smooth and/or inclined. The building foundation should be anchored to the underlying bedrock by dowels to resist overturning forces. For the strip footing, dowels should consist of 15 M steel rebar spaced at 750 mm center to centre, with minimum embedded depth of 500 mm into sound, unfractured bedrock.

6.5 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 earth pressures acting against walls should be in accordance with the CHBDC. For design purposes, the properties outlined in Table 6-1 can be assumed for backfill.

Table 6-1: 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 the retaining structure is free-draining granular material and adequate drainage is provided.

Based on the subsurface condition of the site, temporary shoring is likely not required to support excavations in rock, but if required, the shoring walls below grade can be designed using the following expression:

$$\mathbf{P = K\ (\gamma\ h + q)}$$

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 coefficients of lateral earth pressure given in Table 6-2 may be used for the design of the temporary shoring systems, based on the borehole results, if required.

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

Soil Type	K_a	K_O	K_p	γ (kN/m^3)
Sand trace Gravel	0.33	0.5	3.0	18.5
Bedrock (upper 0.5 m)	0.24	0.32	3.6	23.0
Bedrock (below 0.5 m)	0.12	0.15	5.0	24.0

Notes:
- K_a - coefficient of active earth pressure
- K_o - coefficient of earth pressure at rest
- K_p - coefficient of passive earth pressure
- **γ** – unit weight

6.6 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 'B' (Rock) be applied for structural design at this site.

Seismic information for the Priddle Rock 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	Priddle Rock	Source
Site Class	B	2006 Ontario Building Code Table 4.1.8.4.A
$S_a(0.2)$	0.116	2005 National Building Code Seismic Hazard Calculation
$S_a(1.0)$	0.033	2005 National Building Code Seismic Hazard Calculation
F_a	0.8	2006 Ontario Building Code Table 4.1.8.4.B
F_v	0.6	2006 Ontario Building Code Table 4.1.8.4.C

Based on the subsurface condition at the site where bedrock was found at surface and shallow depth, dynamic and static liquefaction are not a concern at this site.

6.7 Dewatering and Drainage

It is expected that the foundation excavations to approximately 0.75 m to 1.6 m below existing ground surface will not encounter significant groundwater. Localized lenses of perched groundwater within the rock may exist, but amounts should be minor.

If the groundwater table 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.

6.8 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 sand soils with a trace to some gravel encountered at the site may be classified as Type 3 soils above the groundwater level and Type 4 soils below the groundwater level, in accordance with the OHSA.

Cut slopes in the bedrock should be no more than 1H:4V and, consistent with the OHSA, the walls of the rock excavation must be stripped of any loose rock or other material that could slide, fall, or roll upon a worker. Type 3 soil excavations should be cut with 1H:1V or flatter side slopes. Type 4 soil 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 requirements and specifications of OPSS 206, as amended by SSP 206, and management of excess material should follow the requirements of OPSS 180.

Excavation into the bedrock could be carried out using a hardened-tooth excavator, or drilling and hoe ramming techniques where relatively shallow depths of cut into bedrock are required. Line drilling and pre-shearing techniques will provide better control over the configuration of the founding surface. However, the rock excavation should be carried out so as not to induce excessive vibrations which could damage the foundations and/or superstructure of the existing buildings. For example, if rock blasting is to be implemented, this should be carried out under close supervision with vibration monitoring.

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 sulfate-resistant cement in the concrete mix design.

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 Coordinators	Jennifer Wales, P. Eng. and 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, M.A.Sc., 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

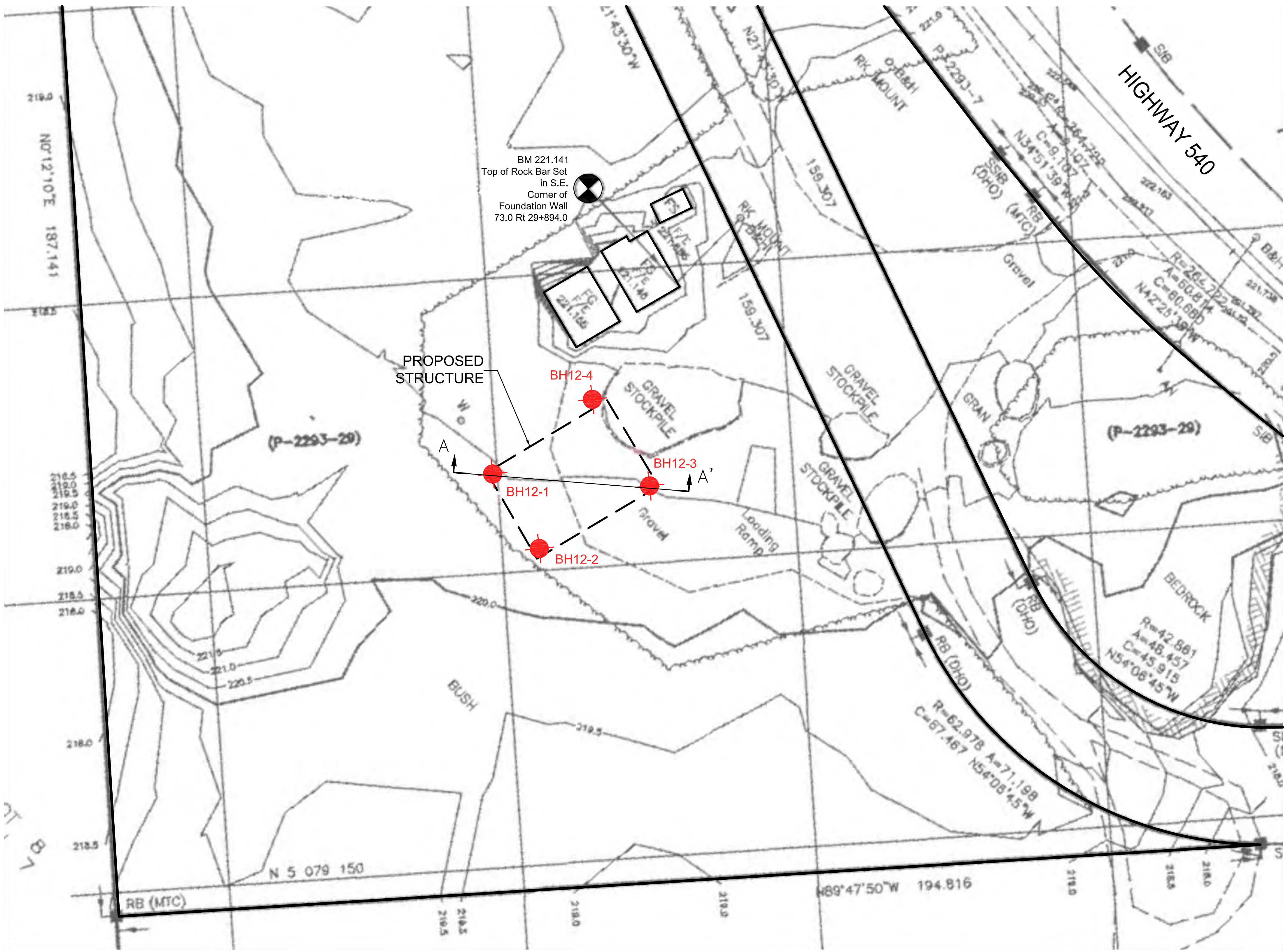
Reviewed by:

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

Drawings

Drawing 1 – Borehole Location Plan

Drawing 2 – Soil Strata



PLAN VIEW

SCALE 1:750

NOTES:

1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
2. COORDINATES AT BOREHOLE LOCATIONS WERE BY HANDHELD GPS.
3. BOREHOLE ELEVATIONS WERE SURVEYED RELATIVE TO IRON BAR AT THE SOUTHEAST CORNER OF THE SHED (EL. 221.141 m).

METRIC

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

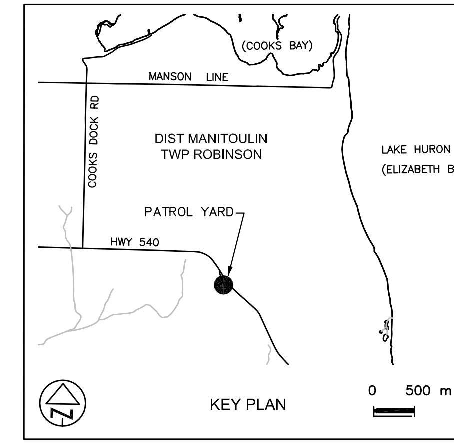
Agreement No.: 5011-E-0010
WO No.: 2011-11040

BOREHOLE LOCATION PLAN

PROPOSED SAND/SALT STORAGE FACILITY
PRIDDLE ROCK PATROL YARD
KING'S HIGHWAY 540

Client: MTO - Northeastern Region

DRAWING 1



KEY PLAN

LEGEND

- Borehole
- Benchmark (221.141 m)
- Proposed Sand/Salt Storage Facility
- A A' Line of Cross Section (See Figure 2)

BH No	ELEVATION (mASL)	COORDINATES (NAD 83 Zone17) NORTHING	EASTING
12-1	220.49	5079206.9	361248.9
12-2	220.29	5079193.6	361255.7
12-3	220.66	5079202.5	361275.3
12-4	220.91	5079218.0	361266.9

– 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 Jan 15/13 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER J. T. Balsdon Jan 15/13 PROVINCE OF ONTARIO

REVISIONS DATE	BY	DESCRIPTION

PROJECT: 121-17876-00 111-10

GENIVAR

SITE PLAN MAPPING REF. NO.:
MTO PLAN H-652-540-1, OCTOBER 6, 2008.

GEOCRES No. 41G13

HWY No 540			DIST MANITOULIN
SUBM'D --	CHECKED JSA	DATE JANUARY 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.: 2011-11040

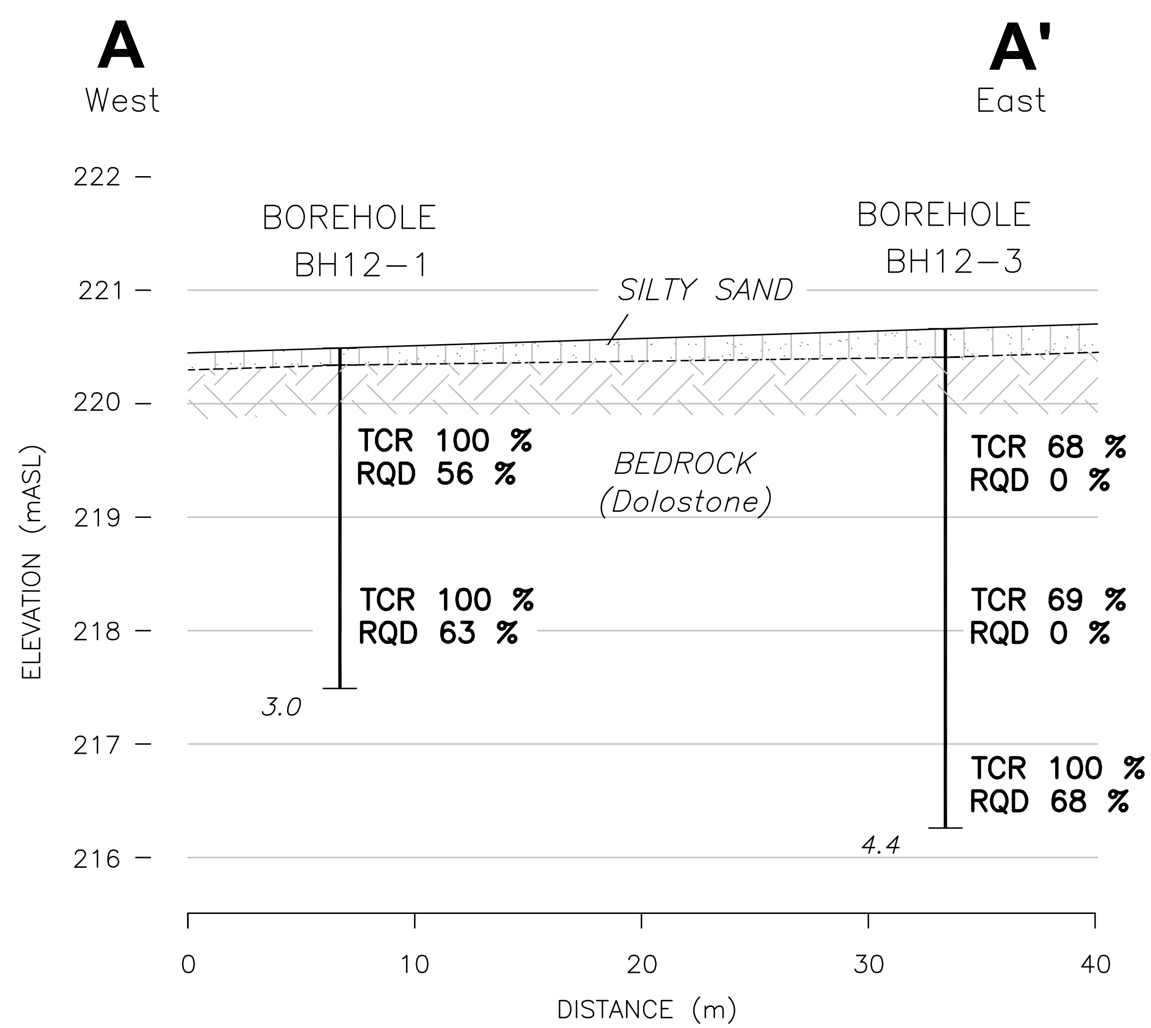
SOIL STRATA

PROPOSED SAND/SALT STORAGE FACILITY
PRIDDLE ROCK PATROL YARD
KING'S HIGHWAY 540

Client: MTO - Northeastern Region

DRAWING 2

CROSS SECTION A-A'



LEGEND

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PROJECT: 121−17876−00 111−10

GENIVAR

LICENSED PROFESSIONAL ENGINEER
J. S. Ash
Jan 15/13
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
J. T. Balsdon
Jan 15/13
PROVINCE OF ONTARIO

SITE PLAN MAPPING REF. NO.:
MTO PLAN H−652−540−1, OCTOBER 6, 2008.

REVISIONS		
DATE	BY	DESCRIPTION

GEOCRES No. 41G13

HWY No 540			DIST MANITOULIN
SUBM'D −−	CHECKED JSA	DATE JANUARY 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

Compactness	Standard Penetration Resistance "N", Blows / 0.3 m
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Over 50

COHESIVE SOIL

Consistency	Standard Penetration Resistance "N", Blows / 0.3 m	Undrained Shear Strength (cu) (kPa)
Very Soft	0 to 2	0 to 12
Soft	2 to 4	12 to 25
Firm	4 to 8	25 to 50
Stiff	8 to 15	50 to 100
Very Stiff	15 to 30	100 to 200
Hard	Over 30	Over 200

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

COHESIONLESS SOILS

- Dry
- Moist
- Wet
- Saturated

COHESIVE SOILS

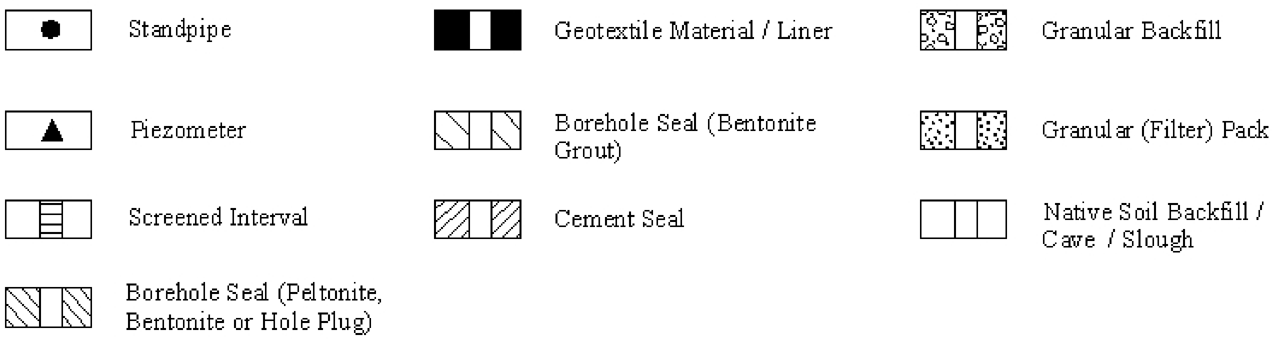
DTPL	-	Drier Than Plastic Limit
APL	-	About Plastic Limit
WTPL	-	Wetter Than Plastic Limit
MWTPL	-	Much Wetter Than Plastic Limit

STRATIGRAPHY

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

MONITOR DETAILS

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



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

SAMPLE

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

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

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

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

RQD Classification	RQD (%)
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

TEST DATA

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

Dynamic Penetration Resistance - The number of blows required to advance a 51 mm diameter, 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: PRIDDLE ROCK PATROL YARD N 5 079 206.9; E 361 248.9 — ORIGINATED BY: DCL
BOREHOLE TYPE: CONTINUOUS FLIGHT AUGERS — COMPILED BY: JW
DATUM: GEODETIC — DATE: 6.6.12 - 6.6.12 — CHECKED BY: JSA

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 (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 30 60 90 120 150)	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
220.5											
220.3 / 0.2	**TOPSOIL:** SILTY SAND TOPSOIL, SOME GRAVEL BROWN, MOIST		1	AS					○		
	BEDROCK: DOLOSTONE, FINE-GRAINED, MEDIUM BEDDED (HORIZONTAL), MEDIUM HARD, BROKEN, SLIGHTLY WEATHERED GREY		1	RC	TCR 100%		220				RQD 56%
			2	RC	TCR 100%		219 / 218				RQD 63%
217.5 / 3.0	END OF BOREHOLE										

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

RECORD OF BOREHOLE No BH12-2

METRIC

LOCATION: PRIDDLE ROCK PATROL YARD N 5 079 193.6; E 361 255.7
ORIGINATED BY: DCL

BOREHOLE TYPE: CONTINUOUS FLIGHT AUGERS
COMPILED BY: JW

DATUM: GEODETIC
DATE: 6.6.12 - 6.6.12
CHECKED BY: JSA

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 30, 60, 90, 120, 150)	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
220.3											
220.0 / 0.2	**SAND:** SAND, TRACE GRAVEL BROWN, MOIST										
	END OF BOREHOLE										

ONTARIO MOT PRIDDLE ROCK.GPJ ONTARIO MOT.GDT 7/26/12

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

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH12-3

METRIC

LOCATION: PRIDDLE ROCK PATROL YARD N 5 079 202.5; E 361 275.3 — ORIGINATED BY: DCL

BOREHOLE TYPE: CONTINUOUS FLIGHT AUGERS — COMPILED BY: JW

DATUM: GEODETIC — DATE: 6.6.12 - 6.6.12 — CHECKED BY: JSA

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
220.7 / 0.0	**SAND:** SAND BROWN, MOIST						
220.4 / 0.3	**BEDROCK:** DOLOSTONE, FINE-GRAINED, MEDIUM BEDDED (HORIZONTAL), MEDIUM HARD, VERY BROKEN TO BROKEN, SLIGHTLY WEATHERED GREY	1	RC	TCR 68%		220	RQD 0%
		2	RC	TCR 69%		219	RQD 0%
		3	RC	TCR 100%		218, 217	RQD 68%
216.3 / 4.4	END OF BOREHOLE						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE — 30, 60, 90, 120, 150

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 10, 20, 30

UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No BH12-4

METRIC

LOCATION: PRIDDLE ROCK PATROL YARD N 5 079 218.0; E 361 266.9

ORIGINATED BY: DCL

BOREHOLE TYPE: CONTINUOUS FLIGHT AUGERS

COMPILED BY: JW

DATUM: GEODETIC

DATE: 6.6.12 - 6.6.12

CHECKED BY: JSA

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 (30, 60, 90, 120, 150)	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
220.9											
220.8 / 0.0	**SAND:** SAND, TRACE GRAVEL, TRACE CLAY BROWN, MOIST		1	AS							3 90 (7)
0.2	END OF BOREHOLE										

Legend: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE

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

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

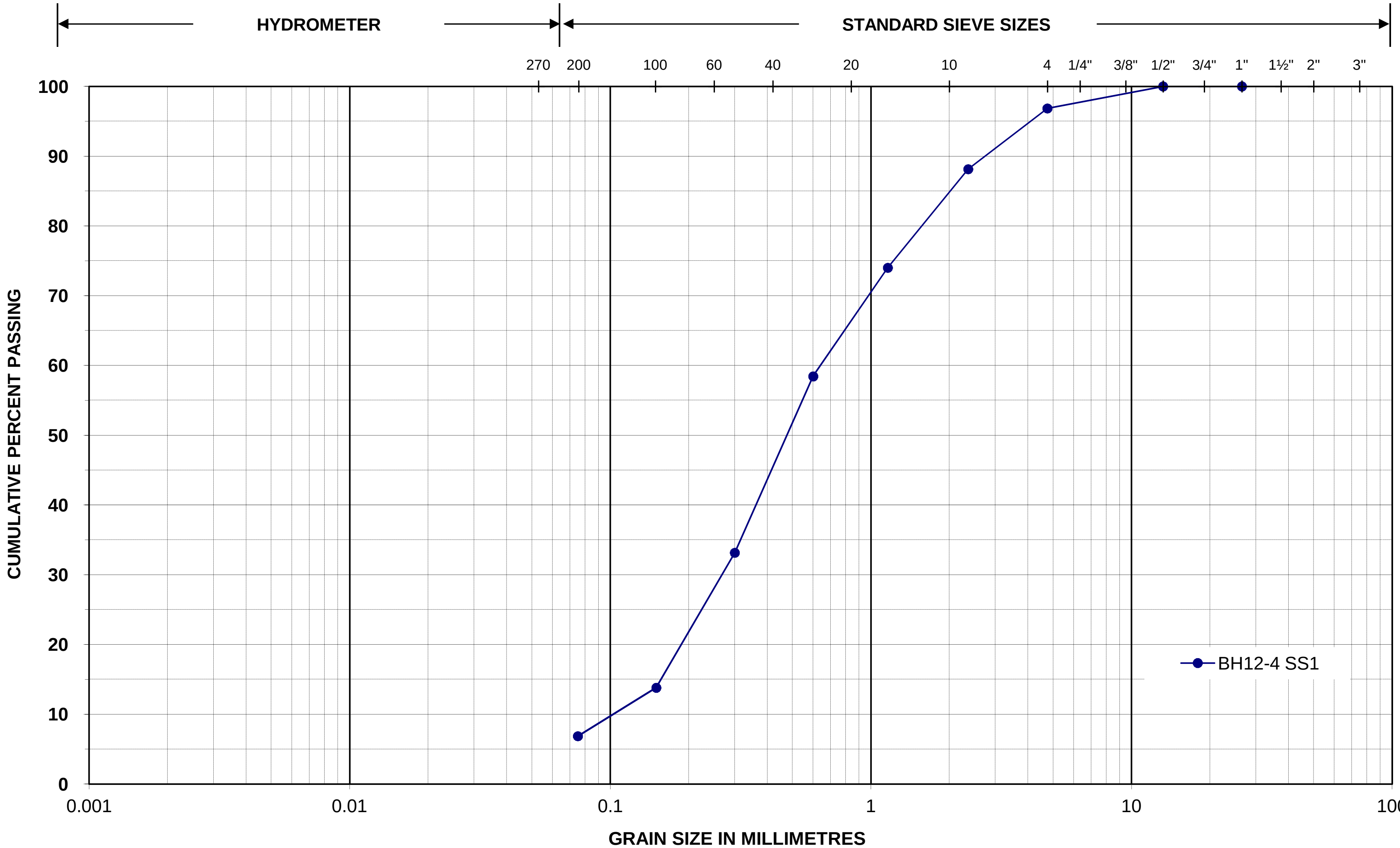
ONTARIO MOT PRIDDLE ROCK.GPJ ONTARIO MOT.GDT 7/26/12

Appendix B

Particle Size Distribution Analysis (Figure B1)

GENIVAR

PARTICLE SIZE DISTRIBUTION ASTM D422



Unified Classification System

SILT AND CLAY	SAND	GRAVEL

Project Name:	MTO Agreement #5011-E-0010 (Priddle Rock)	**Project No.:**	121-17876-00	**Figure No.:**	**B1**
Remarks:	Sand, trace silt, trace gravel				

Appendix C

Site Photographs

Bedrock Core Photographs

MTO AGREEMENT #5011-E-0010
PRIDDLE ROCK PATROL YARD

Photograph 1: BH12-1 and existing structures. Looking northeast.

Photograph 2: Bedrock outcrops. Looking south.

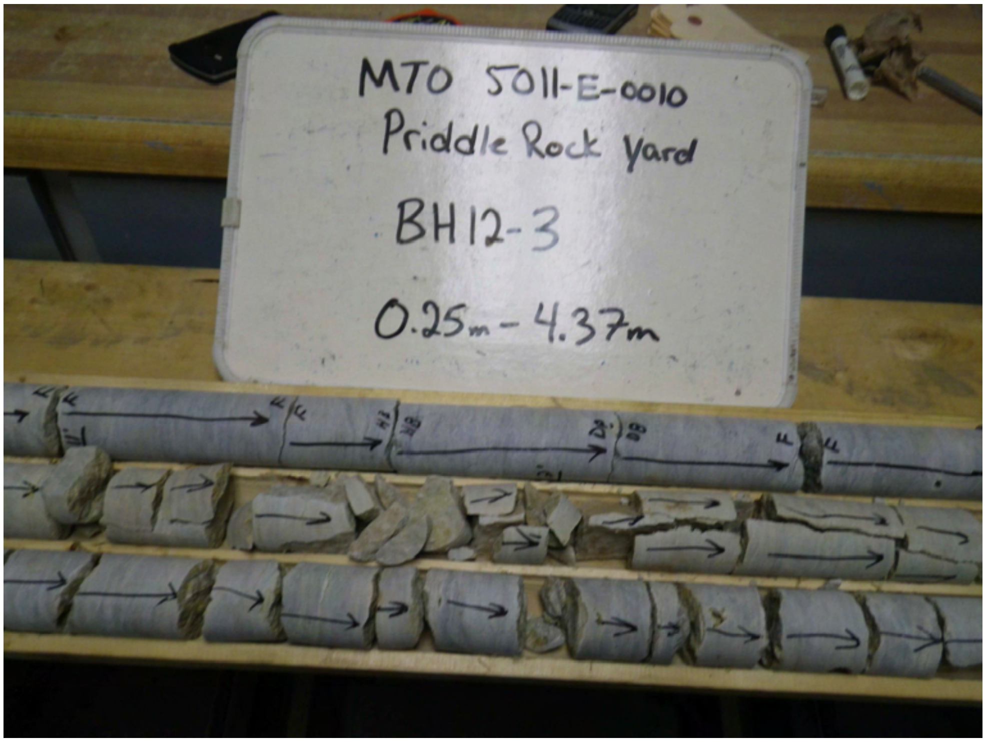
Photograph 3: Borehole BH12-2.

Photograph 4: Karst-like topography at site.

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

Photograph 1: BH12-1 Rock Core (0.2 m to 2.97 m).

Photograph 2: BH12-3 Rock Core (0.25 m to 4.37 m).



Photograph 3: BH12-3 Rock Core.