National Research Council Canada
Conseil national de recherches Canada

Administrative Services and Property Management Branch
Direction des services administratifs et gestion de l'immobilier

NRC · CNRC

Addendum / Addenda

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Project Description / Description de projet		
M-38 Flexible Research Facility		
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Project Engineer / Ingénieur de projet Allan Smith		Date September 14th , 2016

Notice:	Nota:
This addendum shall form part of the tender documents and all conditions shall apply and be read in conjunction with the original plans and specifications.	Cet addenda fait partie intégrale des dossiers d'appel d'offres; toutes les conditions énoncées doivent être lues et appliquées en conjonction avec les plans et les devis originaux.

Delete and Replace Addendum 1 with the attached

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January 2016

REPORT

REPORT ON

Geotechnical Investigation
Proposed Flexible Laboratory Facility M-26
National Research Council Canada (NRC)
Montreal Road Campus
1200 Montreal Road
Ottawa, Ontario

Submitted to:
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2 copies - National Research Council Canada
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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for a proposed flexible laboratory facility (Building M-26) to be located at 1200 Montreal Road on the National Research Council Canada (NRC) Montreal Road Campus in Ottawa, Ontario.

The purpose of this geotechnical investigation was to assess the general subsurface conditions in the area of the proposed building by means of eight test pits. Based on an interpretation of the factual information obtained, a general description of the subsurface conditions is presented. These interpreted subsurface conditions and available project details were used to prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The reader is referred to the "Important Information and Limitations of This Report" which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

Plans are being prepared for the construction of a flexible laboratory facility (Building M-26) to be located at 1200 Montreal Road on the NRC Montreal Road Campus in Ottawa, Ontario. The approximate location of the site is shown on the Key Map inset provided on the attached Site Plan (Figure 1).

The following is known about the project and site:

- The proposed site is located at the north end of the Montreal Road Campus between Douglas Street and Howlett Street.
- The building is planned to be constructed in three stages (the initial building will have two additions/expansions constructed in the future). Initially, the building will be about 36 metres wide and 35 metres long in plan area. The subsequent two additions will increase the length of the building in the north direction by about 35 and 27 metres, respectively. The width of the building will remain unchanged.
- The building will be about 12 metres in height and will be of slab-on-grade construction (i.e., no basement level).
- The site of the "initial" building is vegetated with grass and is currently occupied by a fenced-in area containing two satellite dishes. To the north, where the future extensions will be located, the site is undeveloped and vegetated with grass and a small forested area.
- The ground surface is relatively flat to gently sloping, with ground surface elevations ranging from about 99 to 101 metres.

Golder Associates has carried out several previous geotechnical investigations on the NRC Montreal Road Campus. Based on the results of those previous investigations, as well as published geological mapping, the subsurface conditions at this site are expected to consist of glacial till overlying shallow limestone bedrock. The bedrock is indicated to be at depths of about 0 to 1 metre below the ground surface. Bedrock geology mapping published by the Geological Survey of Canada indicates that the bedrock at the site consists of interbedded limestone and dolomite of the Gull River Formation. However, the site is also indicated to be in close proximity to an area mapped as shale of the Rockcliffe Formation (adjacent to the north portion of the site). A fault is also mapped to the north, at the boundary between the two rock formations.

3.0 PROCEDURE

3.1 Subsurface Investigation

The fieldwork for this investigation was carried out on December 4, 2015. At that time, eight test pits (numbered 15-1 to 15-8, inclusive) were excavated at the approximate locations shown on the attached Site Plan (Figure 1).

The test pits were advanced to practical refusal on the bedrock, which was encountered at depths ranging from about 0.5 to 1.7 metres below the existing ground surface.

The test pits were excavated using a rubber-tired backhoe supplied and operated by Glenn Wright Excavating of Ottawa, Ontario. The soils exposed on the sides of the test pits were classified by visual and tactile examination. The groundwater seepage conditions were observed in the open test pits (during the short time that they remained open), and the test pits were loosely backfilled upon completion of excavating and sampling.

The fieldwork was supervised by an experienced technician from our staff who located the test pits, directed the excavating operations, logged the test pits and samples, and took custody of the samples retrieved. On completion of the excavating operations, samples of the soils obtained from the test pits were transported to our laboratory for examination by the project engineer.

One sample of soil from test pit 15-5 was submitted to EXOVA Laboratories for chemical analysis related to potential corrosion of buried steel elements and potential sulphate attack on buried concrete elements.

The test pit locations were selected in consultation with the NRC, and subsequently marked in the field and surveyed by Golder Associates personnel. The coordinates and ground surface elevations at the test pit locations were determined using a Trimble R8 GPS survey unit. The Geodetic reference system used for the survey is the North American datum of 1983 (NAD83). The test pit coordinates are based on the Universal Transverse Mercator (UTM Zone 18) coordinate system. The elevations are referenced to Geodetic datum (CGVD28).

3.2 Geophysical Testing

In addition to the subsurface investigation, shear wave velocity testing was carried out at the site for the purpose of determining the site classification for seismic site response, in accordance with the 2010 National Building Code of Canada (NBCC). The testing was completed using the Multichannel Analysis of Surface Waves (MASW) technique. The MASW fieldwork was carried out on November 26, 2015, by personnel from Golder's Mississauga office. For this testing, a series of 24 low frequency (4.5 Hz) geophones were laid out at approximately 3 metre intervals. A seismic weight-drop of 45 kilograms and a 9.9 kilogram sledge hammer were used as seismic sources for this testing. Seismic records were collected with seismic sources located approximately 5, 10, 15 and 20 metres from and collinear to the geophone array. The results of the MASW test include the calculated shear wave velocity profile measured from the field testing and a graphical representation of the shear wave velocity profile with depth.

4.0 SUBSURFACE CONDITIONS

Information on the subsurface conditions is presented as follows:

- The Record of Test Pits are provided in Table 1.
- The results of the MASW shear wave velocity testing are provided in Appendix A.
- The results of the basic chemical analysis are provided in Appendix B.

In general, the overburden soils at this site consist of about 100 to 400 millimetres of topsoil overlying thin, discontinuous deposits of silty clay, silty sand, and glacial till. These deposits generally range from about 0.1 to 0.5 metres in thickness. The glacial till at TP 15-8 is up to about 1.5 metres in thickness. The glacial till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand. A possible layer of silty sand fill, which extends to about 0.6 metres depth, was also encountered at test pit 15-5.

The overburden is underlain by limestone bedrock, which was encountered in all of the test pits at depths ranging from about 0.1 to 1.7 metres below the existing ground surface. Bedrock outcrops are also visible in other areas of the site. The bedrock surface was uneven/stepped in test pit 15-8, where it ranged in depth from about 0.8 to 1.7 metres below the existing ground surface.

At test pit 15-4, the upper bedrock was fractured and could be excavated (in boulder sized slabs) from about 0.1 to 0.5 metres depth, before encountering practical refusal to excavating. A thin layer of fractured bedrock, about 0.1 metres in thickness, was also encountered at test pit 15-2 at about 0.5 metres depth. Elsewhere, and below the fractured bedrock at test pits 15-2 and 15-4, practical refusal to excavating was encountered on a sound/competent bedrock surface. A summary of the depths and elevations of practical refusal to excavating at each of the test pits is provided in the following table:

Test Pit Number	Ground Surface Elevation (m)	Practical Refusal on Sound Bedrock	
		Depth (m)	Elevation (m)
15-1	100.1	0.7	99.4
15-2	99.9	0.6	99.3
15-3	100.2	0.7	99.5
15-4	99.4	0.5	99.4
15-5	100.1	0.9	99.2
15-6	100.3	0.8	99.5
15-7	100.4	0.8	99.6
15-8	100.7	0.8 / 1.7(1)	99.9 / 99.0(1)

Notes: (1) A step in the bedrock was encountered within TP 15-8.

All of the test pits were observed to be dry at the time of excavation (i.e., the groundwater level was not encountered). However, groundwater levels are expected to fluctuate seasonally, and higher groundwater levels are expected during wet periods of the year, such as spring.

One sample of soil from test pit 15-5 was submitted to EXOVA Laboratories for chemical analysis related to potential corrosion of buried steel elements and potential sulphate attack on buried concrete elements. The results of this testing are provided in Appendix B and are summarized in the table below.

Test Pit/Sample Number	Sample Depth (m)	Chloride (%)	SO_4 (%)	pH	Resistivity (ohm-cm)
15-5 / Sa 2	0.6 – 0.9	<0.002	0.03	8.0	3,120

5.0 DISCUSSION

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements.

Reference should be made to the "Important Information and Limitations of this Report" which follows the text of this report but forms an integral part of this document.

The foundation engineering guidelines presented in this section have been developed in a manner consistent with the procedures outlined in Part 4 of the 2010 NBCC for Limit States Design.

5.2 Site Grading

In general, the overburden soils at this site consists of about 100 to 400 millimetres of topsoil overlying thin, discontinuous deposits of silty clay, silty sand, and glacial till. The overburden is underlain by limestone bedrock, which is generally less than 1 metre deep across the site.

Based on the results of this investigation, there is no practical limit on the amount of grade raise fill that can be placed on this site (from the perspective of the compressibility of the underlying soil).

As a more general guideline regarding the site grading, the preparation for filling of the site should include stripping the topsoil and fill from within the footprint of the proposed structure. The topsoil should also be removed from beneath pavement areas (if planned). The topsoil and fill should be stockpiled separately for re-use in landscaping applications only.

5.3 Excavations

No unusual problems are anticipated in excavating the overburden soils using conventional hydraulic excavating equipment, recognizing that large cobbles and boulders should be expected within the glacial till deposit. Large slabs of fractured bedrock may also need to be excavated near the bedrock surface. Boulders larger than 0.3 metres in size should be removed from the walls of the excavations for worker safety.

Provided that the groundwater level is not encountered during excavation (which is expected to be the case), the Occupational Health and Safety Act (OHSA) of Ontario indicates that side slopes in the overburden soils could be sloped at 1 horizontal to 1 vertical (i.e., Type 3 soils), or flatter.

Excavations will likely be carried out above the groundwater level; therefore, significant groundwater infiltration to the excavations is not anticipated. Water that accumulates in the bottom of the excavations (e.g., from perched groundwater, surface water, or precipitation) can be handled by pumping from well filtered sumps established in the floor of the excavations. The pumping volumes are anticipated to be less than 50,000 litres per day; therefore, the requirement for a Permit-To-Take-Water (PTTW) is not anticipated.

If excavation of the bedrock is required (e.g., for service pipe connections or deeper footigns), it is anticipated that the bedrock removal could be carried out using mechanical methods (e.g., hoe ramming), potentially in conjunction with closely spaced line drilling. Blasting could also be considered as a means of bedrock removal; however, it is anticipated that hoe ramming will likely be the most economical method of bedrock removal for the anticipated relatively small bedrock excavations required for this project.

Loose rock should be removed from the sidewalls of the excavations. Relatively steep to near-vertical walls in the bedrock should stand unsupported for the construction period. However, the rock walls should be inspected at the time of excavation so the rock wall stability guidelines can be confirmed.

Significant caution should be exercised in carrying out bedrock removal because of the near proximity of underground services and existing buildings, which may contain vibration-sensitive equipment. Hoe-ramming and/or blasting should be controlled to limit the peak particle velocities at all adjacent structures and services such that vibration induced damage will be avoided. The frequency dependent peak vibration limits from Ontario Provincial Standard Specification for Municipal projects (OPSS.MUNI) 120 should be specified in the contract for *all* construction activities (including blasting, hoe-ramming, etc.). The vibration limits from OPSS.MUNI 120 are summarized as follows:

Frequency Range (Hz)	**Vibration Limits (mm/s)**
≤ 40	20
> 40	50

The vibration limits given above are intended to prevent *structural* damage to nearby structures and utilities. If vibration-sensitive equipment is located in proximity to the construction, more stringent vibration limits may be required. Further guidance regarding vibration limits can be provided, if requested.

Regardless of the method of bedrock removal, a pre-construction survey should be carried out on all of the surrounding structures. Selected existing interior and exterior cracks in the adjacent structures should be identified during the survey and should be monitored during construction for lateral or shear movements by means of pins, glass plate telltales, and/or movement telltales.

If blasting is the chosen method of construction, a blast design by a specialist in this field will be required. If blasting, the contractor should be limited to only small controlled shots. Blasting should be carried out in accordance with OPSS.MUNI 120, which provides the requirements for blast design and submissions, including pre-construction surveys.

The contractor should be required to submit a complete and detailed bedrock excavation/blasting and monitoring proposal prepared by a blasting/vibrations specialist prior to commencing blasting or other bedrock excavation activities (such as hoe-ramming). This would have to be reviewed and accepted in relation to the requirements of the vibration specifications given above.

If practical, blasting or hoe-ramming should commence at the furthest points from the closest structure or service to assess the ground vibration attenuation characteristics and to confirm the anticipated ground vibration levels based on the contractor's proposal.

5.4 Foundations

5.4.1 Axial Bearing Resistance

The proposed building at this site can be supported on conventional spread footings founded on or within the limestone bedrock.

Footings constructed on or within competent bedrock can be sized using an Ultimate Limit States (ULS) factored bearing resistance of 1,000 kilopascals. Any weathered or highly fractured bedrock, which includes bedrock which can be excavated using hydraulic excavating equipment with only moderate effort, should first be removed from the footing areas. Provided the bedrock surface is acceptably cleaned of soil and loose bedrock, and that any weathered bedrock has been removed, the settlement of footings at the corresponding service (unfactored) load levels will be less than 25 millimetres and therefore Serviceability Limits States (SLS) need not be considered in the foundation design.

The ULS factored bearing resistance given above assumes that the bedrock below founding level may contain some limited voids, mud seams, and zones of fracturing and/or weathering within the depth of influence of the footings. If desired, bedrock coreholes could be carried out in advance of construction to confirm the presence/absence of these features. If the bedrock is fresh and free of voids and mud seams, a higher ULS factored bearing resistance value could be given (potentially in the range of 2,000 to 3,000 kilopascals).

It is anticipated that the bedrock surface is generally flat to gently sloping at this site. At the test pit locations, the competent bedrock surface elevation ranges between about 99 and 100 metres. However, there may be locations where the competent bedrock subgrade elevation is lower than the underside of footing elevation (e.g., in the area of TP 15-8). Where this is the case, the native soils below the footings should be sub-excavated down to the bedrock surface and then either:

- Spread footings constructed directly on the deeper bedrock surface; or
- The excavation filled back up to a higher founding level using a levelling mat of mass lean concrete with a compressive strength of at least 5 megapascals.

In addition to these two options, footings could also potentially be placed on the native soil (e.g., glacial till) or engineered fill that extends to the bedrock surface; however, in either of these cases differential settlement of the foundations will occur due to the varying stiffnesses of the different bearing stratums (e.g., soil versus bedrock). Also, footings bearing on soil will have a much lower ULS factored bearing resistance and SLS conditions would need to be considered. In general, the volume of sub-excavation is anticipated to be low and therefore not a detriment to the overall cost of construction. For predictable performance of the structure, a single bearing stratum (bedrock) is recommended. Additional guidelines regarding multiple bearing stratums can be provided, if requested.

If the first option is chosen, and sloping bedrock is encountered, a stepped footing may be required. The dimensions of the steps should be determined at the time of construction in consultation with the geotechnical and structural engineer, once the bedrock subgrade is exposed.

5.4.2 Resistance to Sliding

A coefficient of friction (i.e., tan δ) of 0.7 (unfactored) may be used in the assessment of sliding resistance between cast-in-place concrete footings and the bedrock surface. If insulation is used between the concrete footing and the bedrock subgrade, the potential for shearing across the interface with the insulation should also be checked using a coefficient of friction of 0.45. If greater resistance is required, the footings could be provided with shear keys or prestressed rock anchors could be used to increase the normal stress level across the concrete/bedrock interface. Further guidelines on shear key and prestressed rock anchor design can be provided, if required.

5.5 Seismic Design

The seismic design provisions of the 2010 NBCC depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock below founding level. To support a Site Class designation, MASW shear wave velocity testing was carried out at this site. The results of the shear wave velocity testing are provided in Appendix A. Interpretation of the shear wave velocity data indicates that the average shear wave velocity to 30 metres depth is 1,536 metres per second (interpreted from ground surface). Accordingly, a Site Class A designation is appropriate for this site.

5.6 Frost Protection

Unless confirmed to be otherwise, the bedrock at this site is presumed to be frost susceptible. Therefore, all exterior foundation elements should be provided with a minimum of 1.5 metres of earth cover for frost protection purposes. Isolated, unheated footings adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover. Insulation could also be considered as an alternative to earth cover, as discussed below.

Shallow bedrock may be frost susceptible, especially in the upper highly fractured zone, where it may contain joints filled with frost susceptible soil. If/where the earth cover requirements over the rock bearing surface cannot be provided, the absence of soil-filled seams in the underlying rock should be confirmed at the time of construction. This assessment can be carried out by drilling 50 millimetre diameter probe holes within the footing areas at a 3 metre spacing and to at least 1.8 metres below the finished grade level. In the case that soil-filled seams are encountered, the following two options could be considered:

- The footing and bearing surface could be insulated; or,
- The potentially frost-susceptible bedrock could be removed (sub-excavated) and replaced with mass concrete, or the footing founded at that new lower depth.

For planning purposes, a typical detail for the use of high density polystyrene rigid foam insulation is provided on Figure 2. In preparation for the insulation, a levelling mat consisting of 25 millimetres of concrete sand or 50 millimetres of lean concrete should be placed on the approved bearing surface. Care must be taken to ensure that the insulation is not damaged during construction. Joints should be carefully lap jointed and glued where and if possible. Footings may then be constructed on the surface of the insulation. The type of insulation should be selected such that the bearing pressure on the insulation placed under the footings does not exceed about 35 percent of the insulation's quoted compressive strength. This is due to the time dependant creep characteristics of this material. For example, bearing resistance values for several strengths of insulation for SLS and ULS design are provided below:

Insulation Type	SLS Resistance (kPa)	ULS Factored Resistance (kPa)
Dow SM	65	100
Dow Styrofoam Highload 40	90	135
Dow Styrofoam Highload 60	145	205
Dow Styrofoam Highload 100	240	340

The insulation which projects beyond the edge of the footings can consist of Dow SM (or equivalent), except beneath flexible pavements, where Highload 60 (or equivalent) should be used beyond the footing.

If sub-excavation of the frost-susceptible bedrock is carried out, the depth of sub-excavation will need to be sufficient so as to expose a non-frost susceptible bedrock surface or bring the subgrade level down to the frost penetration depth (i.e., 1.5 metres), whichever is shallowest.

5.7 Rock Anchors

If required, passive rock anchors can be considered to resist seismic overturning moments. The anchors could consist of either grouted or mechanical anchors.

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

i) Failure of the steel tendon or top anchorage.

ii) Failure of the grout/tendon bond.

iii) Failure of the rock/grout bond.

iv) Failure within the rock mass, or rock cone pull-out.

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion.

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as 750 kilopascals for ULS design purposes. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance should be calculated based on the buoyant weight of the potential mass of rock which could be mobilized by the anchor. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \phi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

Where: Q_r = Factored uplift resistance of the anchor, kilonewtons;

ϕ = Resistance factor, 0.3;

γ' = Effective unit weight of rock, use 17 kilonewtons per cubic metre;

D = Anchor length in metres; and,

θ = ½ of the apex angle of the rock failure cone, use 30 degrees.

Where the anchor load is applied at an angle to the vertical, the anchor capacity should be reduced as follows:

$$Q_r' = Q_r \cos(\alpha)$$

Where: Q_r' = Factored uplift resistance of the anchor subject to inclined load in kilonewtons;

Q_r = Factored uplift resistance of the anchor, kilonewtons; and,

α = Angle between the load direction and the vertical.

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. In the case of group effects for a series of rock anchors in a rectangle with width "a" and length "b" installed to a depth "D", the equation for the volume of the truncated trapezoid failure zone would be as follows:

$$V = \frac{4}{3}\, D^3 \sin^2\varphi + \ aD^2 \sin\varphi + bD^2 \sin\varphi + abD$$

Where: V = Volume of the truncated trapezoid failure zone in cubic metres;

D = Depth of anchor group in metres;

a = Width of anchor group in metres;

b = Length of the anchor group in metres; and,

φ = ½ of the apex angle of the rock failure cone, use 30 degrees.

The ULS factored geotechnical resistance for the truncated trapezoid failure formed by the group of anchors can then be calculated based on the following equation:

$$Q_r = \phi\ \gamma'\, V$$

Where: Q_r = Factored uplift resistance of the anchor, kilonewtons;

ϕ = Resistance factor, use 0.3;

γ' = Effective unit weight of rock, use 17 kilonewtons per cubic metre; and,

V = Volume of truncated trapezoid in cubic metres.

It is suggested that proof-load tests be carried out on the anchors. The proof-load tests should be carried out to 1.3 times the anchor service loads, and at least 10 percent of the anchors should be tested in this manner.

The installation and testing of the anchors should be supervised by the geotechnical engineer. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grout area with a minimum of voids. It is also suggested that the anchor holes be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the anchor holes to be grouted to ensure an adequate bond between the grout and the rock.

5.8 Slab on Grade

Conventional slab on grade construction can be used for the proposed building at this site.

In preparation for construction of the slab on grade, the topsoil, fill, and all loose, wet, and disturbed material should be removed from within the building footprint. Provision should be made for at least 150 millimetres of OPSS Granular A to form the base for the slab on grade. Any bulk fill required to raise the grade to the underside of the Granular A should consist of OPSS Granular B Type II. The underslab fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.9 Foundation Wall Backfill

The soils at this site are frost susceptible and should not be used as backfill against exterior or unheated foundation elements. To avoid problems with frost adhesion and heaving, these foundation elements should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill and the adjacent areas. To reduce this differential heaving, the backfill adjacent to the wall should be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 metres below finished exterior grade, or to the bedrock surface (whichever is shallower), at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall. The granular fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

The pavement or hard surfacing could be expected to perform better in the long term if the granular backfill against the foundation walls is drained by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in a geotextile, which leads by gravity drainage to a positive outlet.

5.10 Site Servicing

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for sewer and water pipes. If bedrock is present at the subgrade level (which is expected to be the case at this site), the bedding should be thickened to 300 millimetres. Where unavoidable disturbance to the subgrade surface occurs during construction, it may be necessary to place a sub-bedding layer consisting of 300 millimetres of compacted OPSS Granular B Type II beneath the Granular A, or the Granular A could be thickened. The bedding material should in all cases extend to the spring line of the pipe and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials and native soils could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from the spring line of the pipe to at least 300 millimetres above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 millimetres. The cover material should be compacted to at least 95 percent of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the excavated inorganic soils as trench backfill. Where the trench will be covered with hard surfaced areas (e.g., pavements and sidewalks), the type of material placed in the frost zone (between subgrade level and 1.8 metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

Excavated bedrock could also potentially be used as trench backfill for the lower portion of the trench in areas where the excavations are in rock, provided that the rock fill is broken/crushed to form a well-graded material. However, the reuse of such rock fill should be reviewed and approved by the geotechnical engineer at the time of construction once the grading of the material proposed for reuse can be determined. The rock fill should only be placed higher than at least 300 millimetres above the pipe to minimize damage due to impact or point load. The pieces of the rock fill used as trench backfill should be limited to a maximum of 300 millimetres in nominal size and the rock fill should be disseminated throughout (i.e., nests of large rock pieces should not be permitted).

5.11 Pavement Design

In preparation for pavement construction (if required), all topsoil and any unsuitable fill (i.e., fill containing organic matter) should be excavated from the pavement areas for predictable pavement performance.

Areas requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material. Grade raise fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

The surface of the subgrade or fill should be crowned to promote drainage of the pavement granular structure. Perforated pipe subdrains should be provided at subgrade level extending from the catch basins for a distance of at least 3 metres in four orthogonal directions, or longitudinally where parallel to a curb. Alternatively, the subdrains could outlet into a nearby drainage swale.

The pavement structure for access roadways and truck traffic areas should consist of the following:

Pavement Component	Thickness (mm)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	450

The pavement structure for car parking areas should consist of the following:

Pavement Component	Thickness (mm)
Asphaltic Concrete	50
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	300

The granular base and subbase materials should be uniformly compacted to at least 100 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with OPSS 310.

The composition of the asphaltic concrete pavement in car parking areas should be as follows:

- Superpave 12.5 Surface Course – 50 millimetres

The composition of the asphaltic concrete pavement in access roadways and truck traffic areas should be as follows:

- Superpave 12.5 Surface Course – 40 millimetres
- Superpave 19.0 Binder Course – 50 millimetres

The pavement design should be based on a Traffic Category of Level B. The asphalt cement used on this project should be made with PG 58-34 asphalt cement on all lifts.

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have been adequately compacted to the required densities and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

The bedrock at this site could be encountered at depths shallower than the proposed pavement structure thicknesses given above. If the bedrock surface is encountered above the proposed subgrade level, and is flat to gradually sloping, the thickness of the Granular B Type II subbase can be reduced accordingly to avoid the requirement for rock removal/shattering. If a step in the bedrock is encountered, a transition between the different pavement structure thicknesses will be required. For planning purposes, a transition sloped at 3 horizontal to 1 vertical would likely be adequate; however, this should be confirmed by the geotechnical engineer at the time of construction once the pavement subgrade is completely exposed.

5.12 Corrosion and Cement Type

One sample of soil from test pit 15-5 was submitted to EXOVA Laboratories for chemical analysis related to potential corrosion of buried steel elements and potential sulphate attack on buried concrete elements. The results of this testing are provided in Appendix B and are summarized in Section 4.0.

The results indicate that concrete made with Type GU Portland cement should be acceptable for substructures. The results also indicate a potential for corrosion of exposed ferrous metal.

6.0 ADDITIONAL CONSIDERATIONS

The test pits excavated and filled on site constitute zones of disturbance to the surficial soils. These disturbed areas could affect the performance of surface structures and hard surfacing (e.g., slabs on grade and pavements). If the test pit locations are located within these areas, the backfill soil in the test pits will need to be removed and replaced with engineered fill.

The soils at this site are sensitive to disturbance from ponded water, construction traffic, and frost.

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that soil having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction view point.

At the time of the writing of this report, only preliminary details for the proposed building were available. Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

7.0 CLOSURE

We trust that this report meets your current needs. If you have any questions, or if we may be of further assistance, please do not hesitate to contact the undersigned.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
S. W. DUNLOP
100151238
PROVINCE OF ONTARIO

Troy

Stephen Dunlop, P.Eng.
Geotechnical Engineer

Troy Skinner, P.Eng.
Associate, Geotechnical Engineer

SD/TMS/ob

n:\active\2015\3 proj\1543311 nrc 1200 montreal flexible rf ottawa\08_reports\1543311 report-001 january final 2016 docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, **National Research Council Canada.** The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS
OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

TABLE 1
RECORD OF TEST PITS

Test Pit Number (Elevation)	Depth (metres)	Description
15-1 (100.1 metres)	0.0 – 0.1	TOPSOIL
	0.1 – 0.3	(CI) SILTY CLAY; grey-brown, (WEATHERED CRUST); cohesive, moist
	0.3 – 0.5	(SM) gravelly SILTY SAND; dark brown to black, with cobbles and organic matter, (GLACIAL TILL); non-cohesive, moist
	0.5 – 0.7	(SM) gravelly SILTY SAND; red-brown, with cobbles, (GLACIAL TILL); non-cohesive, moist
	0.7	Practical refusal on BEDROCK
		Notes: Test pit dry upon completion
15-2 (99.9 metres)	0.0 – 0.1	TOPSOIL
	0.1 – 0.2	(CI) SILTY CLAY; grey-brown, (WEATHERED CRUST); cohesive, moist
	0.2 – 0.5	(SM) SILTY SAND; red-brown; non-cohesive, moist
	0.5 – 0.6	Fractured BEDROCK
	0.6	Practical refusal on BEDROCK
		Notes: Test pit dry upon completion

Sample	Depth (m)
1	0.1 – 0.2
2	0.2 – 0.5
3	0.5 – 0.6

15-3 (100.2 metres)	0.0 – 0.2	TOPSOIL
	0.2 – 0.7	(SM) gravelly SILTY SAND; grey-brown, with clay seams and cobbles, (GLACIAL TILL); non-cohesive, moist
	0.7	Practical refusal on BEDROCK
		Notes: Test pit dry upon completion

Sample	Depth (m)
1	0.2 – 0.7

15-4 (99.4 metres)	0.0 – 0.1	TOPSOIL
	0.1 – 0.5	Fractured BEDROCK slabs/boulders and cobbles; grey to brown, with organic matter and silty sand; non-cohesive, dry to moist
	0.5	Practical refusal on BEDROCK
		Notes: Test pit dry upon completion

TABLE 1

RECORD OF TEST PITS

Test Pit Number (Elevation)	Depth (metres)	Description
15-5 (100.1 metres)	0.0 – 0.2	TOPSOIL
	0.2 – 0.6	(SM) gravelly SILTY SAND; red-brown, (Possible Fill); non-cohesive, moist
	0.6 – 0.9	(SM) gravelly SILTY SAND; grey-brown, with clay seams and cobbles, (GLACIAL TILL); non-cohesive, moist
	0.9	Practical refusal on BEDROCK
		Notes: Test pit dry upon completion

Sample	Depth (m)
1	0.2 – 0.6
2	0.6 – 0.9

15-6 (100.3 metres)	0.0 – 0.2	TOPSOIL
	0.2 – 0.4	(CI) SILTY CLAY; grey-brown, (WEATHERED CRUST); cohesive, moist
	0.4 – 0.8	(SM) gravelly SILTY SAND; red-brown; with cobbles and boulders (GLACIAL TILL); non-cohesive, moist
	0.8	Practical refusal on BEDROCK
		Notes: Test pit dry upon completion

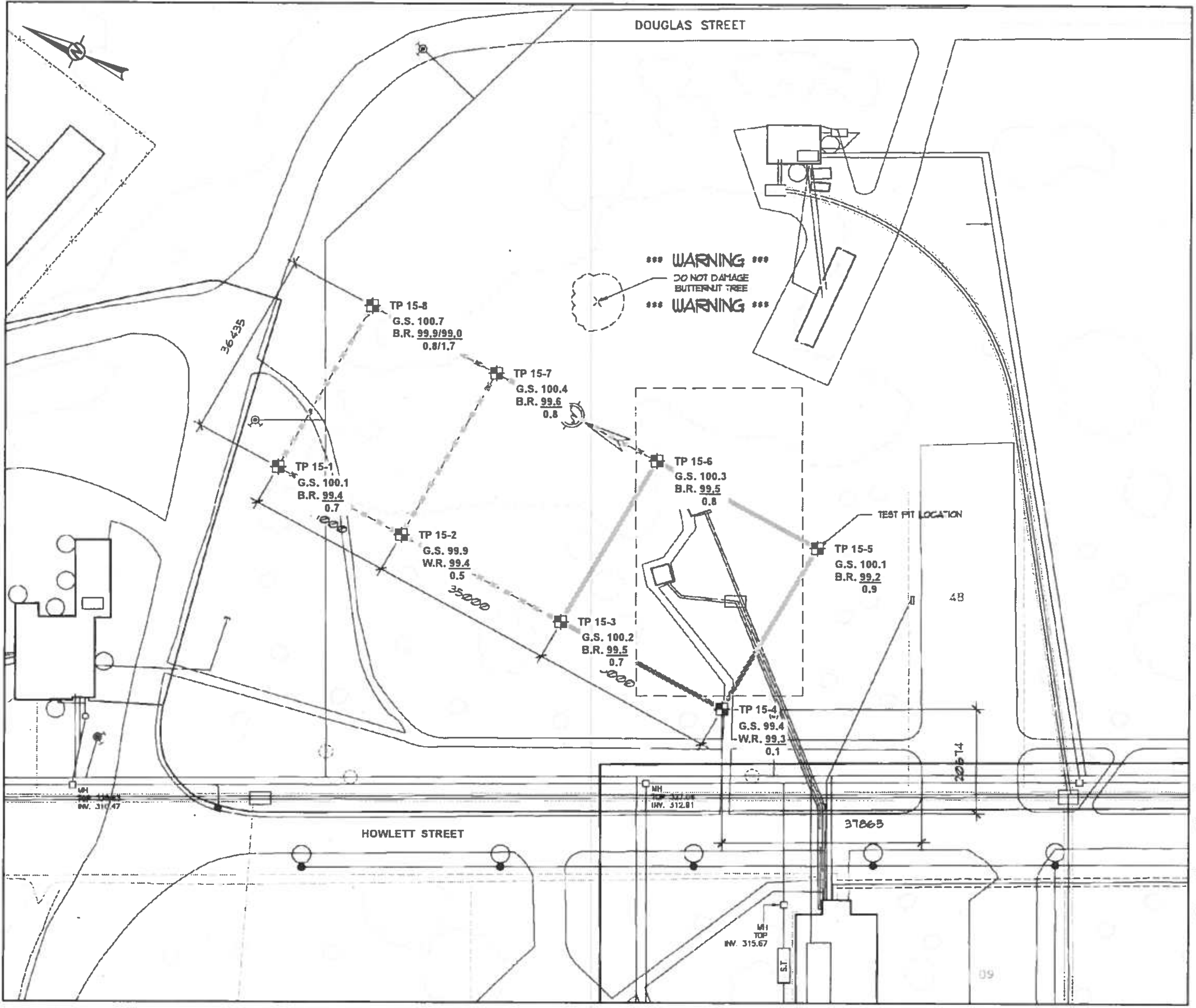
Sample	Depth (m)
1	0.2 – 0.4
2	0.4 – 0.8

15-7 (100.4 metres)	0.0 – 0.4	TOPSOIL
	0.4 – 0.8	(SM) gravelly SILTY SAND; brown, with cobbles, (GLACIAL TILL); non-cohesive, moist
	0.8	Practical refusal on BEDROCK
		Notes: Test pit dry upon completion

Sample	Depth (m)
1	0.4 – 0.8

15-8 (100.7 metres)	0.0 – 0.2	TOPSOIL
	0.2 – 0.8/1.7	(SM) gravelly SILTY SAND; brown, with cobbles and boulders, (GLACIAL TILL); non-cohesive, moist
	0.8/1.7	Practical refusal on BEDROCK
		Notes: 1) A step in the bedrock was encountered within the test pit. The depth to rock varied between 0.8 and 1.7 m 2) Test pit dry upon completion

Sample	Depth (m)
1	0.2 – 0.8



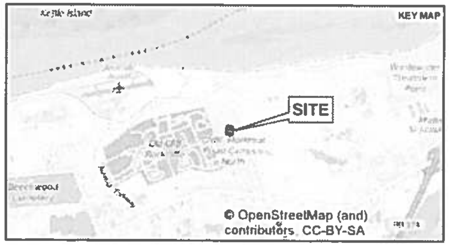
LEGEND

- APPROXIMATE TEST PIT LOCATION
- PROPOSED BUILDING M-26
- FUTURE EXPANSION

G.S. 100.4 GROUND SURFACE ELEVATION, METRES

W.R. 100.0 WEATHERED ROCK - ELEVATION, METRES
0.4 WEATHERED ROCK - DEPTH, METRES

B.R. 100.0 REFUSAL ON BEDROCK - ELEVATION, METRES
0.4 REFUSAL ON BEDROCK - DEPTH, METRES



© OpenStreetMap (and) contributors, CC-BY-SA

0 5 10 20 30 40 50
1:750
METRES

NOTE(S)
1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 1543311

REFERENCE(S)
1. SITE PLAN PROVIDED BY NRC.
2. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83
COORDINATE SYSTEM: UTM ZONE 18 VERTICAL DATUM: CGVD28

CLIENT
NATIONAL RESEARCH COUNCIL CANADA (NRC)

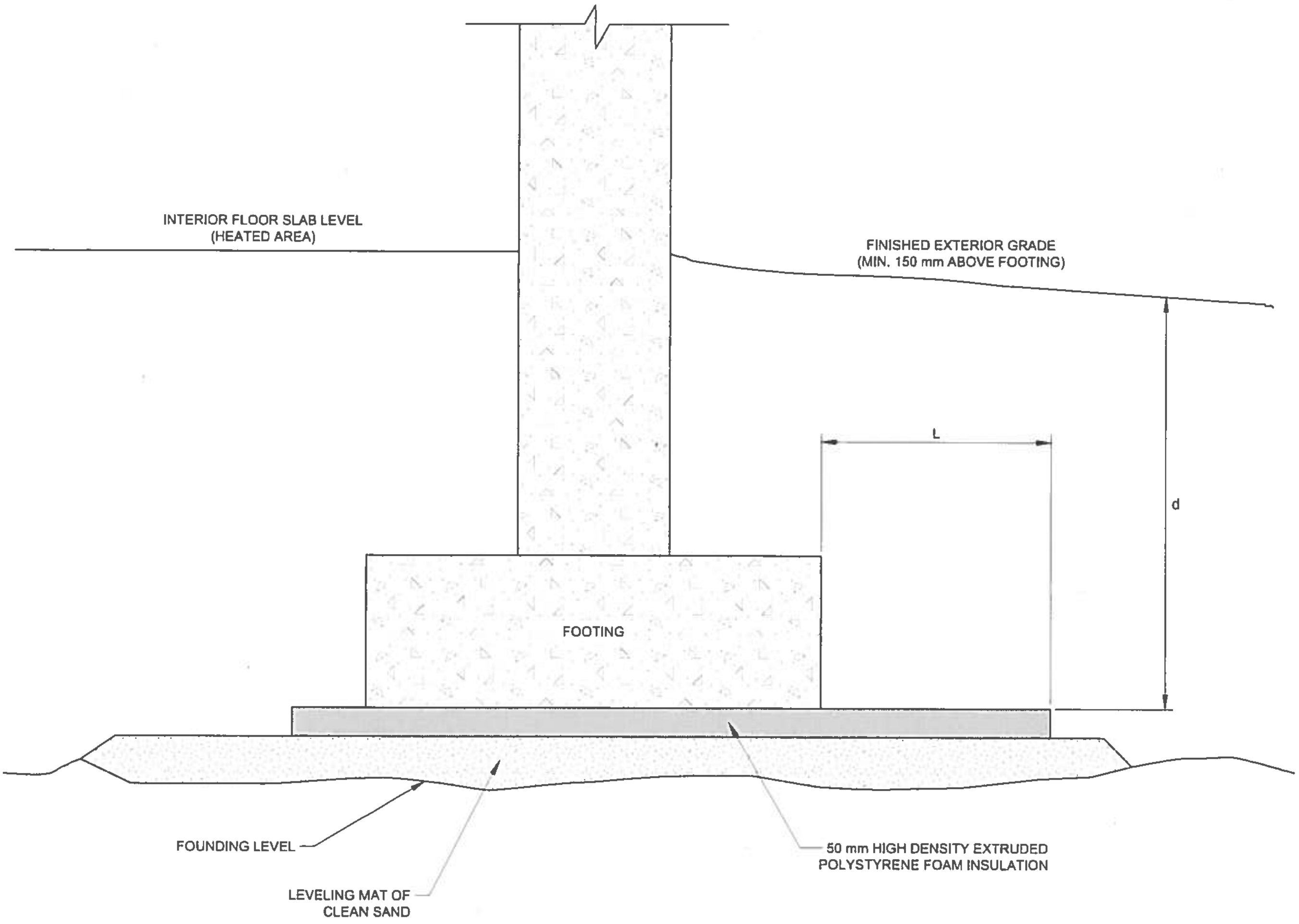
PROJECT
GEOTECHNICAL INVESTIGATION, PROPOSED FLEXIBLE LABORATORY FACILITY (M-26), NRC MONTREAL ROAD CAMPUS
1200 MONTREAL ROAD, OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT
Golder Associates

YYYY-MM-DD	2016-01-04
DESIGNED	—
PREPARED	PJM
REVIEWED	SD
APPROVED	TMS

PROJECT NO.	PHASE	REV.	FIGURE
1543311	—	A	1



LEGEND

d	THICKNESS OF EARTH COVER ABOVE TOP OF INSULATION
L	PROJECTED LENGTH OF INSULATION

NOTES

1. INSULATION JOINTS TO BE GLUED AND / OR LAPPED
2. FOR ADEQUATE FROST PROTECTION d + L ≥ 1.5 m
3. FOR d > 0.9 m, INSULATION THICKNESS OF 25 mm IS ADEQUATE
4. ALLOWABLE BEARING PRESSURE FOR FOOTING DESIGN IS DEPENDANT ON INSULATION TYPE - SEE REPORT FOR BEARING PRESSURE INFORMATION
5. FOR ISOLATED UNHEATED FOUNDATIONS, ADDITIONAL DETAILS ARE REQUIRED
6. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 1543311
7. NOT TO SCALE

CLIENT
NATIONAL RESEARCH COUNCIL CANADA (NRC)

PROJECT
GEOTECHNICAL INVESTIGATION, PROPOSED FLEXIBLE LABORATORY FACILITY (M-26), NRC MONTREAL ROAD CAMPUS
1200 MONTREAL ROAD, OTTAWA, ONTARIO

CONSULTANT

Golder Associates

YYYY-MM-DD	2015-10-20
PREPARED	PJM
DESIGN	SD
REVIEW	SD
APPROVED	TMS

TITLE
TYPICAL FOOTING INSULATION DETAIL

PROJECT No.	PHASE	Rev.	FIGURE
1543311	----	A	2

APPENDIX A

Results of MASW Shear Wave Velocity Testing

Golder Associates

TECHNICAL MEMORANDUM

DATE December 23, 2015 **PROJECT No.** 1543311

TO Stephen Dunlop
Golder Associates Ltd.

FROM Stephane Sol, Christopher Phillips **EMAIL** ssol@golder.com;cphillips@golder.com

NBCC SEISMIC SITE CLASS TESTING RESULTS PROPOSED FLEXIBLE RESEARCH FACILITY 1200 MONTREAL ROAD, OTTAWA

This technical memorandum presents the results of a Multichannel Analysis of Surface Waves (MASW) test performed for the purpose of the 2010 National Building Code of Canada (NBCC2010) Seismic Site Classification for a proposed research facility, NRC Montreal Road Campus at 1200 Montreal Road in Ottawa, Ontario. The geophysical testing was performed by Golder personnel on November 26, 2015.

Methodology

The MASW method measures variations in surface-wave velocity with increasing distance and wavelength and can be used to infer the rock/soil types, stratigraphy and soil conditions.

A typical MASW survey requires a seismic source, to generate surface waves, and a minimum of two geophone receivers, to measure the ground response at some distance from the source. Surface waves are a special type of seismic wave whose propagation is confined to the near surface medium.

The depth of penetration of a surface wave into a medium is directly proportional to its wavelength. In a non-homogeneous medium, surface waves are dispersive, i.e., each wavelength has a characteristic velocity owing to the subsurface heterogeneities within the depth interval that particular wavelength of surface wave propagates through. The relationship between surface-wave velocity and wavelength is used to obtain the shear-wave velocity and attenuation profile of the medium with increasing depth.

The seismic source used can be either active or passive, depending on the application and location of the survey. Examples of active sources include explosives, weight-drops, sledge hammer and vibrating pads. Examples of passive sources are road traffic, micro-tremors, and water-wave action (in near-shore environments).

The geophone receivers measure the wave-train associated with the surface wave travelling from a seismic source at different distances from the source.

The participation of surface waves with different wavelengths can be determined from the wave-train by transforming the wave-train results into the frequency domain. The surface-wave velocity profile with respect to wavelength (called the 'dispersion curve') is determined by the delay in wave propagation measured between the geophone receivers. The dispersion curve is then matched to a theoretical dispersion curve using an iterative forward-modelling procedure. The result is a shear-wave velocity profile of the tested medium with depth, which can be used to estimate the dynamic shear-modulus of the medium as a function of depth.

Field Work

The MASW field work was conducted on November 26, 2015, by personnel from the Golder Mississauga office. The MASW line, a series of 24 low frequency (4.5 Hz) geophones, was laid out at 3-metre intervals. Both active and passive readings were recorded at this site. For the active investigation, a seismic drop of 45 kg and a 9.9 kg sledge hammer were used as seismic sources. Active seismic records were collected with seismic sources located 5, 10, 15, and 20 metres from and collinear to the geophone array. An example of an active seismic record collected is shown in Figure 1 (below).

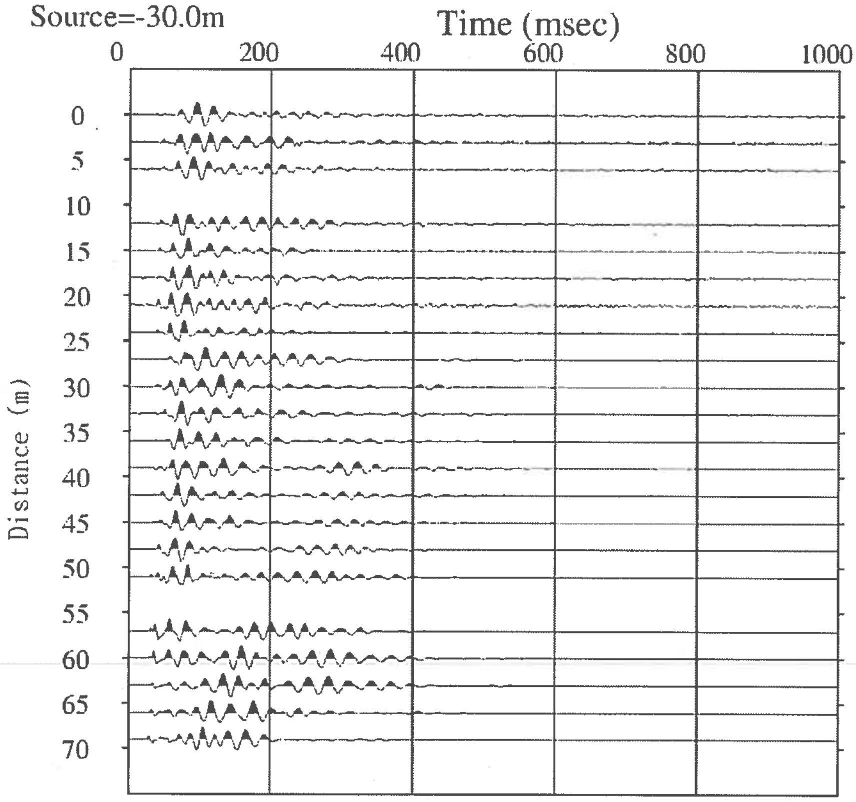


Figure 1: Typical seismic record collected at the site.

Data Processing

Processing of the MASW test results consisted of the following main steps:

1) Transformation of the time domain data into the frequency domain using a Fast-Fourier Transform (FFT) for each source location;

2) Calculation of the phase for each frequency component;

3) Linear regression to calculate phase velocity for each frequency component;

4) Filtering of the calculated phase velocities based on the Pearson correlation coefficient (r2) between the data and the linear regression best fit line used to calculate phase velocity;

5) Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and,

6) Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.

Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for a seismic shot point were combined and the dispersion curve generated by choosing the minimum phase velocity calculated for each frequency component as shown on Figure 2. Shear-wave velocity profiles were generated through inverse modelling to best fit the calculated dispersion curves. The active survey provided a dispersion curve with a suitable frequency range (12 to 41 Hz), providing information for both shallow and deeper depths. The minimum measured surface-wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 12 Hz.

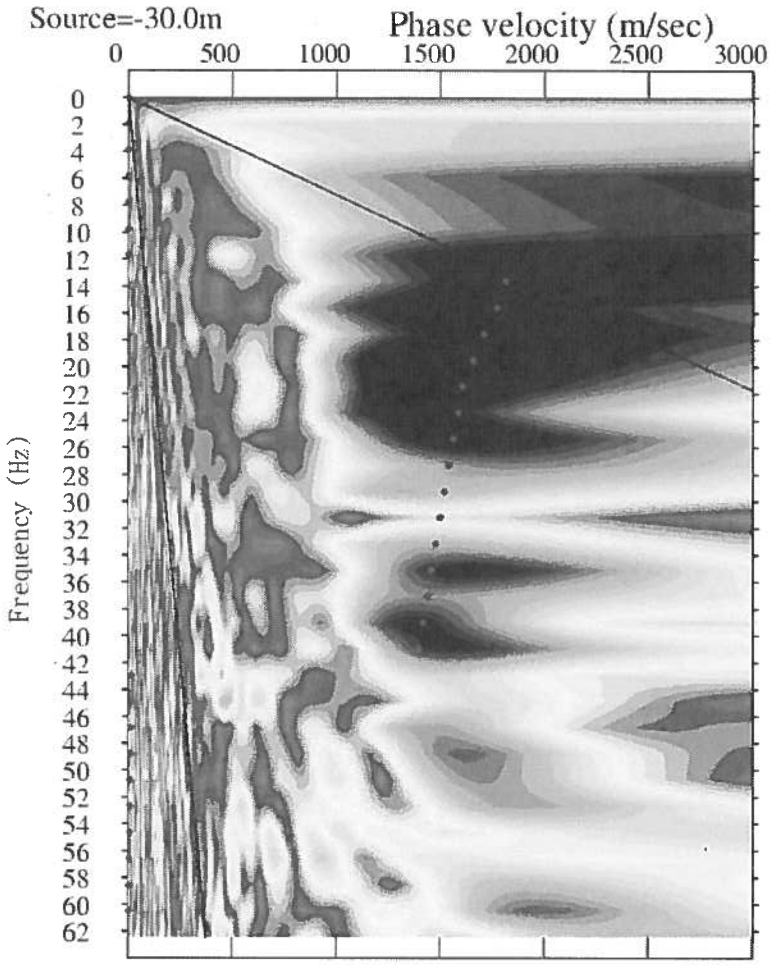


Figure 2: Active MASW Dispersion Curve Picks (red dots).

Results

The MASW test results are presented in Figure 3, which presents the calculated shear-wave velocity profile derived from the field testing. The results have been inferred using weight-drop located at 10 metres from the last geophone. The field collected dispersion curves are compared with the model generated dispersion curves on Figure 4. There is a good correlation between the field collected and model calculated dispersion curves, with a root mean squared error of less than 1%.

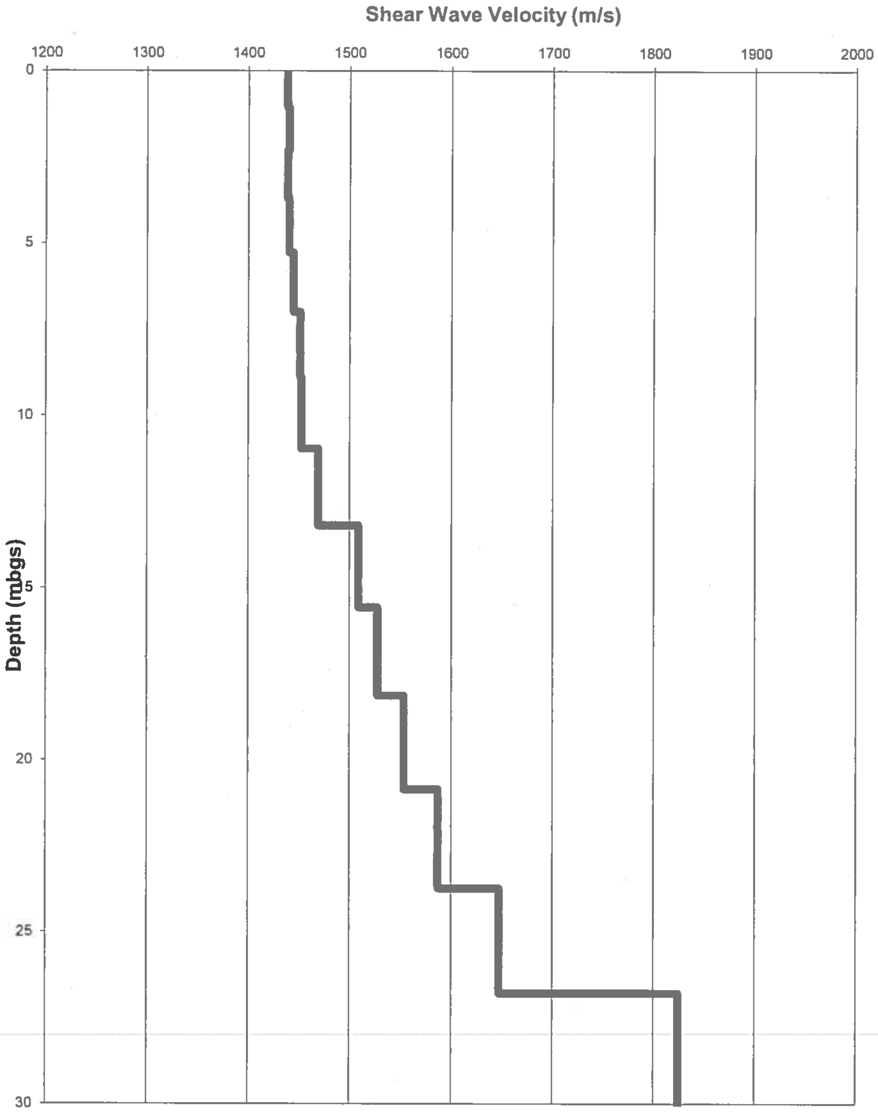


Figure 3: MASW Modelled Shear-Wave Velocity Depth profile.

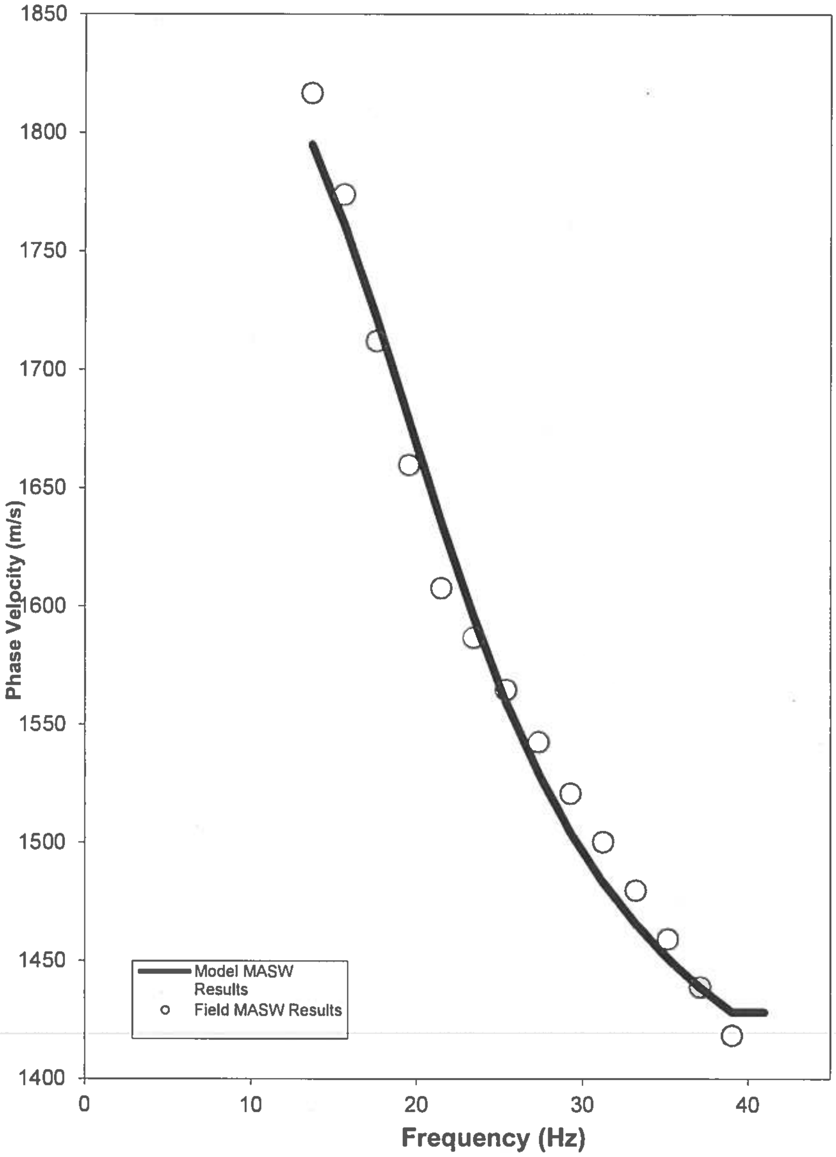


Figure 4: Comparison of Field (red dots) vs. Modelled Data (blue line).

To calculate the average shear-wave velocity as required by the NBCC2010, the results were modelled to 30 metres below ground surface. The average shear-wave velocity was found to be 1536 m/s (Table 1).

Table 1: Shear-Wave Velocity Profile

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.07	1.07	1439	0.000745
1.07	2.31	1.24	1441	0.000858
2.31	3.71	1.40	1439	0.000974
3.71	5.27	1.57	1441	0.001087
5.27	7.01	1.73	1445	0.001198
7.01	8.90	1.90	1451	0.001306
8.90	10.96	2.06	1453	0.001418
10.96	13.19	2.23	1470	0.001514
13.19	15.58	2.39	1509	0.001584
15.58	18.13	2.55	1528	0.001672
18.13	20.85	2.72	1554	0.001750
20.85	23.74	2.88	1587	0.001817
23.74	26.79	3.05	1648	0.001851
26.79	30.00	3.21	1825	0.001762
			Vs Average to 30 mbgs (m/s)	**1536**

Closure

We trust that this technical memorandum meets your needs at the present time. If you have any questions or require clarification, please contact the undersigned at your convenience.

GOLDER ASSOCIATES LTD.

Stephane Sol, Ph.D, P. Geo.
Senior Geophysicist

Christopher Phillips, M. Sc., P. Geo.
Senior Geophysicist, Associate

SS/CRP/jl

\\golder.gds\gal\ottawa\active\2015\3 proj\1543311 nrc 1200 montreal flexible rf ottawa\13_survey data\geophysics\report\1543311 tech memo 2015dec22 masw.docx

APPENDIX B

Results of Chemical Analysis
EXOVA Laboratories Report No. 1524472

EXOVA ENVIRONMENTAL ONTARIO

Certificate of Analysis

Client: Golder Associates Ltd. (Ottawa)
1931 Robertson Road
Ottawa, ON
K2H 5B7
Attention: Mr. Steve Dunlop
PO#:
Invoice to: Golder Associates Ltd. (Ottawa)

Report Number: 1524472
Date Submitted: 2015-12-15
Date Reported: 2015-12-18
Project: 1543311
COC #: 804149

				Lab I.D.	1219053
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2015-12-14
				Sample I.D.	TP 15-5 SA 2/0.6-0.86m
Group	**Analyte**	**MRL**	**Units**	**Guideline**	
Agri. - Soil	pH	2.0			8.0
General Chemistry	Cl	0.002	%		<0.002
	Electrical Conductivity	0.05	mS/cm		0.32
	Resistivity	1	ohm-cm		3120
	SO4	0.01	%		0.03

Guideline = *** = Guideline Exceedence**

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).
Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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South America	+ 55 21 3095 9500

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Golder Associates

July 2008

LIMITED REPORT ON

Geotechnical Considerations NRC CO, H2 and N2 Docking and Piping Facility NRC Montreal Road Campus Blair Road Ottawa, Ontario

Submitted to:
National Research Council Canada
Building M-19
120 Montreal Road
Ottawa, Ontario
K1A 0R6

REPORT

Report Number: 08-1121-0099

Distribution:

4 copies- National Research Council Canada
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Golder
Associates

Golder Associates

July 23, 2008

Project No. 08-1121-0099

Mr. Bruno Vallieres, Administrative Services and Property Branch
National Research Council Canada
Building M-19
120 Montreal Road
Ottawa, Ontario
K1A 0R6

**RE: NRC CO, H2 AND N2
DOCKING AND PIPING FACILITY
NRC MONTREAL ROAD CAMPUS
BLAIR ROAD
OTTAWA, ONTARIO**

Dear Mr. Vallieres

Please find attached our limited report on geotechnical considerations for the proposed Docking and Piping Facility to be constructed at the NRC Montreal Road Campus, Blair Road, Ontario.

We trust that this limited report is sufficient for your present requirements. If you have any questions concerning this limited report or, if we can be of further assistance, please let us know.

Yours truly,

GOLDER ASSOCIATES LTD.

M.W. St-Louis, P.Eng.
Senior Geotechnical Engineer

T.J. Nicholas, P.Eng.
Principal

MSTL/TJN/ch

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GEOTECHNICAL CONSIDERATIONS

1.0 INTRODUCTION

This limited report addresses geotechnical consideration related to the site of the Docking and Piping Facility to be located on the NRC Montreal Road Campus, Blair Road, Ottawa (see Figure 1, Key Plan). Geotechnical studies had been prepared by McRostie Genest St-Louis (MGS) in 2002 and 2005 (reference reports SF-4553B and SF-4932). The results of the pertinent subsurface information from the above studies are included in this report for completeness.

The purpose of this assignment was to review the general soil and groundwater conditions in the area of the proposed duct bank routes for the docking and piping facility by means of an additional four (4) boreholes (08-1 to 08-4 inclusive) and fourteen (14) test pits (08-5 to 08-17 inclusive and 08-15A) and, based on an interpretation of factual information including that from past subsurface records obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The reader is referred to the "Important Information and Limitations of this Report", which follows the text but forms and integral part of this document.

2.0 DESCRIPTION OF PROJECT

Plans are being prepared to construct a docking and piping facility at the NRC Montreal Road Campus (see Figure 1, Key Plan). The project will include duct banks within about 2.5 metres of the existing ground surface, foundations for a nitrogen tank that will be about 12 metres in height and 3 metres in diameter supported on three (3) legs, and 3 blast walls to be in compliance with NFP 55 requirements in the docking facility.

Geological mapping indicates that the bedrock underlying this site is sedimentary in nature and consists of limestone of the Bobcaygeon formation.

The site also falls within the Western Québec Seismic Zone (WQSZ) according to Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montréal to Témiscaming, and which encompasses the Ottawa area. Within the WQSZ, recent seismic activity has been concentrated in two (2) subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the WQSZ from 1900 to 2000 includes the 1935 Témiscaming event which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2 and in 1944, a Cornwall-Massena event had a magnitude of 5.6. In comparison with other seismically active areas in the world (i.e., California, Japan and New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

Under the 2006 Ontario Building Code (OBC), a seismic hazard with a 2% probability of exceedance in 50 years has been retained for design. For the subject site, the reference (Site Class C) peak horizontal ground acceleration (PGA) is 0.42g (g = acceleration by gravity) (Adams and Halchuck, 2003).

3.0 PROCEDURE

The field work for this investigation was carried out on June 18, 2008 (test pits) and on July 3 and 4, 2008 (boreholes). At that time fourteen (14) test pits (numbered 08-5 to 08-17 inclusive and 08-15A) and four (4) boreholes (numbered 08-1 to 08-4 inclusive) were put down at the approximate locations shown on the Site Plan, Figure 2.

The test pits were excavated by a rubber tired backhoe. The test pits were advanced to depths of between 0.4 and 2.2 metres below the existing ground surface.

The boreholes were advanced using a track-mounted CME 45 hollow-stem auger drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of between 2.8 and 3.5 metres below the existing ground surface.

Within the boreholes, standard penetration tests (SPT) were carried out at regular intervals of depth and samples of the soils encountered were recovered using drive open sampling equipment. All four (4) boreholes were advanced through the overburden and into the underlying limestone bedrock. In all boreholes, the limestone bedrock was proven for a depth of between 1.5 and 1.7 metres by rotary core drilling in NQ size.

The field work was supervised by an experienced technician from our staff who directed the drilling operations, logged the test pits, the boreholes and samples, directed the in-situ testing and took custody of the soil samples and rock cores.

On completion of the drilling operations, the soil samples and rock cores were transported to our laboratory.

A standpipe was installed in boreholes 08-1 and 08-3 to determine the stabilized groundwater conditions at the site. The groundwater level in the standpipe was measure on July 9, 2008.

The borehole and test pit locations were selected by the National Research Council. Subsequently, the locations and ground surface elevations for the test pits and boreholes for this subsurface investigation were surveyed by Stantec Geomatics Ltd. The ground surface elevations supplied to Golder Associates are understood to be referenced to Geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered during the present 2008 investigation are shown on the Record of Borehole and Record of Test Pit sheets in Appendix A.

The subsurface information from previous studies was compiled as part of the present study and is included in Appendix B.

The subsurface conditions at this site can be generalized as consisting of surficial deposits of topsoil and fill material underlain by glacial till in turn underlain by limestone bedrock. The depth to bedrock is variable at this site.

The following sections provide a more detailed summary of the subsurface conditions encountered within the boreholes and test pits from the present and previous investigations.

4.2 Fill Material and Topsoil

Fill material and/or topsoil were encountered at the existing ground surface and found to range in thickness between 100 millimetres to about 2.15 metres (see Test Pit 08-15). The fill material generally consists of sand, gravel, topsoil, cobbles, boulders and rock blocks but at some locations also contains wood, brick, and concrete. In test pit 08-15, tires were found within the fill.

4.3 Glacial Till

A deposit of glacial till is often found between the surficial layer of fill and/or topsoil and the bedrock surface. The glacial till consists of a heterogeneous mixture of gravel, cobbles and boulders in a matrix of silty sand with a trace of some clay. There are a few locations where no glacial till was encountered and where the fill material and/or topsoil veneers the limestone bedrock.

4.4 Limestone Bedrock

Limestone bedrock underlies the fill material and the glacial till at all boreholes put down as part of the present subsurface investigation.

The bedrock surface varies from elevation 97.5 to 99.3 metres which is about 1.0 to 2.0 metres below the existing ground surface. In borehole 08-1 and 08-3, the upper layer of bedrock was weathered and was sampled using drive open soil sampling equipment over depths of 0.3 and 0.1 metres, respectively. Below this upper bedrock layer, the degree of weathering is moderate to slight.

The Total Core Recovery (TCR) varies from about 88 to 100 percent of the length drilled. The Solid Core Recovery (SCR), the percentage of core that is completely circular in section, ranges fro 67 to 98 percent. The Rock Quality Designation (RQD), the percent length of intact core longer than 100 millimetres, varies between 50 and 77 percent

4.5 Groundwater

The groundwater levels in the two (2) boreholes with standpipes sealed into the underlying limestone bedrock (boreholes 08-1 and 08-3) were measured on July 9, 2008. At that time, groundwater levels varied from about 2.5 to 2.6 metres below the existing ground surface (i.e. about elevations 97.1 to 97.7 metres).

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

5.0 PROPOSED DOCKING AND PIPING FACILITY

5.1 General

This section of the report provides limited engineering guidelines on the geotechnical aspects of the project for the service duct banks, the foundations for the nitrogen tower and the blast wall foundations portion of the project and based on our interpretation of subsurface information and project requirements and is subject to the limitations in the "Important Information and Limitations of This Report" attachment which follows the text of this report.

The professional services retained for this project include only the geotechnical aspects of subsurface conditions at the site. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off site sources are outside the terms of reference for this project and have not been investigated nor addressed.

5.2 Excavations and Site Servicing

Excavations for the installation of site services (duct banks) will be through fill materials, topsoil, glacial till and at some locations will extend into bedrock.

No unusual problems are anticipated in trenching in the overburden using conventional hydraulic excavating equipment, although significant cobble and boulder removal could be required in the glacial till. Furthermore, large rock blocks should also be expected to be present in fill materials. Old concrete foundation walls and basement floor slabs may also be found at some locations as it is understood that buildings were demolished before the construction of the NRC Montreal Road Campus.

It is expected that the bedrock removal for the project will be carried out using drill and blast techniques. Should bedrock removal be carried out by drilling and blasting, special care will be required to prevent overblasting and fracturing of the bedrock below foundation levels.

The blasting should be controlled to limit the peak particle velocities at all adjacent structures such that blast induced damage will be avoided. This will require blast designs by a specialist in this field.

A pre-blast survey should be carried out on all surrounding structures. Selected existing interior and exterior cracks in the structure should be identified during the pre-blast survey and should be monitored for lateral or shear movements by means of glass telltales and/or movement telltales.

The contractor should be limited to only small controlled shots. The following frequency dependent vibration limits at the nearest structures and services are suggested

Frequency Range (Hz)	Vibration Limits (millimeters/second)
<10	5
10 to 40	5 to 50 (sliding scale)
>40	50

These limits should be practical and achievable for most of this project. In areas in close proximity to structures and services, limestone bedrock removal should be accomplished using mechanical methods such as hoe-ramming in conjunction with closely spaced line drilling to establish the limit of the excavation.

5.3 Foundations

It is considered that the proposed nitrogen tank structure and the three (3) blast walls will be founded on spread footings placed on limestone bedrock or by caissons extending into the limestone bedrock layers underlying the site.

For footing design purposes, footings placed directly on limestone bedrock, below any upper weathered zones, may be sized using an Ultimate Limit States (ULS) factored bearing resistance of 1000 kilopascals. Provided that the bedrock surface is properly cleaned of soil or any loose rock fragments at the time of construction, the settlement of footings sized using the above factored bearing resistance should be negligible, therefore, Serviceability Limit States (SLS) need not be considered.

Caissons, as an alternative foundation scheme, could be designed based on a rock socket to concrete bond value of 500 kilopascals (SLS); end bearing should be ignored. In addition, the bond (adhesion) in the upper weathered or fractured zone should also be ignored.

An advantage to the rock socketed caissons is their ability to be reinforced for both downward loading and uplift resistance.

5.4 Rock Anchors

If required, rock anchors could be provided to resist uplift loads on footing type foundations.

The anchors could consist of either grouted or mechanical anchors.

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. Further guidance, at the final design stage, should be provided for assessing the resistance of a single anchor and the effect of a group of anchors.

5.5 Frost Protection

All exterior foundation elements in unheated areas should be provided with a minimum of 1.5 metres of earth cover for frost protection purposes. Isolated foundations or foundations in unheated areas which are adjacent to any surface cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover.

For footings founded on competent bedrock, the requirement for 1.5 or 1.8 metres of earth cover could be waived where it could be shown by check drilling during construction that the bedrock below footing level does not contain any joints filled with frost-susceptible soil.

5.6 Duct Bank Route

The concrete encased duct bank should be made to bear on the bedrock surface or within the bedrock over the entire route for this project in order to prevent conditions of differential support and potential settlement where soil supported.

Excavation of the limestone bedrock would be required at some locations where bedrock is shallow. Lean concrete infill would be required in localized areas where the bedrock surface is somewhat deeper.

5.7 Seismic Site Response Classification

The 2006 OBC contains an updated seismic analysis and design methodology which uses a seismic site response classification system defined by the shear stiffness of the upper 30 metres of ground of interest. Seismic response is now defined by uniform hazard spectra (UHS) corresponding to design earthquake with a probability of exceedance of 2% in 50 years. There are six site classes (from A to F), decreasing in soil stiffness from A (hard rock) to E (soft soil); Site Class F denotes problematic soils for which a site-specific evaluation is required. The site class is used to obtain soil factors (F_a and F_v) used to modify the UHS to account for the effects of site-specific soil conditions on the seismic response of the site to the design earthquake.

To support a site class designation, a shear wave velocity of 700 metres per second was assigned to the limestone bedrock, based on actual measurements in similar bedrock formations. Interpreting the data available indicates that a Site Class C designation would be appropriate. It may be possible to achieve a higher Site Class designation by obtaining site specific shear wave velocities.

5.8 Corrosion and Cement Type

As part of several studies performed by McRostie Genest St.-Louis (MGS) at the NRC Montreal Road Campus over the years, groundwater samples were collected and submitted for chemical analysis related to potential corrosion of buried ferrous elements and sulphate attack on buried concrete elements.

There has not been a history of potential problems with corrosion of exposed ferrous elements or sulphate attack on buried concrete elements.

Based on the past performance of older existing foundations exposed at the time of recent additions to the NRC Montreal Road Campus, concrete made with Type GU Portland cement should be acceptable for substructures.

6.0 ADDITIONAL CONSIDERATIONS

All foundation areas and duct bank trenches should be inspected by experienced geotechnical personnel prior to concreting to ensure that the limestone bedrock having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared including the removal of fractured bedrock by overblasting.

At the time of writing this report, only conceptual details of the proposed docking and piping facility were available.

We trust that this limited report that only cover the geotechnical aspects within the latter is sufficient for your present requirements. If you have any questions concerning this report or require additional geotechnical recommendations, please call us.

Report Signature Page

GOLDER ASSOCIATES LTD.

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Principal

MSTL/TJN/sr

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder can not be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

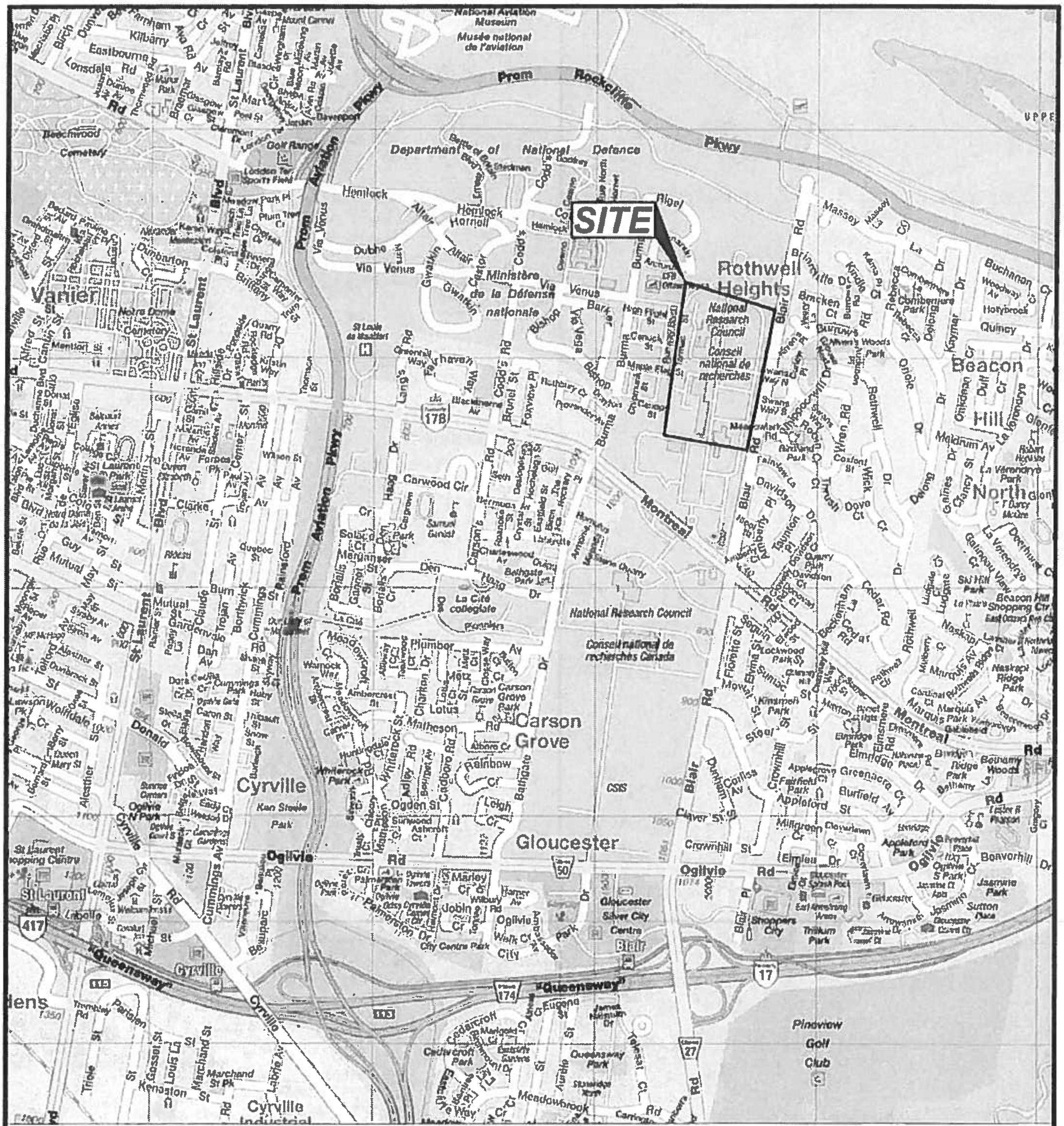
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT

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Ottawa, Ontario

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PROJECT No 08-1121-0099 REV 0

TITLE

KEY PLAN

GEOTECHNICAL INVESTIGATION - NRC CO, H_2 & N_2
DOCKING FACILITY, MONTREAL ROAD, OTTAWA, ONTARIO

FIGURE 1

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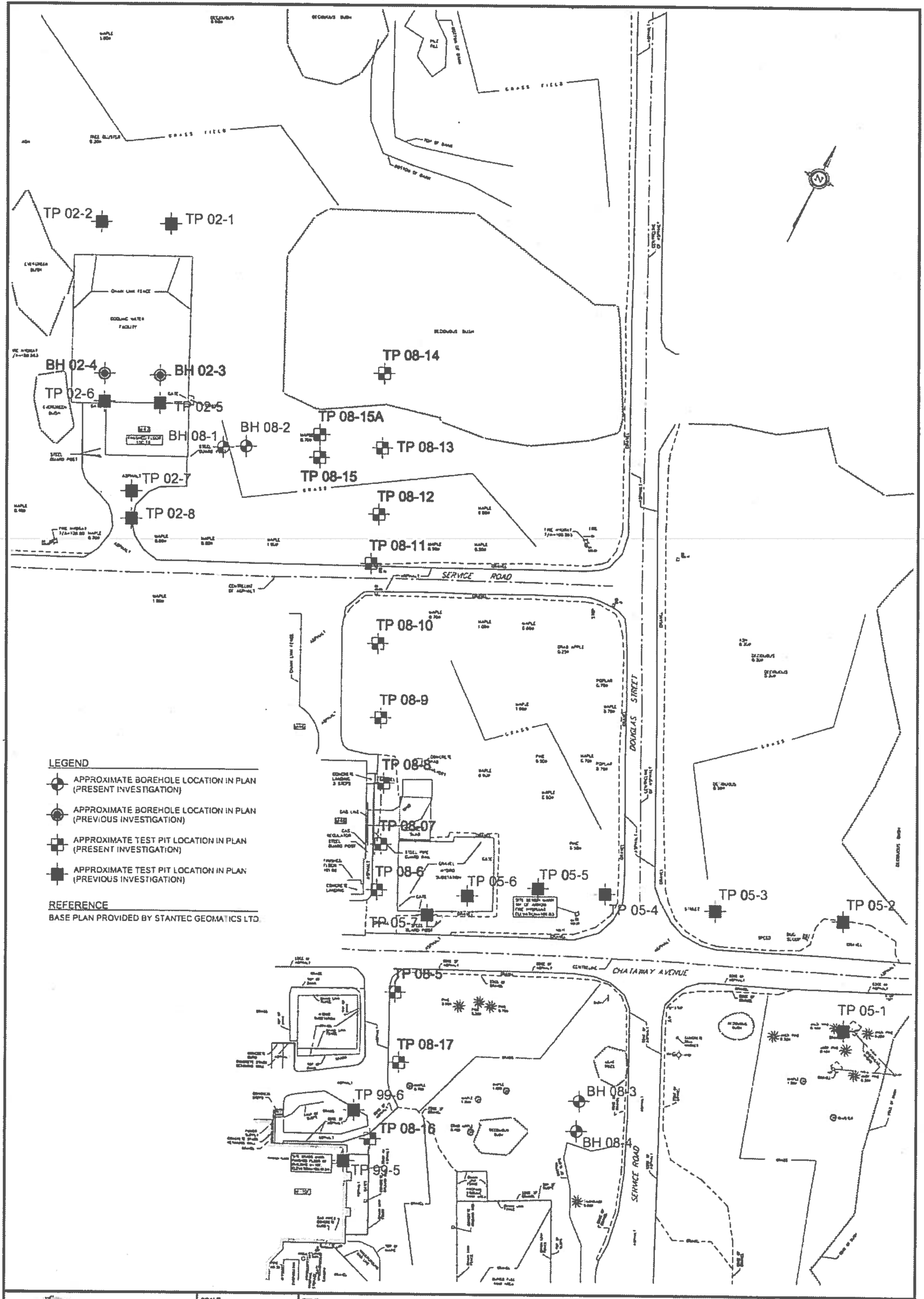

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SITE PLAN

GEOTECHNICAL INVESTIGATION - NRC CO, H$_2$ & N$_2$ DOCKING FACILITY
MONTREAL ROAD, OTTAWA, ONTARIO

FIGURE 2

APPENDIX A

Abbreviations and Symbols
Record of Borehole and Test Pit Sheets

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open Sampler for a distance of 300 mm (12 in.)

DD- Diamond Drilling

Dynamic Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive Uncased a 50 mm (2 in.) diameter, 60^0 cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Peizo-Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60^0 conical tip and a projected end area of $10\ cm^2$ pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded Electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils $C_{u2}S_u$

Consistency	Kpa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limited
w_l	liquid limit
C	consolidaiton (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	modified Proctor compaction test
SPC	standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:

1. Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	= 3.1416
ln x, natural logarithm of x	
$\log_{10}$ x or log x, logarithm of x to base 10	
g	Acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma'$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma''$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1\sigma_2\sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress = $(\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma'=\gamma-\gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R= p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is p. Unit weight symbol is γ where γ=pg(i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity Index=(w_l-w_p)
w_s	shrinkage limit
I_L	liquidity index=$(w-w_p)/I_p$
I_c	consistency index=$(w_l-w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index-$(e_{max}-e)/(e_{max}-e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio=σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p\tau_r$	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction=tan δ
c'	effective cohesion
c_u, S_u	undrained shear strength (ϕ=0 analysis)
p	mean total stress $(\sigma_1+\sigma_3)/2$
p'	mean effective stress $(\sigma'_1+\sigma'_3)/2$
q	$(\sigma_1-\sigma_3)/2$ or $(\sigma'_1-\sigma_3)/2$
q_u	compressive strength $(\sigma_1-\sigma_3)$
S_t	sensitivity

Notes: 1. τ=c'σ' tan |'

2. Shear strength=(Compressive strength)/2

PROJECT: 08-1121-0099

RECORD OF BOREHOLE: BH 08-1

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: 4 July 2008

DATUM:

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa (20 40 60 80) nat V. + Q - ● rem V. ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		100.26							
0	Power Auger 200 mm Diam. (Hollow Stem)	Brown sand and gravel, some cobbles and boulders (FILL)		0.00							Native Backfill and Silica Sand
				99.44		50 DO	31				
		Grey brown SILTY SAND, some gravel		99.25	1A	50 DO	31				
1		Weathered Grey LIMESTONE BEDROCK		1.01							
	Rotary Drill NQ Core										Bentonite Seal
2					2	NQ RC	DD	T C R (%) 100 S C R (%) 83 R Q D (%) 50			Silica Sand Slot Screen
				97.43							
		End of Borehole		2.83							
3											W.L. in screen at elev. 97.73 m on July 9, 2008
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: J.D.

CHECKED:

PROJECT: 08-1121-0099

RECORD OF BOREHOLE: BH 08-2

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: July 3, 2008

DATUM:

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV / DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		100.20							
	Power Auger 130 mm Diam (Hollow Stem)	FILL and GLACIAL TILL, with cobbles and boulders	▨	0.00	1	GRAB	-				
					2	NQ RC	DD				
1				99.10	3	50 DO	30				
	Rotary Drill NQ Core	Weathered Grey LIMESTONE BEDROCK		1.10							
					4	NQ RC	DD	T.C.R. (%) 97 S.C.R. (%) 82 R.Q.D. (%) 77			
2											
					5	NQ RC	DD	T.C.R. (%) 88 S.C.R. (%) 87 R.Q.D. (%) 52			
		End of Borehole		97.40 2.77							
3											
4											
5											
6											
7											
8											
9											
10											

BOREHOLE 08112-0099.GPJ HYDROGEO.GDT 7/22/08

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: J.D.

CHECKED:

PROJECT: 08-1121-0099

RECORD OF BOREHOLE: BH 08-3

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: July 3, 2008

DATUM:

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V + Q - ● rem V ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10⁻⁶ 10⁻⁵ 10⁻⁴ 10⁻³ / WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 10 20 30 40	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface	99.67							
0	Power Auger 200 mm Diam. (Hollow Stem)	Loose brown sand and gravel, some cobbles and boulders (FILL)	0.00							
				1	50 DO	13				
1		Compact grey brown SILTY SAND, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)	98.71 0.96	1A	50 DO	13				Native Backfill and Silica Sand
		Weathered grey LIMESTONE BEDROCK, some mud seams	98.15 1.52	2	50 DO	30				
2	Rotary Drill NQ Core									Bentonite Seal
				3	NQ RC	DD	T C R (%) 100 S C R (%) 98 R Q D (%) 73			Silica Sand
3										Slot Screen
		End of Borehole	96.20 3.47							
4										W.L. in screen at elev. 97.05 m on July 9, 2008
5										
6										
7										
8										
9										
10										

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: J.D.

CHECKED:

PROJECT: 08-1121-0099

RECORD OF BOREHOLE: BH 08-4

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: 3 July 2008

DATUM:

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa (20 40 60 80) nat V + Q - ● rem V ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s (10⁻⁶ 10⁻⁵ 10⁻⁴ 10⁻³); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		99.44							
0	Power Auger 200 mm Diam. (Hollow Stem)	Loose brown sand and gravel, some cobbles and boulders (FILL)		0.00							
1		Compact grey brown SILTY SAND, some gravel, cobbles, boulders, trace clay (GLACIAL TILL)		98.59 0.85	1A 1	GRAB 50 DO	- 12				
					2	50 DO	60				
2	Rotary Drill NQ Core	Weathered grey LIMESTONE BEDROCK, some mud seams		97.49 1.95							
3					3	NQ RC	DD	T.C.R. (%) 100 S.C.R. (%) 94 R.Q.D. (%) 71			
		End of Borehole		95.97 3.47							
4											
5											
6											
7											
8											
9											
10											

BOREHOLE 0811210099.GPJ HYDROGEO.GDT 14/7/08

DEPTH SCALE

1 : 50

Golder Associates

LOGGED J.D.

CHECKED

TABLE 1

RECORD OF TEST PITS

Test Pit Number	Depth (Metres)	Description
TP 08-5	0.00 – 0.30	FILL – Brown SAND, GRAVEL and TOPSOIL
(Elevation	0.30 – 1.55	Brown SANDY TILL
100.55 m)	1.55	End of test pit. Refusal on BEDROCK.
TP 08-6	0.00 – 0.20	FILL – Crushed Limestone
(Elevation	0.20 – 0.35	Light brown fine SAND
100.44 m)	0.35 – 1.52	Medium dense brown SANDY TILL
	1.52	End of test pit. Refusal on BEDROCK.
TP 08-7	0.00 – 0.20	FILL – Crushed LIMESTONE
(Elevation	0.20 – 1.30	FILL – Brown SAND, BOULDERS, GRAVEL, pieces of WOOD and BRICK
100.39 m)		
	1.30 – 1.90	Brown SANDY TILL
	1.90	End of test pit. Refusal on BEDROCK.
TP 08-8	0.00 – 0.25	FILL – Crushed LIMESTONE
(Elevation	0.25 – 0.80	FILL – Brown SAND, GRAVEL, and pieces of BRICK
100.31 m)	0.80 – 1.50	Brown SANDY TILL
	1.50	End of test pit. Refusal on BEDROCK.
TP 08-9	0.00 – 1.00	FILL – Black TOPSOIL, SAND, GRAVEL, and BRICK
(Elevation	1.00	End of test pit. Refusal on BEDROCK.
100.11 m)		
TP 08-10	0.00 – 0.30	Black TOPSOIL
(Elevation	0.30 – 0.70	Brown SANDY TILL
99.89 m)	0.70	End of test pit. Refusal on BEDROCK.
TP 08-11	0.00 – 0.60	FILL – SAND, GRAVEL and BRICK
(Elevation	0.60 – 1.00	WEATHERED BEDROCK
99.77 m)	1.00	End of test pit. Refusal on BEDROCK.

TABLE 1 (continued)

TP 08-12 (Elevation 100.06 m)	0.00 – 0.30	Black TOPSOIL and pieces of WEATHERED ROCK
	0.30 – 0.50	WEATHERED BEDROCK
	0.50	End of test pit. Refusal on BEDROCK.
TP 08-13 (Elevation 100.21 m)	0.00 – 0.15	Black TOPSOIL
	0.15 – 0.65	Brown SANDY TILL
	0.65 – 0.80	WEATHERED BEDROCK
	0.80	End of test pit. Refusal on BEDROCK.
TP 08-14 (Elevation 100.53 m)	0.00 – 0.30	Black TOPSOIL
	0.30 – 0.45	WEATHERED BEDROCK
	0.45	End of test pit. Refusal on BEDROCK.
TP 08-15 (Elevation 100.18 m)	0.00 – 0.30	Black TOPSOIL
	0.30 – 2.15	FILL – SAND, GRAVEL, pieces of CONCRETE, BRICK, TIRES
	2.15	End of test pit. Refusal on BEDROCK.
TP 08-15A (Elevation 100.18 m)	0.00 – 0.20	Black TOPSOIL
	0.20 – 0.35	WEATHERED BEDROCK
	0.35	End of test pit. Refusal on BEDROCK.
TP 08-16 (Elev. 100.67 m)	0.00 – 0.60	FILL – Crushed LIMESTONE
	0.60 – 1.70	Light brown SANDY TILL
	1.70	End of test pit. Refusal on BEDROCK.
TP 08-17 (Elev. 100.77 m)	0.00 – 0.30	FILL – Crushed LIMESTONE
	0.30 – 0.80	Dark brown SAND and GRAVEL
	0.80 – 2.20	Light brown SANDY TILL
	2.20	End of test pit. Refusal on BEDROCK.

APPENDIX B

Boreholes and Test Pits from Previous Studies

NRC – NEW ELECTRICAL SUB-STATION	B.M.(ELEV 100.20m)geodetic: Floor at building M-10 at door No. 11	TEST PIT NO: 05-1
		PROJECT NO: E-8890
START DATE: 05/09/02		ELEVATION: 97.54 m

SAMPLE TYPE ■ REMOULDED ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▯▯ NO RECOVERY ▮ CORE

DEPTH(m)	SMALL PEN. (kPa) SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(m)
0.0	sides stable			TOPSOIL and ROOTS		
				97.24 medium dense sandy TILL		97.0
	no water seepage			Bottom of test pit on possible rock 96.74		
1.0						
						96.0
2.0						
						95.0
3.0						
						94.0
4.0						

McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 0.8 m
	REVIEWED BY: E.S.	COMPLETE: 05/09/02
	Fig. No: 2	Page 1 of 1

05/09/20 08:07AM (STD-SHTP)

NRC – NEW ELECTRICAL SUB-STATION	B.M.(ELEV 100.20m) geodetic: Floor at building M-10, at door No.11	TEST PIT NO: 05-2
		PROJECT NO: E-8890
START DATE: 05/09/02		ELEVATION: 98.02 m

SAMPLE TYPE ■ REMOULDED ⧄ SHELBY TUBE ☒ SPLIT-SPOON ☰ PROBING [ ] NO RECOVERY [▮] CORE

DEPTH(m)	SMALL PEN. (kPa) / SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(m)
0.0	sides stable			FILL – crushed limestone		98.0
				97.67 medium dense sandy TILL		
	no water seepage			Bottom of test pit on possible rock 97.42		
1.0						97.0
2.0						96.0
3.0						95.0
4.0						

McROSTIE GENEST ST-LOUIS
Ottawa, Canada

LOGGED BY: JML
REVIEWED BY: E.S.
Fig. No: 3

COMPLETION DEPTH: 0.6 m
COMPLETE: 05/09/02
Page 1 of 1

05/05/20 08:07AM (STD-SHIP)

NRC – NEW ELECTRICAL SUB-STATION	B.M.(ELEV 100.20m) geodetic; Floor of building M-10 at door No.11	TEST PIT NO: 05-3
		PROJECT NO: E-8890
START DATE: 05/09/02		ELEVATION: 99.43 m

SAMPLE TYPE ■ REMOULDED ◿ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▯ NO RECOVERY ▮ CORE

DEPTH(m)	SMALL PEN. (kPa) SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(m)
0.0	sides stable			TOPSOIL		
				99.23		
				medium dense sandy TILL		99.0
	no water seepage			Bottom of test pit on possible rock 98.63		
1.0						
						98.0
2.0						
						97.0
3.0						
						96.0
4.0						

McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 0.8 m
	REVIEWED BY: E.S.	COMPLETE: 05/09/02
	Fig. No: 4	Page 1 of 1

05/09/20 08:02AM (STD-SHTP)

NRC – NEW ELECTRICAL SUB-STATION	B.M.(ELEV 100.20m)geodetic: Floor of building M-10 at door No. 11	TEST PIT NO: 05-4
		PROJECT NO: E-8890
START DATE: 05/09/02		ELEVATION: 100.34 m

SAMPLE TYPE ■ REMOULDED ◸ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▥ NO RECOVERY ▯ CORE

DEPTH(m)	SMALL PEN. (kPa) SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(m)
0.0	sides stable			TOPSOIL		
				100.14		
						100.0
				medium dense sandy TILL		
1.0						
						99.0
	no water seepage			Bottom of test pit on possible rock 98.94		
2.0						
						98.0
3.0						
						97.0
4.0						

McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 1.4 m
	REVIEWED BY: E.S.	COMPLETE: 05/09/02
	Fig. No: 5	Page 1 of 1

05/09/20 08:02AM (STD-SHIP)

NRC – NEW ELECTRICAL SUB-STATION	B.M.(ELEV 100.20m)geodetic; Floor of building M-10 at door No.11	TEST PIT NO: 05-5
		PROJECT NO: E-8890
START DATE: 05/09/02		ELEVATION: 100.41 m

SAMPLE TYPE ■ REMOULDED ◸ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▯ NO RECOVERY ▯ CORE

DEPTH(m)	SMALL PEN. (kPa) SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(m)
0.0	sides stable			TOPSOIL		
				100.16		
						100.0
				medium dense sandy TILL		
1.0						
	no water seepage			Bottom of test pit on possible rock 99.06		99.0
2.0						
						98.0
3.0						
						97.0
4.0						

McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML REVIEWED BY: E.S. Fig. No: 6	COMPLETION DEPTH: 1.35 m COMPLETE: 05/09/02 Page 1 of 1

05/09/20 08:02AM (STD-SHTP)

NRC – NEW ELECTRICAL SUB-STATION	B.M.(ELEV 100.20m) geodetic: Floor of building M-10 at door No.11	TEST PIT NO: 05-6
		PROJECT NO: E-8890
START DATE: 05/09/02		ELEVATION: 100.31 m

SAMPLE TYPE ■ REMOULDED ◿ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING [ ] NO RECOVERY [▌] CORE

DEPTH(m)	SMALL PEN (kPa) SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(m)
0.0	sides stable			TOPSOIL		
				100.01		100.0
1.0				medium dense sandy TILL		
						99.0
	no water seepage			98.51 Bottom of test pit on possible rock		
2.0						
						98.0
3.0						
						97.0
4.0						

McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 1.8 m
	REVIEWED BY: E.S.	COMPLETE: 05/09/02
	Fig. No: 7	Page 1 of 1

05/09/20 08:02AM (STD-SHTP)

NRC – NEW ELECTRICAL SUB-STATION

B.M.(ELEV 100.20m)geodetic; Floor of building M-10 at door No.11

TEST PIT NO: 05-7

PROJECT NO: E-8890

START DATE: 05/09/02

ELEVATION: 100.37 m

SAMPLE TYPE ■ REMOULDED ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▯ NO RECOVERY ▮ CORE

DEPTH(m)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(m)
0.0	sides stable				TOPSOIL		
					100.07		100.0
					medium dense sandy TILL		
1.0							
							99.0
	no water seepage				Bottom of test pit on possible rock 98.87		
2.0							
							98.0
3.0							
							97.0
4.0							

McROSTIE GENEST ST-LOUIS
Ottawa, Canada

LOGGED BY: JML
REVIEWED BY: E.S.
Fig. No: 8

COMPLETION DEPTH: 1.5 m
COMPLETE: 05/09/02
Page 1 of 1

05/09/20 08.03AM (STD-SHTP)

MONTREAL RD. NRC M-10 & COOLING TOWER	B.M.(ELEV 328.75FT.)geodetic: Floor of	TEST PIT NO: 02-1
NATIONAL RESEARCH COUNCIL CANADA	bldg. M-10 at door No. 11	PROJECT NO: E-8230
START DATE: 02/04/22		ELEVATION: 327.61 ft

SAMPLE TYPE ■ REMOULDED ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▯ NO RECOVERY ▮ CORE

DEPTH(ft)	SMALL PEN. (kPa) SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(ft)
0.0	sides stable			TOPSOIL		
1.0				327.19 FILL pieces of broken rock in sand & gravel		327.0
2.0				325.86		326.0
3.0				clayey SAND		325.0
4.0						324.0
5.0				medium dense sandy TILL 323.03		323.0
6.0	no water seepage			Bottom of pit on probable rock 322.11		322.0
7.0						321.0
8.0						320.0
9.0						319.0
10.0						318.0
11.0						317.0
12.0						316.0

McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 5.5 ft
	REVIEWED BY: E.S.	COMPLETE: 02/04/22
	Fig. No: 2	Page 1 of 1

02/04/30 11:12AM (ST-SHMP)

MONTREAL RD. NRC M-10 & COOLING TOWER	B.M.(ELEV 328.75FT.)geodetic: Floor of	TEST PIT NO: 02-2
NATIONAL RESEARCH COUNCIL CANADA	bldg. M-10 at door No. 11	PROJECT NO: E-8230
START DATE: 02/04/22		ELEVATION: 327.16 ft

SAMPLE TYPE ■ REMOULDED ▨ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING][NO RECOVERY ▮ CORE

DEPTH(ft)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(ft)
0.0	sides stable				TOPSOIL		327.0
					326.83		
1.0							326.0
2.0					FILL large pieces of broken rock up to 2.5'x2.5'x 1.0' in sand and gravel with pieces of tin and steel rebar		325.0
3.0							324.0
4.0	no water seepage				Bottom of pit on probable rock 323.16		323.0
5.0							322.0
6.0							321.0
7.0							320.0
8.0							319.0
9.0							318.0
10.0							317.0
11.0							316.0
12.0							315.0

McROSTIE GENEST ST-LOUIS	LOGGED BY: JML	COMPLETION DEPTH: 4 ft
Ottawa, Canada	REVIEWED BY: E.S.	COMPLETE: 02/04/22
	Fig. No: 3	Page 1 of 1

02/04/30 09:53AM (ST-SHMP)

MONTREAL RD. NRC M-10 & COOLING TOWER	B.M.(ELEV 328.75FT.)geodetic: Floor of	BOREHOLE NO: 02-3
NATIONAL RESEARCH COUNCIL CANADA	bldg. M-10 at door No. 11	PROJECT NO: E-8230
START DATE: 02/04/26		ELEVATION: 328.76 ft

SAMPLE TYPE ■ REMOULDED-AUGER ◪ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE

DEPTH(ft)	SMALL PEN. (kPa) / SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(ft)
0.0					FILL topsoil, sand and gravel		
1.7	3/6" 12/2" split barrel refusal	☒	1		FILL topsoil, sand, gravel and wood		327.26
2.7		▮		85	LIMESTONE		326.09
5.4		▮		83	LIMESTONE		323.34
8.4	▼ WL				Water level April 29/02 elev 320.34'		▼
10.4		▮		98	LIMESTONE		318.34
15.4		▮		100	LIMESTONE		313.34
18.4					Bottom of hole		310.34

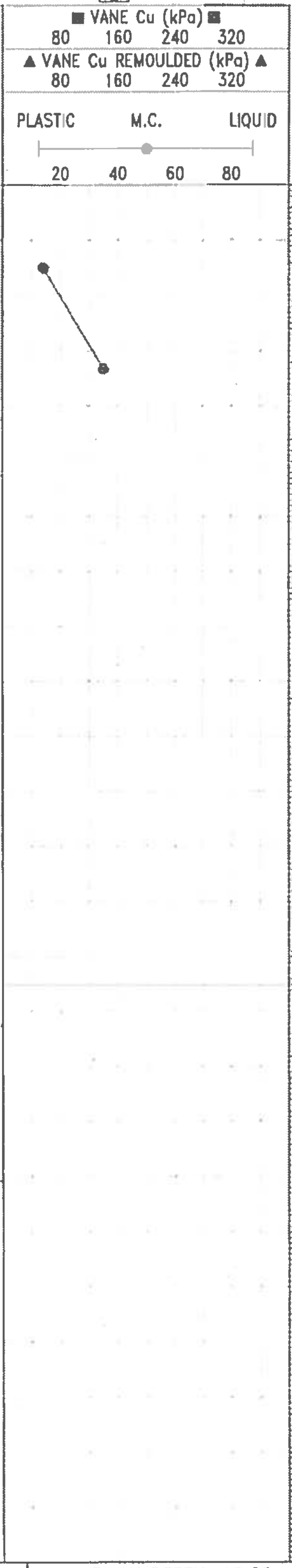
McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 18.42 ft
	REVIEWED BY: E.S.	COMPLETE: 02/04/26
	Fig. No: 4	Page 1 of 1

02/05/01 02:02PM (NQ-IMP)

MONTREAL RD. NRC M-10 & COOLING TOWER	B.M.(ELEV 328.75FT.)geodetic: Floor of	BOREHOLE NO: 02-4
NATIONAL RESEARCH COUNCIL CANADA	bldg. M-10 at door No. 11	PROJECT NO: E-8230
START DATE: 02/04/26		ELEVATION: 328.41 ft

SAMPLE TYPE ■ REMOULDED-AUGER ▱ SHELBY TUBE ☒ SPLIT-SPOON ▤ NW-CASING ▯ NO RECOVERY ▮ NQ CORE

DEPTH(ft)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	% CORE RECOVERY	SOIL / ROCK DESCRIPTION	ELEVATION(ft)
0.0						TOPSOIL topsoil, sand and gravel	328.0
1.0	split barrel refusal	6/6" 20/6"	☒	1		FILL 327.41 topsoil, sand, gravel	327.0
2.2						sandy TILL 326.41	326.0
4.2			▮		100	LIMESTONE 324.16	324.0
5.2	WL (7.4)		▮		80	323.16 LIMESTONE Water level April 29/02 elev 320.99'	323.0
10.2			▮		100	318.16 LIMESTONE	318.0
15.2			▮		100	313.16 LIMESTONE	313.0
18.08						Bottom of hole 310.33	310.0



DEPTH(ft): 0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0 16.0 17.0 18.0 19.0 20.0 21.0 22.0 23.0 24.0 25.0

ELEVATION(ft): 328.0 327.0 326.0 325.0 324.0 323.0 322.0 321.0 320.0 319.0 318.0 317.0 316.0 315.0 314.0 313.0 312.0 311.0 310.0 309.0 308.0 307.0 306.0 305.0 304.0

McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 18.08 ft
	REVIEWED BY: E.S.	COMPLETE: 02/04/26
	Fig. No: 5	Page 1 of 1

02/05/01 01:43PM (NO-IMP)

MONTREAL RD. NRC M-10 & COOLING TOWER	B.M.(ELEV 328.75FT.)geodetic: Floor of	TEST PIT NO: 02-5
NATIONAL RESEARCH COUNCIL CANADA	bldg. M-10 at door No. 11	PROJECT NO: E-8230
START DATE: 02/04/22		ELEVATION: 328.93 ft

SAMPLE TYPE ■ REMOULDED ◸ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▯▯ NO RECOVERY ▮ CORE

DEPTH(ft)	SMALL PEN. (kPa) SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(ft)
0.0	sides stable			FILL large pieces of broken rock up to (2.5'x2.5'x1.0') in sand and gravel		
1.0						328.0
2.0						327.0
				medium dense sandy TILL 326.43		
3.0	no water seepage					326.0
				Bottom of pit on probable rock 325.76		
4.0						325.0
5.0						324.0
6.0						323.0
7.0						322.0
8.0						321.0
9.0						320.0
10.0						319.0
11.0						318.0
12.0						317.0

McROSTIE GENEST ST-LOUIS	LOGGED BY: JML	COMPLETION DEPTH: 3.25 ft
	REVIEWED BY: e.s	COMPLETE: 02/04/22
Ottawa, Canada	Fig. No: 6	Page 1 of 1

02/04/30 11:12AM (ST-SHMP)

MONTREAL RD. NRC M-10 & COOLING TOWER
NATIONAL RESEARCH COUNCIL CANADA
START DATE: 02/04/22

B.M.(ELEV 328.75FT.)geodetic: Floor of bldg. M-10 at door No. 11

TEST PIT NO: 02-6
PROJECT NO: E-8230
ELEVATION: 328.77 ft

SAMPLE TYPE ■ REMOULDED ╱ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING [ ] NO RECOVERY [█] CORE

DEPTH(ft)	SMALL PEN. (kPa)	SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(ft)
0.0	sides stable				TOPSOIL		
					328.19 FILL rock blocks in sand and gravel		
							328.0
1.0							
							327.0
2.0					326.77 medium dense sandy TILL		
							326.0
3.0	no water seepage				Bottom of pit on probable rock 325.77		
							325.0
4.0							
							324.0
5.0							
							323.0
6.0							
							322.0
7.0							
							321.0
8.0							
							320.0
9.0							
							319.0
10.0							
							318.0
11.0							
							317.0
12.0							

McROSTIE GENEST ST-LOUIS
Ottawa, Canada

LOGGED BY: JML
REVIEWED BY: E.S.
Fig. No: 7

COMPLETION DEPTH: 3 ft
COMPLETE: 02/04/22
Page 1 of 1

02/04/30 09:53AM (ST-STAMP)

MONTREAL RD. NRC M-10 & COOLING TOWER	B.M.(ELEV 328.75FT.)geodetic: Floor of	TEST PIT NO: 02-7
NATIONAL RESEARCH COUNCIL CANADA	bldg. M-10 at door No. 11	PROJECT NO: E-8230
START DATE: 02/04/22		ELEVATION: 328.36 ft

SAMPLE TYPE ■ REMOULDED ▨ SHELBY TUBE ☒ SPLIT-SPOON ☰ PROBING [ ] NO RECOVERY [▮] CORE

DEPTH(ft)	SMALL PEN. (kPa) SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(ft)
0.0	sides stable			FILL topsoil with a trace of brick		328.0
1.0				327.69		327.0
2.0				FILL large pieces of broken rock up to (2.5'x2.5'x1.0') in sand and gravel with traces of concrete and metal		326.0
3.0						325.0
4.0						324.0
5.0						323.0
6.0	no water seepage			Bottom of pit on probable rock 322.36		322.0
7.0						321.0
8.0						320.0
9.0						319.0
10.0						318.0
11.0						317.0
12.0						316.0

McROSTIE GENEST ST-LOUIS	LOGGED BY: JML	COMPLETION DEPTH: 6 ft
Ottawa, Canada	REVIEWED BY: E.S.	COMPLETE: 02/04/22
	Fig. No: 8	Page 1 of 1

02/04/30 09:53AM (ST-SHMP)

MONTREAL RD. NRC M-10 & COOLING TOWER	B.M.(ELEV 328.75FT.)geodetic: Floor of	TEST PIT NO: 02-8
NATIONAL RESEARCH COUNCIL CANADA	bldg. M-10 at door No. 11	PROJECT NO: E-8230
START DATE: 02/04/22		ELEVATION: 328.08 ft

SAMPLE TYPE ■ REMOULDED ⧄ SHELBY TUBE ☒ SPLIT-SPOON ▤ PROBING ▯ NO RECOVERY ▮ CORE

DEPTH(ft)	SMALL PEN. (kPa) SPT (N)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION	■ VANE Cu (kPa) ■ 80 160 240 320 ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 PLASTIC M.C. LIQUID 20 40 60 80	ELEVATION(ft)
0.0	sides stable			TOPSOIL		328.0
1.0				327.41 FILL large pieces of broken rock up to (2.5'x2.5'x1.0') in sandy soil and traces of brick		327.0
2.0						326.0
3.0				medium dense sandy TILL 325.08		325.0
4.0	no water seepage			Bottom of pit on probable rock 324.08		324.0
5.0						323.0
6.0						322.0
7.0						321.0
8.0						320.0
9.0						319.0
10.0						318.0
11.0						317.0
12.0						316.0

McROSTIE GENEST ST-LOUIS Ottawa, Canada	LOGGED BY: JML	COMPLETION DEPTH: 4 ft
	REVIEWED BY: E.S.	COMPLETE: 02/04/22
	Fig. No: 9	Page 1 of 1

02/04/30 09:54AM (ST-SHMP)

McROSTIE GENEST ST-LOUIS & Associates Ltd. Consulting Engineers OTTAWA, CANADA	TEST PIT RECORD	Test Pit No. 99-6
	Date :	JUNE 11, 1999

N.R.C. BLDG. M-10 ADDITION
MONTREAL ROAD

ELEV. 329.23	DEPTH in feet	DESCRIPTION	REMARKS
		TOPSOIL	sides stable
328.41	0.82		
328.23	-- 1 --		
		BOULDERS up to 1.6' O in dense sandy TILL	
327.23	-- 2 --		
326.23	-- 3 --		
325.62	3.61	Bottom of pit on probable rock	no water seepage

Plate No.
8

McROSTIE GENEST ST-LOUIS & Associates Ltd. Consulting Engineers OTTAWA, CANADA	TEST PIT RECORD	Test Pit No. 99-5
	Date :	JUNE 11, 1999

N.R.C. BLDG. M-10 ADDITION
MONTREAL ROAD

ELEV. 328.54	DEPTH in feet	DESCRIPTION	REMARKS
		FILL crushed limestone	sides stable
327.54	-- 1 --		
327.23	1.31		
326.54	-- 2 --	dense sandy TILL	
325.54	-- 3 --		
325.26	3.28	Bottom of pit on probable rock	no water seepage

Plate No.
7