REPORT

Geotechnical Report

Proposed New Church, 35 Highbury Park Drive, Ottawa, Ontario

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Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for a proposed church to be located at 35 Highbury Park Drive in Ottawa, Ontario.

The purpose of this subsurface investigation was to determine the general soil, bedrock, and groundwater conditions across the site by means of 14 test pits and one borehole and, based on an interpretation of the factual information obtained, along with the existing subsurface information available for the site, to provide engineering guidelines on the geotechnical design aspects of the proposed church, including construction considerations which could influence design decisions.

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

2.0 DESCRIPTION OF PROJECT AND SITE

Plans are being prepared for a proposed Sequoia Community Church to be located at 35 Highbury Park Drive in Ottawa, Ontario (for location see Key Plan inset, Figure 1).

The following information is known about the site:

- The site is located along the north side of Highbury Park Drive and is bounded to the north by vacant land and the CN rail line, to the west by Greenbank Road and to the east by the Transitway and residential developments.
- The site is roughly rectangular in shape and measures about 125 metres by 90 metres in plan area.
- The site is presently undeveloped and contains variable amounts of tree and brush coverage. In the past, the site was used as a snow dump for the City of Ottawa. The topography of the site generally slopes down from the south to north.
- It is understood that the new church will be two storeys in height with one basement level. The church will be located at the south end of the property with at grade parking to the north.

Based on a review of the published geological mapping, the subsurface conditions at this site are expected to consist of about 0 to 2 metres of sand, silty clay and glacial till overlying bedrock. The depth to the bedrock surface generally increases from the south to the north. The bedrock geology mapping indicates the bedrock on this site to consist of sandstone and dolomite of the March Formation.

3.0 PROCEDURE

The field work for this investigation was carried out on November 24 and 25, 2011. On November 24, 2011, 14 test pits (numbered TP 11-1 to TP 11-7 and TP 11-9 to TP 11-15, inclusive) were put down; and on November 25, 2011, one borehole (numbered BH 11-8) was advanced at the approximate locations shown on Figure 1.

The test pits were excavated using a track-mounted hydraulic excavator supplied and operated by Glen Wright Excavating of Ottawa, Ontario. The test pits were all advanced to the bedrock surface, at depths ranging from approximately 0.2 to 2.4 metres below the existing ground surface.

The borehole was advanced using a truck mounted drill rig, supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The borehole was advanced into bedrock to a depth of 9.3 metres below the ground surface; the drilling was carried out using rotary diamond drilling techniques and retrieved NQ sized bedrock core.

A monitoring well was installed in borehole 11-8 for the subsequent measurement of the groundwater level at the site. The groundwater level was measured on November 30, 2011.

The soil 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 and the test pits were loosely backfilled upon completion of excavating and sampling.

The field work was supervised by a member of our technical staff who located the test pits and borehole, directed the excavating and drilling operations, logged the test pits, borehole and samples, and took custody of the samples retrieved.

Upon completion of the field work, samples of the soils and bedrock core encountered in the test pits and borehole were transported to our laboratory for examination by the project engineer.

One sample of soil from test pit 11-10 was submitted to Exova Laboratories Ltd. for chemical analysis related to potential corrosion of buried steel elements and potential sulphate attack on buried concrete elements.

The test pit and borehole locations were selected and subsequently surveyed by Golder Associates using a Trimble R8 GPS unit. The ground surface elevations are referenced to Geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the borehole advanced for this investigation are shown on the Record of Drillhole in Appendix A. The subsurface conditions encountered in the test pits excavated for this investigation are shown on the Record of Test Pits in Table 1. The results of the basic chemical analysis carried out on one sample of soil from test pit 11-10 are provided in Appendix B.

The subsurface conditions on this site generally consist of discontinuous layers of sand and gravel fill, silty sand, sand and gravel, and sandy silt, underlain by a discontinuous layer of glacial till over sandstone bedrock.
The bedrock surface typically exists at depths ranging from about 0.2 to 2.4 metres below ground surface.

The following section presents an overview of the subsurface conditions encountered in test pits and the borehole.

4.2 Fill and Topsoil

A layer of fill exists at the ground surface at test pits 11-5, 11-6, 11-7 and 11-9. The fill material generally consists of sand to sand and gravel and organic silty sand with traces of asphaltic concrete, plastic, metals, and organics. Test pits 11-5, 11-6, and 11-7 are located within the access road to the previously existing snow dump and test pit 11-9 is located within the snow dump. The fill ranges in thickness from about 200 to 600 millimetres.

A layer of topsoil exists at ground surface at test pits 11-1, 11-3, 11-4, and 11-10 to 11-15 and below the fill at test pits 11-7 and 11-9. The topsoil thickness ranges from approximately 60 to 200 millimetres.

4.3 Silty Sand and Silty Sand and Gravel

A thin deposit of silty sand or silty sand and gravel was encountered below the topsoil in test pits 11-1, 11-3, 11-4, and 11-10 and below the fill in test pit 11-5.

The deposit of silty sand extends down to depths ranging from about 0.3 to 0.6 metres below the existing ground surface and ranges in thickness from 170 to 420 millimetres. The silty sand deposit was noted to have some gravel and cobbles and trace to some organics.

A deposit of sand and gravel was encountered at the ground surface in test pit 11-2 and below the topsoil in test pits 11-6 and 11-9. The deposit of sand and gravel extends down to depths ranging from about 0.6 to 0.8 metres below the existing ground surface and ranges in thickness from about 250 to 600 millimetres. The sand and gravel deposit was noted to have some silt, cobbles, boulders and rock slabs, and trace organics.

4.4 Silty Clay to Clayey Silt and Sandy Silt

The topsoil is underlain by a deposit of silty clay to clayey silt in test pits 11-14 and 11-15. The deposit of silty clay to clayey silt extends to depths of about 0.5 and 0.4 metres below the existing ground surface and has a thickness of about 300 and 230 millimetres in test pits 11-14 and 11-15, respectively.

A deposit of sandy silt was encountered below the topsoil in test pits 11-12 and 11-13 and below the silty clay to clayey silt in test pits 11-14 and 11-15. The deposit of sandy silt extends to depths ranging from about 0.6 to 1.4 metres below the existing ground surface and ranges in thickness from 130 millimetres to 1.0 metres.

4.5 Glacial Till

A discontinuous deposit of glacial till underlies the fill, topsoil, clayey soils, sandy silt, and sand and gravel deposits in test pits 11-1, 11-3, 11-4, and 11-12 to 11-15. In general, the glacial till is a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand. The glacial till was fully penetrated in the test pits, where encountered, and extends down to depths ranging from about 0.6 to 2.4 metres below the existing ground surface and ranges in thickness from about 400 millimetres to 1.7 metres.

4.6 Sandstone Bedrock and Refusal

Sandstone bedrock was encountered beneath the overburden soils in the test pits. The bedrock surface exists at depths ranging from about 0.2 to 2.4 metres below the existing ground surface at the test pit locations and was encountered at ground surface at borehole 11-8.

In test pits 11-7 and 11-11, the upper portion of the bedrock is weathered and the excavation could be advanced into the bedrock by up to an additional 190 and 120 millimetres, respectively, before encountering practical refusal to excavating.

Test Pit / Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
TP 11-1	98.3	0.6	97.7
TP 11-2	99.1	0.8	98.3
TP 11-3	97.6	0.6	97.0
TP 11-4	98.4	0.6	97.8
TP 11-5	97.2	0.4	96.8

Test Pit / Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
TP 11-6	97.8	0.6	97.2
TP 11-7	98.5	0.7	97.8
BH 11-8	96.0	0.0	96.0
TP 11-9	96.1	0.7	95.4
TP 11-10	96.7	0.6	96.1
TP 11-11	97.0	0.1	96.9
TP 11-12	95.6	1.1	94.5
TP 11-13	95.3	1.2	94.1
TP 11-14	94.9	2.4	92.6
TP 11-15	94.9	1.8	93.1

4.7 Groundwater Conditions

No groundwater seepage was observed in the test pits during the short time that they remained open (i.e., the test pits were dry upon completion of excavating).

A monitoring well was installed in borehole 11-8 for the subsequent measurement of the groundwater level at the site. The groundwater level was measured on November 30, 2011 and is summarized in the table below.

Borehole Number	Screen Depth (m)	Ground Surface Elevation (m)	Depth to Groundwater Level (m)	Groundwater Level Elevation (m)	Date of Reading
11-8	4.6 – 9.3	96.0	5.0	91.0	November 30, 2011

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 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 borehole and test pit information and project requirements and is subject to the limitations in the “Important Information and Limitations of This Report’ which follows the text but forms an integral part of this report.

5.2 Grade Raise Restrictions

In general, the subsurface conditions on this site generally consist of silty sand, sand and gravel and sandy silt, underlain by glacial till and sandstone bedrock. Within the area of the proposed church, the depth to sandstone bedrock typically ranges between 0.4 to 0.6 metres below the existing ground surface. The groundwater level was measured to be at about 5 metres below the existing ground surface.

From a foundation design perspective, no restrictions apply to the thickness of grade raise fill that may be placed on the site of the proposed church.

For predictable performance of the structures, hard or paved surfaces, and site services, preparation for filling the site should include stripping the existing fill. The fill is not suitable as general fill and should be stockpiled separately for re-use in landscaping applications only.

5.3 Excavations

Current plans indicate that the proposed church will have a basement level. It is expected that the excavation will extend about 3 to 4 metres below the existing ground surface to accommodate the footing construction and elevator pit, if required. Excavations for the construction of the foundations and site servicing will be through the overburden and into the underlying sandstone bedrock.

5.3.1 Overburden

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 fill. 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 for the overburden soils, based on the measured groundwater level in borehole 11-8), 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. If the water table is encountered within the overburden materials, the soils would generally be classified as Type 4 soils, and excavation side slopes must be sloped at a minimum of 3 horizontal to 1 vertical or be shored.

5.3.2 Bedrock

The bedrock surface was encountered at depths of about 0.6 metres below the existing ground surface within the proposed building footprint. Bedrock removal will therefore likely be required for construction of the basement level and the foundations.

Where shallow excavation of the sound bedrock is required (e.g., for sewers or footings), 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. Where larger volumes of bedrock removal are required, blasting may be more economical and could also be considered as a means of bedrock removal.

Near vertical trench walls in the bedrock should stand unsupported for the construction period.

Blasting (if required) should be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be avoided. This will require blast designs by a specialist in this field.

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

The contractor should be required to submit a complete and detailed blasting design and monitoring proposal prepared by a blasting/vibrations specialist prior to commencing blasting. This would have to be reviewed and accepted in relation to the requirements of the blasting specifications.

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

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

It is recommended that the monitoring of ground vibration intensities (peak ground vibrations and accelerations) from the blasting operations be carried out both in the ground adjacent to the closest structures and within the structures themselves.

5.4 Foundations

Based on the drawings provided, the proposed church will be provided with one basement level. It is assumed that the footings will be at a depth of up to 3 to 4 metres below the existing ground surface. Therefore, it is considered that the church can be supported on conventional spread footings placed on or within the bedrock.

The factored bearing resistance at Ultimate Limit States (ULS) for spread footing foundations founded on or within the bedrock may be taken as 1 megapascal. Provided the bedrock surface is acceptably cleaned of soil or loose bedrock (i.e., any bedrock that can easily be removed with a hydraulic excavator), the settlement of footings at the corresponding service (unfactored) load levels will be less than 25 millimetres and therefore Serviceability Limit States (SLS) need not be considered in the foundation design. Accordingly, the post construction settlement of structural elements which derive their support from footings bearing on bedrock should be negligible.

5.5 Temporary Dewatering

All test pits were dry upon completion of excavation. The stabilized groundwater level measured in borehole 11-8 is at a depth of 5 metres below ground surface (elevation 91.0 metres). As such, groundwater inflow into the excavations at the site that are above the stabilized groundwater level should be minor, and should be handled by pumping from sumps within the excavation.

Under the new regulations, a Permit-To-Take-Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water greater than 400,000 litres per day is pumped from the excavation. If the volume of water to be pumped will be less than 400,000 litres per day, but more than 50,000 litres per day, the water taking will not require a PTTW, but will need to be registered in the Environmental Activity and Sector Registry (EASR) as a prescribed activity. Based on the groundwater information collected during the investigation, it is considered unlikely that an EASR or PTTW would be required during construction for this project. However, the requirement for registration of an EASR is possible if inflows are greater than expected. The requirement for registration (i.e., if more than 50,000 litres per day is being pumped) can be assessed at the time of construction. Registration is a quick process that will not significantly disrupt the construction schedule.

5.6 Seismic Site Class

The seismic design provisions of the 2012 Ontario Building Code depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock below founding level. However, the OBC also permits the Site Class to be specified based solely on the stratigraphy and in situ testing data, rather than from direct measurements of the shear wave velocity. Using that methodology, a Site Class of C can be used for design of the proposed church.

5.7 Frost Protection

All exterior foundation elements or 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 adjacent to unheated areas which are 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.

It is expected that these requirements will be satisfied for all of the structure footings due to the deep founding levels required to accommodate the basement level.

5.8 Basement Floor Slab

In preparation for construction of the basement floor slab, all loose, wet, and disturbed material should be removed from beneath the floor slab. Provision should be made for a drainage layer consisting of at least 300 millimetres of free draining granular material, such as 16 millimetre clear crushed stone, to underlie the floor slab. To prevent hydrostatic pressure build up, this granular layer should be drained. This should be achieved by installing rigid 100 millimetre diameter perforated pipes in the floor slab bedding at 6 metre centres. The perforated pipes should discharge to a positive outlet such as a sump from which the water is pumped.

Any bulk fill required to raise the grade to the underside of the clear stone 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 and Lateral Earth Pressures

5.9.1 Open Cut Excavations

The soils at this site are frost susceptible and should not be used as backfill against exterior, unheated, or well insulated foundation elements within the depth of potential frost penetration (1.5 metres) to avoid problems with frost adhesion and heaving. Free draining backfill materials are also required if hydrostatic water pressure against the basement walls (and potential leakage) is to be avoided. The foundation and basement walls therefore should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I.

To avoid ground settlements around the foundations, which could affect site grading and drainage, all of the backfill materials should be placed in 0.3 metre thick lifts and compacted to at least 95 percent of the material’s standard Proctor maximum dry density.

The basement wall backfill should be drained by means of a perforated pipe subdrain in a surround of 19 millimetres clear stone, fully wrapped in a geotextile, which leads by positive drainage to a storm sewer or to a sump from which the water is pumped.

5.9.2 Excavations in Bedrock

Where basement walls will be poured against bedrock, vertical drainage such as Miradrain must be installed on the face of the bedrock to provide the necessary drainage. The top edge of the Miradrain should be sealed or covered with a geotextile to prevent the loss of soil into the void between the sheet and geotextile of the Miradrain.

Low shrinkage concrete and/or a bond break, consisting of polyethylene sheeting placed between the concrete foundation wall and the Miradrain/bedrock, should be provided to reduce the potential for shrinkage cracking of the foundation walls. If the Miradrain is continuous along the bedrock excavation walls (i.e. completely covers the rock along the full foundation length), rather than in discrete strips with exposed rock between the Miradrain strips, the bond break is not required.

Where the basement walls will be constructed using formwork, it will be necessary to backfill a narrow gallery between the shoring or bedrock face and the outside of the walls. The backfill should consist of 6 millimetre clear stone ‘chip’, placed by a stone slinger or chute.

In no case should the clear stone chip be placed in direct contact with other soils. For example, surface landscaping or backfill soils placed near the top of the clear stone back fill should be separated from the clear stone with a geotextile.

Both the drain pipe for the wall backfill and/or the Miradrain should be connected to a perimeter drain at the base of the excavation which is connected to a sump pump.

5.9.3 Lateral Earth Pressures

It is considered that three design conditions exist with regards to the lateral earth pressures that will be exerted on the basements walls:

1) Walls cast directly against the bedrock face.
2) Walls cast against formwork with a narrow backfilled gallery provided between the basement wall and the adjacent excavation bedrock face.
3) Walls cast against formwork with a wide backfilled gallery provided between the basement wall and the adjacent excavation face.

For the first case (wall cast against the bedrock), there will be no effective lateral earth pressures on the basement wall.

For the second case, the magnitude of the lateral earth pressure depends on the magnitude of the arching which can develop in the backfill and therefore depends on the width of the backfill, its angle of internal friction, as well as the interface friction angles between the backfill and both the rock face and the basement wall. The magnitude of the lateral earth pressure can be calculated as:

$$\sigma_h(z) = \frac{\gamma B}{2\tan\delta}\left(1 - e^{-2K\frac{z}{B}\tan\delta}\right) + \mathrm{K\ q}$$

Where:

$\sigma_h(z)$ = Lateral earth pressure on the basement wall at depth z, kilopascals.

K = Earth pressure coefficient, use 0.6.

γ = Unit weight of retained soil, use 20 kilonewtons per cubic metre for clear stone chip.

B = Width of backfill (between basement wall and bedrock face), metres.

δ = Average interface friction angle at backfill-basement wall and backfill-rock face interfaces, use 15 degrees.

z = Depth below top of formwork, metres.

q = Surcharge at ground surface to account for traffic, equipment, or stock piled materials (use 15 kilopascals).

For the third case, the basement walls should be designed to resist lateral earth pressures calculated as:

$$\sigma_h(z) = K_o\ (\gamma z + q)$$

Where:

$\sigma_h(z)$ = Lateral earth pressure on the wall at depth z, kilopascals.

K_o = At-rest earth pressure coefficient, use 0.5.

γ = Unit weight of retained soil, use 22 kilonewtons per cubic metre.

z = Depth below top of wall, metres.

q = Uniform surcharge at ground surface behind the wall to account for traffic, equipment, or stockpiled soil (use 15 kilopascals).

Hydrostatic groundwater pressures would also need to be considered if the structure is designed to be water-tight.

Conventional damp proofing of the basement walls is appropriate with the above design approach. For concrete walls poured against shoring or bedrock, damp proofing using an interior treatment such as Crystal Lok is suggested.

These lateral earth pressures would increase under seismic loading conditions. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The combined pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(z) = K_o\ \gamma\ z + (K_{AE} – K_o)\ \gamma\ (H-z)$$

Where:

K_{AE} = The seismic earth pressure coefficient, use 0.7; and,

H = The total depth to the bottom of the foundation wall (m).

All of the lateral earth pressure equations are given in an unfactored format and will need to be factored for Limit States Design purposes.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill immediately adjacent to the building and the more frost susceptible backfill placed beyond the wall backfill. To reduce the severity of 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 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.

5.9 Site Servicing

Excavations for the installation of site services will be through the overburden soils and into the sandstone bedrock.

No unusual problems are anticipated in excavating in the overburden using conventional hydraulic excavating equipment, recognizing that large boulders may be encountered. Boulders larger than 0.3 metres in size should be removed from the excavation side slopes.

Excavation side slopes above the water table should be stable in the short term at 1 horizontal to 1 vertical. Excavation side slopes below groundwater level in the overburden soils will slough to a somewhat flatter inclination. In accordance with the Occupational Health and Safety Act of Ontario, these excavation side slopes would likely need to be cut back at 3 horizontal to 1 vertical (i.e., Type 4 soils).

For shallow depths of excavation, it may be possible to remove the upper weathered portion of the bedrock, using large hydraulic excavating equipment. Further bedrock removal could be accomplished using mechanical methods (such as hoe ramming). Excavations deep into the rock will likely require drill and blast procedures. Near vertical trench walls in the bedrock should stand unsupported for the construction period, at least for moderate depths (i.e., less than about 3 metres).

Some groundwater inflow into the excavations should be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps in the excavations provided suitably sized pumps are used.

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where a bedrock subgrade is encountered at the bedding level (which is expected to be the case for this site), the Granular A bedding should be thickened to 300 millimetres. Where unavoidable disturbance to the subgrade surface occurs, 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. 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 or surrounding soil could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from 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 standard Proctor maximum dry density using suitable vibratory compaction equipment.

It is should be generally acceptable to re-use the excavated overburden soils as trench backfill. However, some of the overburden materials (such as the sandy silts) may be too wet to compact. Where that is the case, the wet materials should be wasted (and drier materials imported) or these materials should be placed only in the lower portions of the trench, recognizing that some future settlement of the roadways may occur.

Well fractured or well broken bedrock will be acceptable as backfill within the lower portion of the service trenches in areas where the excavation is in rock. The rock fill, however, should only be placed from at least 300 millimetres above the pipes to minimize damage due to impact or point loading. The rock fill should be limited to a maximum of 300 millimetres in size.

In areas where the trench will be covered with hard surfaced materials, the type of material placed within the frost zone (between finished grade and about 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 standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.10 Pavement Design

In preparation for pavement construction, all topsoil and deleterious material (i.e., those material containing organic material) should be removed from all pavement areas.

Those portions of the fill not containing organic matter may be left in place provided that some limited long term settlement of the pavement surface can be tolerated. However, the surface of the fill material at subgrade level should be proof rolled with a heavy smooth drum roller under the supervision of qualified geotechnical personnel to compact the existing fill and to identify soft areas requiring sub-excavation and replacement with more suitable fill. A layer of topsoil was encountered buried beneath the fill material in test pit 11-6, 11-7 and 11-9; therefore the topsoil may have not been removed prior to the access road and snow dump fill material being placed in this area. For predicable performance of the pavement structure, the topsoil should be removed from beneath the fill materials and replaced with more suitable fill.

Sections requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material. These materials 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. Well broken or re-crushed bedrock would be acceptable roadway fill material.

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.

The pavement structure for car parking areas should consist of:

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

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

Pavement Component	Thickness (millimetres)
Asphaltic Concrete	90
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 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 density 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 asphalt cement should consist of PG 58-34.

5.10 Corrosion and Cement Type

A sample of soil from test pit 11-10 was submitted to Exova Laboratories Ltd. for chemical analysis related to potential corrosion of exposed buried ferrous elements and potential sulphate attack on buried concrete elements. The results of this testing are summarized in the table below and are provided in Appendix B.

Test Pit Number / Sample Number	Sample Depth (m)	Chloride (%)	SO_4 (%)	pH	Resistivity (Ohm-cm)
TP 11-10 / Sa 1	0.3 – 0.6	<0.002	0.06	6.2	16,700

The results also indicate that Type GU cement should be acceptable for substructures. The results also indicate an elevated potential for corrosion of exposed ferrous metal, which should be considered in the design of substructures.

As previously mentioned, in the past, this site was used as a snow dump for the City of Ottawa. Therefore, there is a higher potential for chlorides from the road salt contained in the meltwater from the snow that was deposited on this site.

6.0 ADDITIONAL CONSIDERATIONS

The soils and the weathered sandstone bedrock on 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 soils having adequate bearing capacity have been reached and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill as well as sewer bedding and backfill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction view point.

The test pits were loosely backfilled upon completion of excavating and therefore constitute zones of disturbance. Should the proposed church layout change and the test pits be located within building areas, then those test pits will need to be repaired at the time of construction. All test pits unless located in landscaped areas should be re-excavated and the excavated soil placed back into the test pit in maximum 300 millimetre thick lifts compacted to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

At the time of the writing of this report, only conceptual details for the proposed church 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.

Signature Page

Golder Associates Ltd.

W. Alex Meacoe

Alex Meacoe, P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
W. A. MEACOE
100162115
JAN 31, 2019
PROVINCE OF ONTARIO

Bill Cavers, P.Eng.
Associate, Senior Geotechnical Engineer

WAM/WC/MVRD
https://golderassociates.sharepoint.com/sites/101845/deliverables/geotechnical report/18114424-001-r-rev0-sequoia church-jan 2019.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 practicing 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, **Possess the Land**. 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 – m)	Depth (m)	Description
11-1 (98.32)	0.00 – 0.12	Dark brown TOPSOIL
	0.12 – 0.37	Red brown SILTY SAND, some gravel, cobbles, trace organics
	0.37 – 0.60	Grey brown silty sand and gravel some cobbles and boulders (up to 0.35 m dia.) (GLACIAL TILL)
	0.60	SANDSTONE BEDROCK
		Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.12 – 0.37
2	0.37 – 0.60

Test Pit Number (Elevation – m)	Depth (m)	Description
11-2 (99.12)	0.00 – 0.16	Dark brown to black and red brown SAND and GRAVEL with organic matter and roots
	0.16 – 0.75	Red brown and grey brown SILTY SAND and GRAVEL with cobbles, boulders and rock slabs
	0.75	SANDSTONE BEDROCK
		Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.05 – 0.17
2	0.50 – 0.75

TABLE 1 (Continued)

RECORD OF TEST PITS

Test Pit Number (Elevation – m)	Depth (m)	Description
11-3	0.00 – 0.15	Dark brown TOPSOIL
(97.60)	0.15 – 0.32	Red brown SILTY SAND, some gravel and cobbles, trace organics
	0.32 – 0.56	Grey brown SILTY SAND and GRAVEL, with cobbles (up to 0.25 m dia.) (GLACIAL TILL)
	0.56	SANDSTONE BEDROCK

Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.15 – 0.32
2	0.40 – 0.56

Test Pit Number (Elevation – m)	Depth (m)	Description
11-4	0.00 – 0.12	Dark brown to black TOPSOIL
(98.38)	0.12 – 0.35	Red brown SILTY SAND, some gravel, trace to some organics
	0.35 – 0.60	Grey brown SILTY SAND and GRAVEL, with cobbles and boulders (up to 0.4 m dia.) (GLACIAL TILL)
	0.60	SANDSTONE BEDROCK

Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.35 – 0.60

TABLE 1 (Continued)
RECORD OF TEST PITS

Test Pit Number (Elevation – m)	Depth (m)	Description
11-5 (97.15)	0.00 – 0.20	Grey and grey brown SAND and GRAVEL, trace silt (FILL)
	0.20 – 0.40	Brown SILTY SAND with gravel and sandstone fragments of bedrock (top of weathered bedrock)
	0.40	SANDSTONE BEDROCK

Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0 – 0.3

Test Pit Number (Elevation – m)	Depth (m)	Description
11-6 (97.79)	0.00 – 0.30	Grey brown SAND and GRAVEL, some asphaltic concrete pieces and slabs up to 600 x 600 mm, trace plastic, metal, and organics (FILL)
	0.30 – 0.37	Dark brown TOPSOIL
	0.37 – 0.62	Red brown SAND and GRAVEL, some silt, trace organic matter and roots
	0.62	SANDSTONE BEDROCK

Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.10 – 0.30
2	0.45 – 0.62

TABLE 1 (Continued)

RECORD OF TEST PITS

Test Pit Number (Elevation – m)	Depth (m)	Description
11-7 (98.46)	0.00 – 0.18	Grey to dark grey SAND and GRAVEL with crushed stone (FILL)
	0.18 – 0.60	Brown, fine SAND, some clayey silt pockets, occasional asphaltic concrete pieces (FILL)
	0.60 – 0.66	Dark brown TOPSOIL
	0.66 – 0.85	Weathered SANDSTONE BEDROCK
	0.85	Slightly weathered to fresh SANDSTONE BEDROCK

Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.30 – 0.60

11-9 (96.12)	0 – 0.3	Brown and grey SAND and organic SILTY SAND, trace gravel, trace plastic (FILL)
	0.3 – 0.4	Dark brown silty sand, some gravel, trace organics (TOPSOIL)
	0.4 – 0.7	Brown and grey brown silty SAND and GRAVEL, trace organics
	0.7	SANDSTONE BEDROCK

Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.1 – 0.3
2	0.4 – 0.7

TABLE 1 (Continued)

RECORD OF TEST PITS

Test Pit Number (Elevation – m)	Depth (m)	Description
11-10	0.00 – 0.18	Dark brown TOPSOIL
(96.67)	0.18 – 0.60	Red brown to yellow brown SILTY SAND, some gravel and cobbles, trace organics
	0.60	SANDSTONE BEDROCK
		Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.30 – 0.60

Test Pit Number (Elevation – m)	Depth (m)	Description
11-11	0.00 – 0.12	Dark brown TOPSOIL
(97.00)	0.12 – 0.24	Rock fragments with silty sand layers (WEATHERED BEDROCK)
	0.24	SANDSTONE BEDROCK
		Note: Test pit dry upon completion.
		No samples taken

TABLE 1 (Continued)

RECORD OF TEST PITS

Test Pit Number (Elevation – m)	Depth (m)	Description
11-12	0.00 – 0.20	Dark brown TOPSOIL
(95.55)	0.20 – 0.60	Yellow brown SANDY SILT, some rootlets
	0.60 – 1.10	Compact to dense, grey brown SILTY SAND and GRAVEL, some cobbles (GLACIAL TILL)
	1.10	SANDSTONE BEDROCK

Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.30 – 0.50
2	0.90 – 1.10

Test Pit Number (Elevation – m)	Depth (m)	Description
11-13	0.00 – 0.20	Dark brown TOPSOIL
(95.29)	0.20 – 0.57	Yellow brown/green brown SANDY SILT
	0.57 – 1.20	Dense, grey brown SILTY SAND and GRAVEL, with cobbles, some boulders (GLACIAL TILL)
	1.20	SANDSTONE BEDROCK

Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.35 – 0.55
2	0.90 – 1.20

TABLE 1 (Continued)
RECORD OF TEST PITS

Test Pit Number (Elevation – m)	Depth (m)	Description
11-14	0.00 – 0.20	Dark brown TOPSOIL
(94.93)	0.20 – 0.50	Dark green brown SILTY CLAY to CLAYEY SILT
	0.50 – 0.63	Grey brown SANDY SILT
	0.63 – 2.37	Very dense green brown SILTY SAND and GRAVEL, some cobbles and boulders (GLACIAL TILL)
	2.37	SANDSTONE BEDROCK

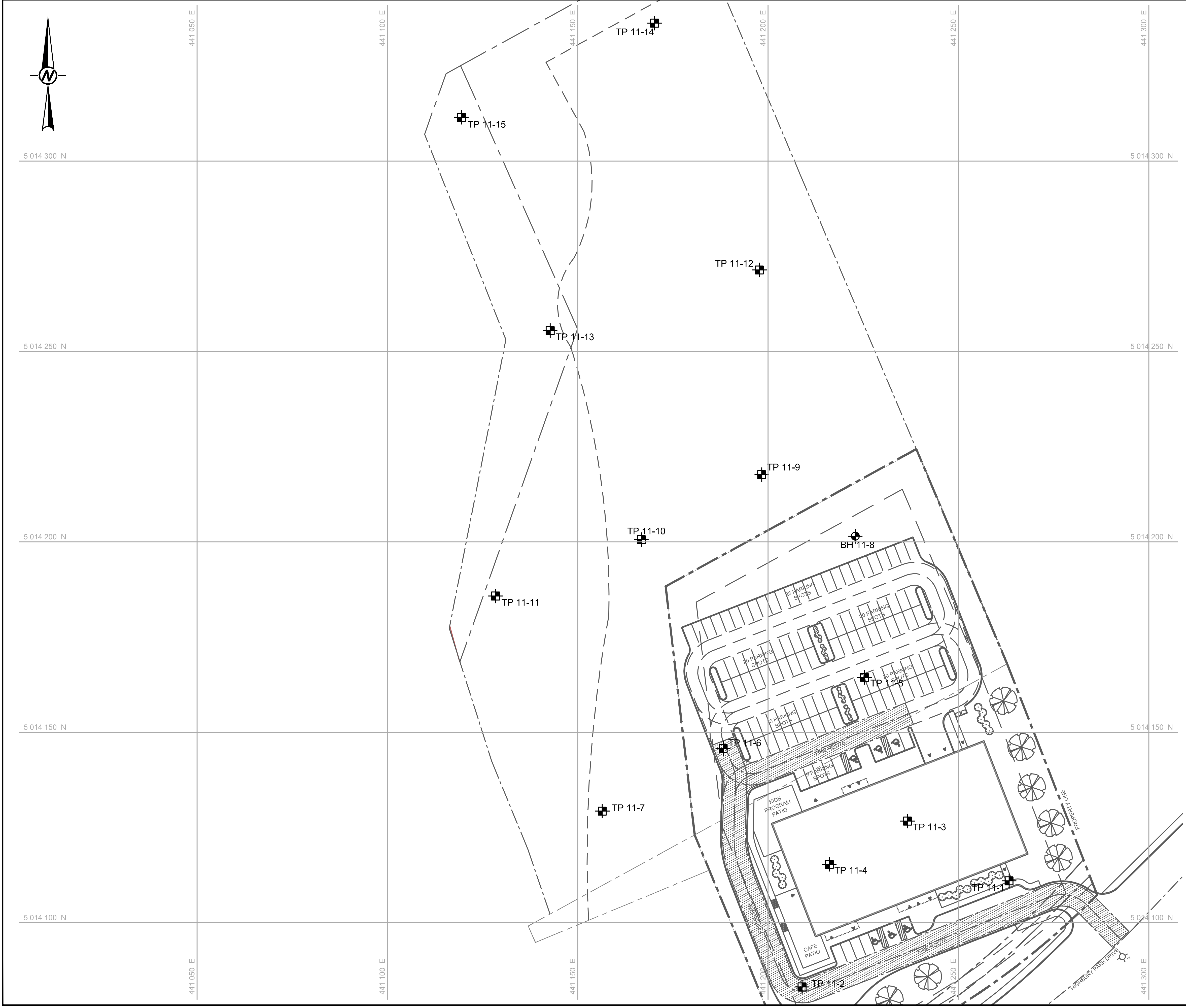
Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.30 – 0.50
2	1.00 – 1.20
3	2.00 – 2.37

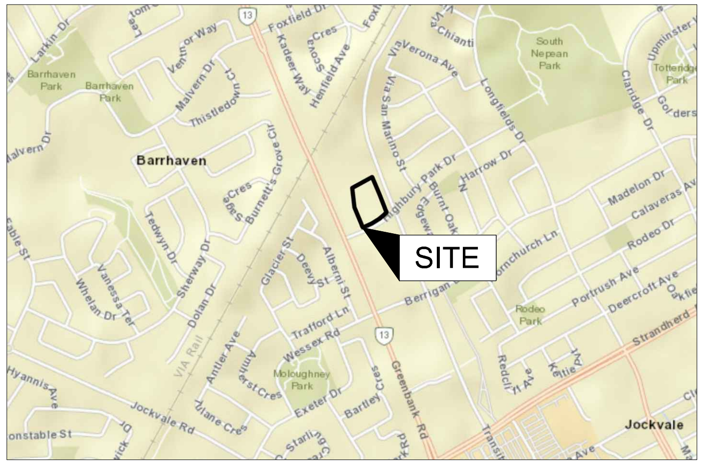
Test Pit Number (Elevation – m)	Depth (m)	Description
11-15	0.00 – 0.17	Dark brown TOPSOIL
(94.86)	0.17 – 0.40	Dark grey SILTY CLAY to CLAYEY SILT with organics
	0.40 – 1.40	Grey brown, stratified SANDY SILT
	1.40 – 1.80	Very dense grey brown SILTY SAND and GRAVEL, with cobbles and boulders (up to 0.6 m dia.) (GLACIAL TILL)
	1.80	SANDSTONE BEDROCK

Note: Test pit dry upon completion.

Sample No.	Depth (m)
1	0.20 – 0.40
2	0.60 – 0.80
3	1.40 – 1.60



KEY MAP



SCALE 1:25,000

LEGEND

⊕	APPROXIMATE BOREHOLE LOCATION
⊞	APPROXIMATE TEST PIT LOCATION
— - —	PROPERTY BOUNDARY

REFERENCE(S)

1. BASE PLAN SUPPLIED IN ELECTRONIC FORMAT BY MCDONALD BROTHERS CONSTRUCTION INC.
2. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83, COORDINATE SYSTEM: UTM ZONE 18, VERTICAL DATUM: CGVD28

0 25 50

1:1,000 METRES

CLIENT
POSSESS THE LAND

PROJECT
PROPOSED NEW CHURCH,
35 HIGHBURY PARK DRIVE, OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT

GOLDER

YYYY-MM-DD	2019-01-31
DESIGNED	AM
PREPARED	ABD
REVIEWED	AM
APPROVED	WC

PROJECT NO.	CONTROL	REV.	FIGURE
18114424	0001	1	1

Path: \\golder.gds\gal\ottawa\Active\Spatial_IM\Sequoia\35 HighburyPark Drive\99_PROJ\18114424_Sequoia\40_PROD\0001_Geotech\ | File Name: 18114424-0001-BG-0001.dwg

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

25 mm

0

APPENDIX A

Lithological and Geotechnical Rock Description Terminology
Record of Drillhole

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of rock material weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index
A count of the number of naturally occuring discontinuities (physical separations) in the rock core. Mechanically induced breaks caused by drilling are not included.

Dip with Respect to Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^o angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

PROJECT: 11-1121-0272

RECORD OF DRILLHOLE: 11-08

SHEET 1 OF 1

LOCATION: See Site Plan

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Nov. 25, 2011

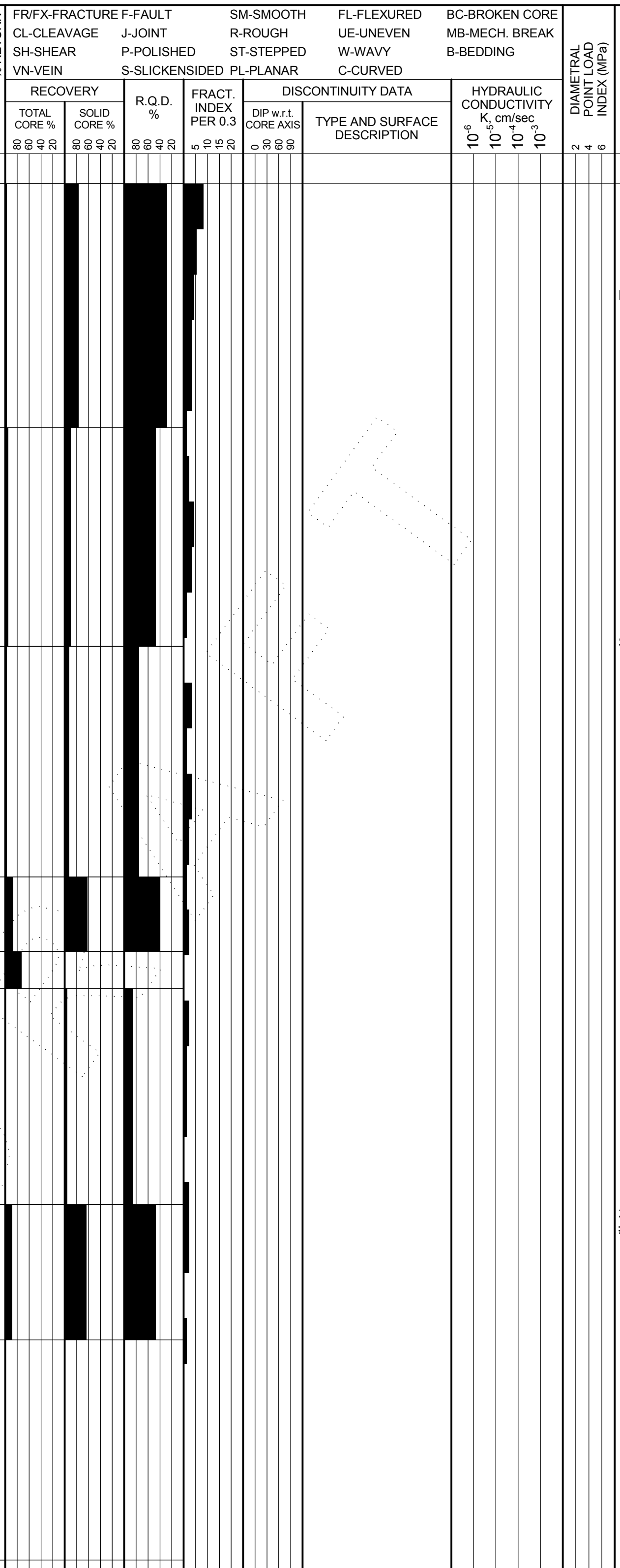
DRILL RIG: CME 55

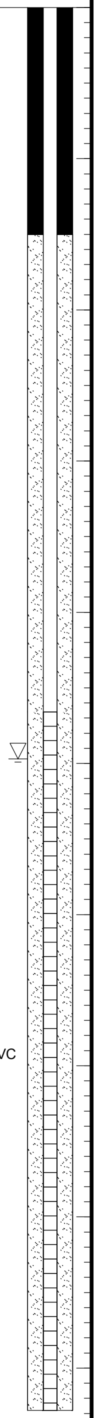
DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR/FX-FRACTURE	F-FAULT	SM-SMOOTH	FL-FLEXURED	BC-BROKEN CORE
CL-CLEAVAGE	J-JOINT	R-ROUGH	UE-UNEVEN	MB-MECH. BREAK
SH-SHEAR	P-POLISHED	ST-STEPPED	W-WAVY	B-BEDDING
VN-VEIN	S-SLICKENSIDED	PL-PLANAR	C-CURVED	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
		GROUND SURFACE	96.03		
0	Rotary Drill NQ Core	Slightly weathered yellow brown and light grey SANDSTONE	0.00	1	Bentonite Seal
2		Fresh grey SANDSTONE	93.59 2.44	2	
3					Silica Sand
4				3	
5				4 5	
6				6	
7				7	38 mm Diam. PVC #10 Slot Screen
8				8	
9		End of Drillhole	86.78 9.25		W.L in Screen at 91.06 m Elev. on Nov. 30, 2011





DEPTH SCALE

1 : 50

LOGGED: P.A.H.

CHECKED: W.A.M.

APPENDIX B

Results of Chemical Analysis
Exova Accutest Report No.1128825

EXOVA ACCUTEST

REPORT OF ANALYSIS

Exova Accutest

Client: Golder Associates Ltd. (Ottawa)
32 Steacie Drive

Kanata, ON
K2K 2A9
Attention: Mr. Alex Meacoe

Report Number: 1128825
Date: 2011-12-20
Date Submitted: 2011-12-12

Project: 11-1121-0272

P.O. Number:
Matrix: Soil

Chain of Custody Number: 138139

		LAB ID:	931701					**GUIDELINE**		
		Sample Date:	2011-11-24							
		Sample ID:	TP 11-10 Sa 1							
PARAMETER	**UNITS**	**MRL**						**TYPE**	**LIMIT**	**UNITS**
Chloride	%	0.002	<0.002							
Electrical Conductivity	mS/cm	0.05	0.06							
pH			6.2							
Resistivity	ohm-cm	1	16700							
Sulphate	%	0.01	0.06							

MRL = Method Reporting Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration

Comment:

APPROVAL:
Loma Wilson
Inorganic Lab Supervisor

Methods references and/or additional QA/QC information available on request.

EXOVA ACCUTEST

REPORT OF ANALYSIS

Client: Golder Associates Ltd. (Ottawa)
32 Steacie Drive

Kanata, ON
K2K 2A9
Attention: Mr. Alex Meacoe

Report Number: 1128825
Date: 2011-12-20
Date Submitted: 2011-12-12

Project: 11-1121-0272

P.O. Number:
Matrix: Soil

Chain of Custody Number: 138139

		LAB ID:						GUIDELINE		
		Sample Date:								
		Sample ID:	LAB BLANK	LAB QC % RECOVERY	QC RECOVERY RANGE	DATE ANALYSED				
PARAMETER	UNITS	MRL						TYPE	LIMIT	UNITS
Chloride	%	0.002	<0.002	104	90-110	2011-12-16				
Electrical Conductivity	mS/cm	0.05	<0.05	100	80-120	2011-12-19				
pH			<2.0	100	90-110	2011-12-19				
Resistivity	ohm-cm	1	<1		-	2011-12-19				
Sulphate	%	0.01	<0.01	106	70-130	2011-12-19				

MRL = Method Reporting Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration

Comment:

APPROVAL:
Lorna Wilson
Inorganic Lab Supervisor

Methods references and/or additional QA/QC information available on request.

GOLDER

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