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Civil • Geotechnical •
Structural • Environmental •
Hydrogeology

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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED CHURCH BUILDING ADDITION
1100 KENASTON STREET
CITY OF OTTAWA, ONTARIO

Submitted to:

Senhor Santo Cristo Parish c/o Jose Vaz
1100 Kenaston Street
Ottawa, Ontario
K1B 3P5

DISTRIBUTION

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November 2016 160749

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November 11, 2016 160749

Senhor Santo Cristo Parish c/o Jose Vaz
1100 Kenaston Street
Ottawa, Ontario
K1B 3P5

RE: GEOTECHNICAL INVESTIGATION
PROPOSED CHURCH ADDITION
1100 KENASTON STREET
CITY OF OTTAWA, ONTARIO

Dear Sirs:

This report presents the results of a geotechnical investigation carried out for the above noted proposed church building addition to be added on the west side of the existing church building at 1100 Kenaston Street in the City of Ottawa, Ontario. The purpose of the investigation was to identify the subsurface conditions at the site based on one borehole. Based on the factual information obtained, Kollaard Associates Inc. was to provide guidelines on the geotechnical engineering aspects of the project design; including construction considerations, which could influence design decisions.

BACKGROUND INFORMATION AND SITE GEOLOGY

The site area consists of an approximate 1.99 hectare (4.93 acre) parcel of land and is currently occupied by a church. The site is located along the south side of Kenaston Street, about 470 metres east of St. Laurent Street, in the City of Ottawa, Ontario (see Key Plan, Figure 1).

It is understood that plans are being prepared for the construction of a two storey, about 177 square metre building residential addition to the existing church. The building is to be of wood frame construction with conventional concrete spread footing foundations and a concrete slab on grade floor. Surface drainage for the proposed building addition will be by sheet flow over the existing parking area along the current drainage patterns to an existing catch basin.

The site is bordered on the east, west and south by commercial development and on the north by residential development. A large storm pond exists southeast of the site.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by glacial till overlying relatively shallow bedrock (*Surficial Geology Map*: Geological Survey of Canada, Surficial Geology, Ottawa, Ontario, Map 1506A, published 1982, scale 1:50,000). A review of the bedrock geology map indicates that the bedrock underlying the site consists of black shale of the Billings Formation (*Bedrock Geology Map*: Geological Survey of Canada, Generalized Bedrock Geology, Ottawa-Hull, Ontario and Quebec, Map 1508A, published 1979, scale 1:125,000).

PROCEDURE

The field work for this investigation was carried out on October 28, 2016. At the time of the field work, one borehole, numbered borehole 1 was put down at the site using a truck mounted drill rig equipped with a hollow stem auger owned and operated by OGS Drilling of Almonte, Ontario. The location of the proposed church building addition was indicated to us on a site plan provided by 'A Cubed' Studio, Drawing Number A1.00, rev. A, dated October 2014.

Sampling of the overburden materials encountered at the boreholes was carried out at regular 0.75 metre depth intervals using a 50 millimetre diameter drive open conventional split spoon sampler in conjunction with standard penetration testing to a depth of about 4.3 metres below the existing ground surface (ASTM D-1586 – Penetration Test and Split Barrel Sampling of Soils).

The subsurface conditions encountered at the borehole was classified based on visual examination of the samples recovered, the results of the standard penetration tests as well as laboratory test results on select samples (ASTM D2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). The soils were classified using the Unified Soil Classification System. No in situ vane shear testing (ASTM D-2573 Standard Test Method for Field Shear Test in Cohesive Soil) was carried as no cohesive materials were encountered within the borehole. The groundwater conditions were observed in the open borehole at the time of drilling.

One soil sample from the borehole was submitted for hydrometer analysis (ASTM D422). A sample of soil was also delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack and soil corrosion on buried concrete and steel.

The field work was supervised throughout by a member of our engineering staff who located the borehole in the field, logged the borehole and cared for the samples obtained. A description of the subsurface conditions encountered at the borehole is given in the attached Record of Borehole Sheet. The results of the laboratory testing of the soil sample are presented in the Laboratory Test

Results section and Attachment A following the text in this report. The approximate locations of the borehole are shown on the attached Site Plan, Figure 2.

The ground surface elevation at the borehole location was determined, in the field, relative to a site benchmark provided by Stantec Geomatics Ltd., Legal Survey, Project Number 16163354-111, dated July 28, 2015. The site benchmark is described as the top of door sill located on the west side of the existing building as identified on Kollaard Associates Inc., Proposed Site Grading Plan, Project 160749-1, dated October 25, 2016. The elevation of the door sill is referenced as 70.62 metres.

SUBSURFACE CONDITIONS

General

As previously indicated, a description of the subsurface conditions encountered at the boreholes is provided in the attached Record of Borehole Sheets following the text of this report. The borehole logs indicate the subsurface conditions at the specific drill locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at locations other than borehole locations may vary from the conditions encountered at the boreholes.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification was in general completed by visual-manual procedures in accordance with ASTM 2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) with select samples being classified by laboratory testing in accordance with ASTM 2487. The soils were classified using the Unified Soil Classification System. Classification and identification of soil involves judgement and Kollaard Associates Inc. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The groundwater conditions described in this report refer only to those observed at the location and on the date the observations were noted in the report and on the borehole log. Groundwater conditions may vary seasonally, or may be affected by construction activities on or in the vicinity of the site.

The following is a brief overview of the subsurface conditions encountered at the borehole.

Fill

Fill material was encountered from the surface at the borehole. The fill material consisted of about a 0.1 metres thickness of asphaltic concrete followed by about a 0.25 metre thickness of grey crushed stone then by yellow brown to grey brown sand, gravel with a trace of silt and wood debris. The fill materials was observed to have a total thickness of about 2.3 metres.

Glacial Till

A deposit of glacial till was encountered beneath the fill materials at the borehole location. The glacial till consisted of gravel, cobbles and boulders, in a matrix of dark grey silty sand with a trace of silty clay and shale fragments. The results of standard penetration testing carried out in the glacial till material, which range from 49 to 62 blows per 0.3 metres with an average value of 56 blows per 0.3 metres, indicate a very dense state of packing.

One soil sample (BH1 - SS6 - 3.8-4.4 metres) was submitted to Stantec for hydrometer testing (ASTM D422). The results of the hydrometer testing indicated that the sample consists of about 5.0 percent clay, 25 percent silt and clay size particles, 40 percent sand and 30 percent gravel. The results are located in Attachment A.

The borehole encountered refusal to further advancement on either large boulders or bedrock at a depth of about 4.3 metres below the existing ground surface.

Bedrock

The borehole was terminated on the surface of bedrock or large boulders with practical refusal at a depth of about 4.3 metres below the existing ground surface level.

Groundwater

The borehole was dry at the time of the field work. It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

Sulphate, Resistivity and pH

The results of the laboratory testing of a soil sample for submitted for chemistry testing related to corrosivity is summarized in the following table.

Item	Threshold of Concern	Test Result	Comment
Chlorides (Cl)	Cl > 0.04 %	0.003	Negligible concern
pH	5.0 < pH	7.8	Neutral/Slightly Basic Negligible concern
Resistivity	R < 1500 ohm-cm	2780	Moderate concern
Sulphates (SO_4)	SO_4 > 0.1%	0.02	Negligible concern

The results of the laboratory testing of a soil sample for sulphate gave a percent sulphate of <0.01. The National Research Council of Canada (NRC) recognizes four categories of potential sulphate attack of buried concrete based on percent sulphate in soil. From 0 to 0.10 percent the potential is negligible, from 0.10 to 0.20 percent the potential is mild but positive, from 0.20 to 0.50 percent the potential is considerable and 0.50 percent and greater the potential is severe. Based on the above, the soils are considered to have a negligible potential for sulphate attack on buried concrete materials and accordingly, conventional GU or MS Portland cement may be used in the construction of the proposed concrete elements

The results of the laboratory testing of a soil sample for resistivity and pH indicates the soil sample tested has an underground corrosion rate of about 0.7 loss-oz./ft^2/yr (2780 ohms-cm). Based on the findings of Fischer and Bue (1981) underground corrosion rates (loss-oz./ft^2/yr) of 0.30 and less are considered nonaggresive, from 0.30 to 0.75 the rate is considered slightly aggressive, from 0.75 to 2.0 the rate is considered aggressive and 2.0 and greater the rate is considered very aggressive. Accordingly, the above mentioned soil sample is considered to have a slightly to mildly aggressive corrosion rate to reinforcement steel within below grade concrete walls.

The pH value for the soil sample was reported to be at 7.8, indicating a durable condition against corrosion. This value was evaluated using Table 2 of Building Research Establishment (BRE) Digest 362 (July 1991).The pH is greater than 5.5 indicating the concrete will not be exposed to attack from acids. The chloride content of the sample was also compared with the threshold level and present negligible concrete corrosion potential.

The chloride content of the sample was also compared with the threshold level and present negligible concrete corrosion potential. Soil resistivity was found to be 2.78 ohm-m for the sample analyzed. Consideration to increasing the specified strength and/or adding air entrainment into any reinforced concrete in contact with the soil should be given. Minimum cover requirements as specified by the American Concrete Institute for concrete construction should be followed.

GEOTECHNICAL DESIGN GUIDELINES

General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the information from the test holes and the project requirements. It is stressed that the information in the following sections is provided for the guidance of the designers and is intended for this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from off site sources are outside the terms of reference for this report.

Foundation for Proposed Church Building Addition

With the exception of the fill materials, the subsurface conditions encountered at the borehole advanced during the investigation are suitable for the support of the proposed church building addition on conventional spread footing foundations. The excavations for the foundations should be taken down through any fill (silty sand and gravel) or otherwise deleterious material to expose the native, undisturbed glacial till. The subgrade surface should then be inspected and approved by geotechnical personnel. The excavations to remove the fill materials and within the silty clay above the groundwater level should not present any serious constraints.

The allowable bearing pressure for any footings depends on the depth of the footings below original ground surface, the width of the footings, and the height above the original ground surface of any landscape grade raise adjacent to the dwelling foundation.

For the proposed building addition, strip and pad footings, a minimum 0.5 metres in width bearing on the native undisturbed glacial till above the groundwater level may be designed using a maximum allowable bearing pressure of 150 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 250 kilopascals using ultimate limit states design.

Since the proposed addition will be constructed in a manner that preserves the existing grading and drainage patterns, there are no additional grade raise restrictions associated with the above bearing capacities.

Provided that any loose and/or disturbed soil is removed from the bearing surfaces prior to pouring concrete, the total and differential settlement of the footings should be less than 20 millimetres and 15 millimetres, respectively.

Because the bedrock underlying the till at the site consists of Billings Shale, it is recommended that a minimum thickness of 0.5 metres of native overburden be left in place above the bedrock.

Should complete removal of all fill and/or deleterious material result in an subgrade surface below the proposed founding level, the subgrade should be built up using engineered fill.

Any fill required to raise the subgrade for the proposed building addition to founding level should consist of imported granular material (engineered fill). The engineered fill should consist of granular material meeting Ontario Provincial Standards Specifications (OPSS) requirements for Granular A or Granular B Type II and should be compacted in maximum 200 millimetre thick loose lifts to at least 95 percent of the standard Proctor maximum dry density. To allow the spread of load beneath the footings, the engineered fill should extend down and out from the edges of the footing at 1 horizontal to 1 vertical, or flatter. The excavations for the proposed building addition should be sized to accommodate this fill placement. Currently, OPSS documents allow recycled asphaltic concrete to be used in Granular A and Granular B Type II materials. It is considered that that any granular materials used below the founding level be composed of virgin materials only. Compaction should be verified by a suitable field compaction test method.

Frost Protection Requirements for Spread Footing Foundations

All exterior footings and those in any unheated parts of the proposed building addition should be provided with at least 1.5 metres of earth cover for frost protection purposes. Exterior footings constructed in areas that are to be cleared of snow during the winter period should be provided with at least 1.8 metres of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided using a combination of earth cover and extruded polystyrene insulation. Detailed guidelines for footing insulation frost protection could be provided upon request.

The native glacial till soils at this site will be sensitive to disturbance from construction operations and from rainwater or snowmelt, and frost. In order to minimize disturbance, construction traffic operating directly on the subgrade should be kept to an absolute minimum and the subgrade should be protected from below freezing temperatures.

Building Basement Foundation Walls and Drainage

A conventional, perforated perimeter drain should be provided at founding level, leading by gravity flow to a sump or storm sewer. The drain should be installed at footing level and provided with a 150 millimetre thick surround of 20 millimetre minus crushed stone. The drain should be provided with a backflow preventer.

It is considered that in view of the groundwater conditions observed at the boreholes, the above perimeter drainage system should adequately handle any groundwater seepage to the basements.

The native soils encountered at this site are considered to be frost susceptible. As such, to prevent possible foundation frost jacking, the backfill against any unheated or insulated walls or isolated walls or piers should consist of free draining, non-frost susceptible material. If imported material is required, it should consist of sand or sand and gravel meeting OPSS Granular B Type I grading requirements. Alternatively, foundations could be backfilled on the exterior with native material in conjunction with the use of an approved proprietary drainage layer system against the foundation wall. It is pointed out that there is potential for possible frost jacking of the upper portion of some types of these drainage layer systems if frost susceptible material is used as backfill. This could be mitigated by backfilling the upper approximately 0.6 metres with non-frost susceptible granular material. Where the granular backfill will ultimately support a pavement structure or walkway, it is suggested that the wall backfill material be compacted in 250 millimetre thick lifts to 95 percent of the standard Proctor dry density value.

Groundwater inflow from the native soils into the basement excavations during construction, if any should be handled by pumping from sumps within the excavations.

The basement foundation walls should be designed to resist the earth pressure, P, acting against the walls at any depth, h, calculated using the following equation.

$$P = k_0 (\gamma h + q)$$

Where:

	P	=	the pressure, at any depth, h, below the finished ground surface
	k_0	=	earth pressure at-rest coefficient, 0.5
	γ	=	unit weight of soil to be retained, estimated at 22 kN/m^3
	q	=	surcharge load (kPa) above backfill material
	h	=	the depth, in metres, below the finished ground surface at which the pressure, P, is being computed

This expression assumes that the water table would be maintained at the founding level by the above mentioned foundation perimeter drainage and backfill requirements.

Where the backfill material will ultimately support a pavement structure or walkway, it is suggested that the foundation wall backfill material be compacted in 250 millimetre thick lifts to 95 percent of the standard Proctor dry density value.

Building Structure Floor Slab

As stated above, it is expected that the proposed building addition will be founded on glacial till. For predictable performance of the proposed concrete floor slab all existing fill material, topsoil and any otherwise deleterious material should be removed from below the proposed floor slab areas. The exposed native subgrade surface should then be inspected and approved by geotechnical personnel. Any soft areas evident should be subexcavated and replaced with suitable engineered fill.

The fill materials beneath the proposed concrete floor slab on grade should consist of a minimum of 150 millimetre thickness of crushed stone meeting OPSS Granular A immediately beneath the concrete floor slab followed by sand, or sand and gravel meeting the OPSS for Granular B Type I, or crushed stone meeting OPSS grading requirements for Granular B Type II, or other material approved by the Geotechnical Engineer. Crushed concrete meeting OPSS grading requirements for Granular B Type II material may also be used as engineered fill beneath the proposed floor stab. The fill materials should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage of the concrete. The saw cut depth should be about one quarter of the thickness of the slab. The crack control cuts should be placed at a grid spacing not exceeding the lesser of 25 times the slab thickness or 4.5 metres.

Groundwater inflow from the native soils into the basement excavation during construction, if any should be handled by pumping from sumps within the excavations.

If any areas of the proposed building addition are to remain unheated during the winter period, thermal protection of the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary.

Foundation Wall Backfill and Drainage

To prevent possible foundation frost jacking due to frost adhesion, the backfill against the foundation walls or isolated walls or piers should consist of free draining, non-frost susceptible material such as sand or sand and gravel meeting OPSS Granular B Type I grading requirements. Alternatively, foundations could be backfilled with native material in conjunction with the use of an approved proprietary drainage layer system such as "System Platon" against the foundation wall. It is pointed out that there is potential for possible frost jacking of the upper portion of some types of these drainage layer systems if frost susceptible material is used as backfill. This could be mitigated by backfilling the upper approximately 0.6 metres with non-frost susceptible granular material.

Where the backfill material will ultimately support a pavement structure or walkway, it is suggested that the foundation wall backfill material be compacted in 250 millimetre thick lifts to 95 percent of the standard Proctor dry density value.

Seismic Design for the Proposed Building Addition

For seismic design purposes, in accordance with the 2012 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class D.

Seismic Site Response Site Class Calculation

Borehole 1					
Layer	Description	Depth (m)	d_i (m)	$N(60)_i$ (blows/0.3m)	d_i/N_i (blows/0.3m)
1	Fill	0	2.28	20	0.114
2	Glacial Till	2.28	2.04	56	0.037
3	Bedrock	4.32	25.68	100	0.257
$sum(d_i/N(60)_i)$					0.407
$d_c/(sum(d_i/N(60)_i)$					73.6

Since the 50 < N(60) = 73.6 < 100, the seismic site response is Site Class C.

Potential for Soil Liquefaction

As indicated above, the results of the borehole indicate that the native deposits underlying the proposed building foundation consist of glacial till followed by bedrock. As these materials are not prone to liquefaction, it is considered that no damage to the proposed building addition should occur due to liquefaction of the native subgrade under seismic conditions.

SITE SERVICES

The proposed addition will be serviced through the existing building. There will be no new exterior service installation. Therefore recommendations for service trench excavation and back fill are not provided in this report.

ACCESS ROADWAY AND PARKING AREA PAVEMENTS

As previously indicated, the proposed addition will be constructed on the west side of the existing church building. The excavation for the addition will be contained within a portion of the site currently occupied by landscaping, sidewalk and parking area. The proposed addition will not affect or alter the proposed access roadway into the site.

Where portions of the existing parking area are disturbed during excavation and construction of the addition, the parking area should be reinstated as follows:

Subgrade Preparation

In preparation for pavement construction at this site the fill and topsoil and any soft, wet or deleterious materials should be removed from the proposed parking lot area. Based on the results of the test pits, the subsurface conditions in the proposed access roadway and parking areas consist of topsoil overlying grey brown silty clay.

The exposed subgrade should be inspected and approved by geotechnical personnel and any soft areas evident should be subexcavated and replaced with suitable earth borrow approved by the geotechnical engineer. The subgrade should be shaped and crowned to promote drainage of the roadway and parking area granulars. Following approval of the preparation of the subgrade, the pavement granulars may be placed.

For any areas of the site that require the subgrade to be raised to proposed roadway and parking area subgrade level, the material used should consist of OPSS select subgrade material, OPSS Granular B Type I or Type II, or crushed concrete meeting the grading requirements for Granular B Type II. Materials used for raising the subgrade to proposed roadway and parking area subgrade level should be placed in maximum 300 millimetre thick loose lifts and be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

Asphaltic Concrete Surfaced Areas:

It is suggested that provision be made for the following minimum pavement structure:

50 millimetres of hot mix asphaltic concrete (HL3) or Superpave 12.5 over
150 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase
(50 or 100 millimetre minus crushed stone)

Performance grade PG 58-34 asphaltic concrete should be specified. Compaction of the granular pavement materials should be carried out in maximum 300 millimetre thick loose lifts to 100 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment.

The above pavement structures will be adequate on an acceptable subgrade, that is, one where any roadway fill and service trench backfill has been adequately compacted. If the roadway subgrade is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or incorporate a non-woven geotextile separator between the roadway subgrade surface and the granular subbase material.

CONSTRUCTION CONSIDERATIONS

It is suggested that the final design drawings for the site, including the proposed site grading plan, be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed development do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

All foundation areas and any engineered fill areas for the proposed church building addition should be inspected by Kollaard Associates Inc. to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundation should be inspected to ensure that the materials used conform to the grading and compaction specifications.

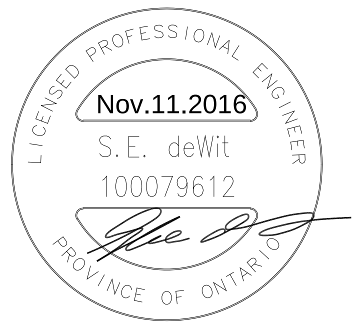
The subgrade for any parking areas should be inspected and approved by geotechnical personnel. In situ density testing should be carried out on the pavement granular materials to ensure the materials meet the specifications from a compaction point of view.

The native silty sand and silty clay within the glacial till materials at this site will be sensitive to disturbance from construction operations, from rainwater or snow melt, and frost. In order to minimize disturbance, construction traffic operating directly on the subgrade should be kept to an absolute minimum and the subgrade should be protected from below freezing temperatures.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we may be of further services to you, please do not hesitate to contact our office.

Regards,

Kollaard Associates Inc.



Dean Tataryn, B.E.S., EP. Steven deWit, P.Eng.

Attachments: Record of Borehole
Key Plan, Figure 1
Site Plan, Figure 2
Laboratory Test Results for Chemical Properties
Laboratory Test Results for Physical Properties – Stantec Laboratory Test Results for Soils

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
MS manual sample
RC rock core
ST slotted tube .
TO thin-walled open Shelby tube
TP thin-walled piston Shelby tube
WS wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance, N

The number of blows by a 63.5 kg hammer dropped 760 millimeter required to drive a 50 mm drive open sampler for a distance of 300 mm. For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

Dynamic Penetration Resistance

The number of blows by a 63.5 kg hammer dropped 760 mm to drive a 50 mm diameter, 60° cone attached to 'A' size drill rods for a distance of 300 mm.

WH

Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drih rig.

PM

Sampler advanced by manual pressure.

SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH sieve and hydrometer analysis
U unconfined compression test
Q undrained triaxial test
V field vane, undisturbed and remolded shear strength

SOIL DESCRIPTIONS

Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to100
Very Stiff	over100

LIST OF COMMON SYMBOLS

Cu undrained shear strength
e void ratio
Cc compression index
Cv coefficient of consolidation
k coefficient of permeability
Ip plasticity index
n porosity
u pore pressure
w moisture content
wL liquid limit
Wp plastic limit
$1 effective angle of friction
r unit weight of soil
y1 unit weight of submerged soil
cr normal stress

RECORD OF BOREHOLE BH1

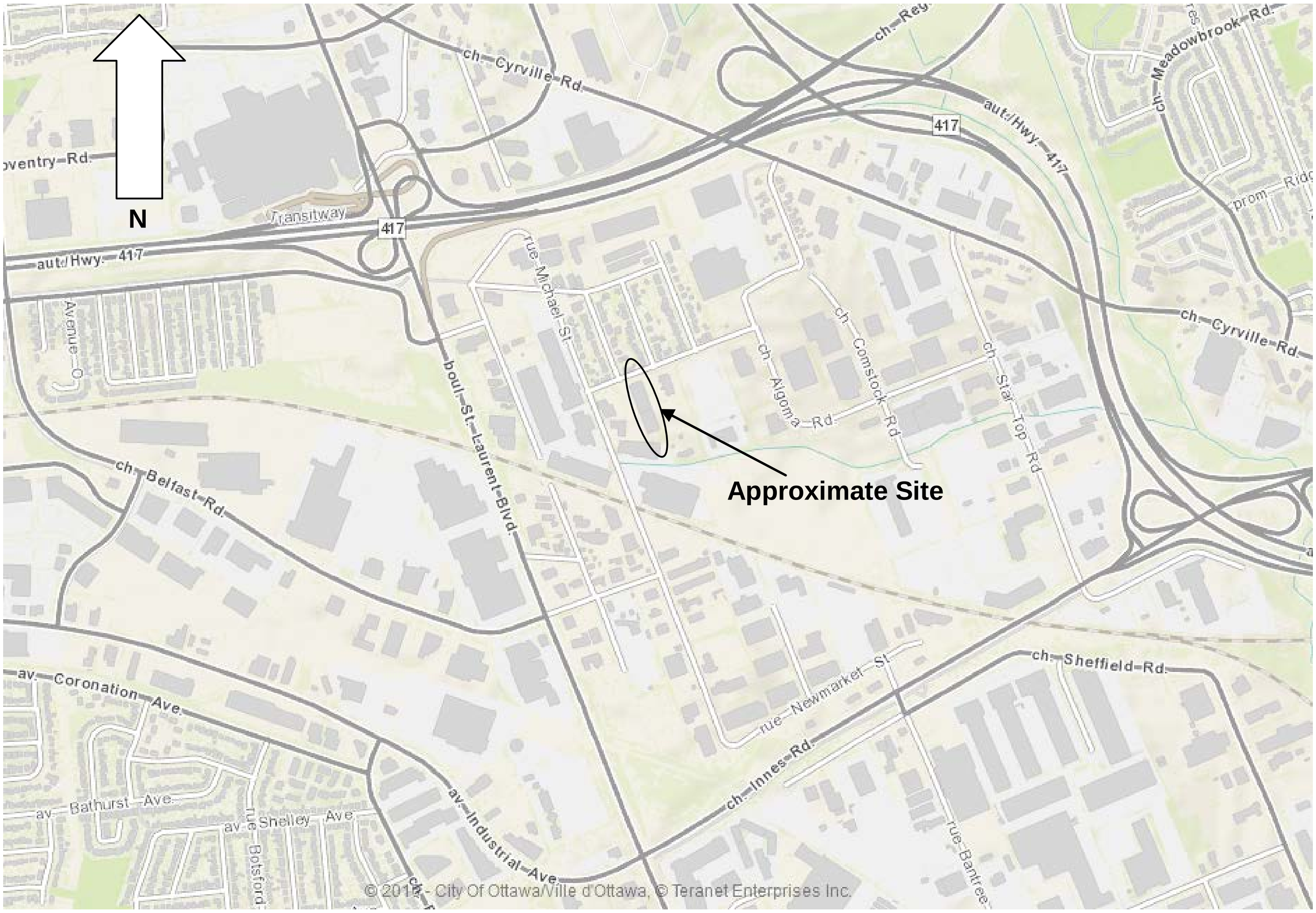
PROJECT: Proposed Church Building Addition
CLIENT: Senhor Santo Cristo Parish c/o Jose Vaz
LOCATION: 1100 Kenaston Street, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 160749
DATE OF BORING: October 28, 2016
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface									
0	Asphaltic Concrete (FILL)		0.00							
	Grey crushed stone (FILL)									
	Yellow brown to grey brown sand and gravel, trace silt and wood (FILL)		0.25	1	SS	23				
1				2	SS	25				
2				3	SS	12				
	Dark grey silty sand, some gravel, cobbles and boulders, trace clay and shale fragments (GLACIAL TILL)		2.28	4	SS	56				
3				5	SS	49				
4	Grey brown silty sand, some gravel,cobbles and boulders, trace clay (GLACIAL TILL)		3.80	6	SS	62				
	Refusal on BEDROCK		4.32							
5										

DEPTH SCALE: 1 to 30
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: DT

KEY PLAN	FIGURE 1



NOT TO SCALE

Kollaard Associates
Engineers

Project No. 160749

Date November 2016

DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

BH1 APPROXIMATE BOREHOLE LOCATION

REFERENCE: PLAN SUPPLIED BY CITY OF OTTAWA EMAPS.

SPECIAL NOTE: THIS DRAWING TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING REPORT.

REV.	NAME	DATE	DESCRIPTION

Kollaard Associates
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KEMPTVILLE ONTARIO
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http://www.kollaard.ca
(613) 860-0923
info@kollaard.ca

CLIENT:
SENHOR SANTO CRISTO PARISH
C/O JOSE VAZ

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED CHURCH BUILDING ADDITION

LOCATION:
1100 KENASTON STREET
CITY OF OTTAWA, ONTARIO

DESIGNED BY: --	*DATE:* OCTOBER 28, 2016
DRAWN BY: DT	*SCALE:* N.T.S

KOLLAARD FILE NUMBER:
160749

Laboratory Test Results for Chemical Properties

Certificate of Analysis

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0	Report Number:	1619518
		Date Submitted:	2016-11-01
		Date Reported:	2016-11-07
Attention:	Mr. Dean Tataryn	Project:	
PO#:		COC #:	183888
Invoice to:	Kollaard Associates Inc.		

Dear Dean Tataryn:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: ______________________________

Shyla Monette
Team Leader, Inorganics

All analysis is completed in Ottawa, Ontario (unless otherwise indicated).

Exova Ottawa is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on our CALA scope of accreditation. It can be found at http://www.cala.ca/scopes/2602.pdf.

Exova (Ottawa) is certified and accredited for specific parameters by OMAFRA, Ontario Ministry of Agriculture, Food and Rural Affairs (for farm soils). Licensed by Ontario MOE for specific tests in drinking water.

Exova (Mississauga) is accredited for specific parameters by SCC, Standards Council of Canada (to ISO 17025)

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Exova recommends consulting the official provincial or federal guideline as required.

Certificate of Analysis

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0
Attention:	Mr. Dean Tataryn
PO#:	
Invoice to:	Kollaard Associates Inc.

Report Number:	1619518
Date Submitted:	2016-11-01
Date Reported:	2016-11-07
Project:	
COC #:	183888

				Lab I.D.	1267666
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2016-10-28
				Sample I.D.	BH1-SS6
Group	**Analyte**	**MRL**	**Units**	**Guideline**	
Agri. - Soil	pH	2.0			7.8
General Chemistry	Cl	0.002	%		0.003
	Electrical Conductivity	0.05	mS/cm		0.36
	Resistivity	1	ohm-cm		2780
	SO4	0.01	%		0.02

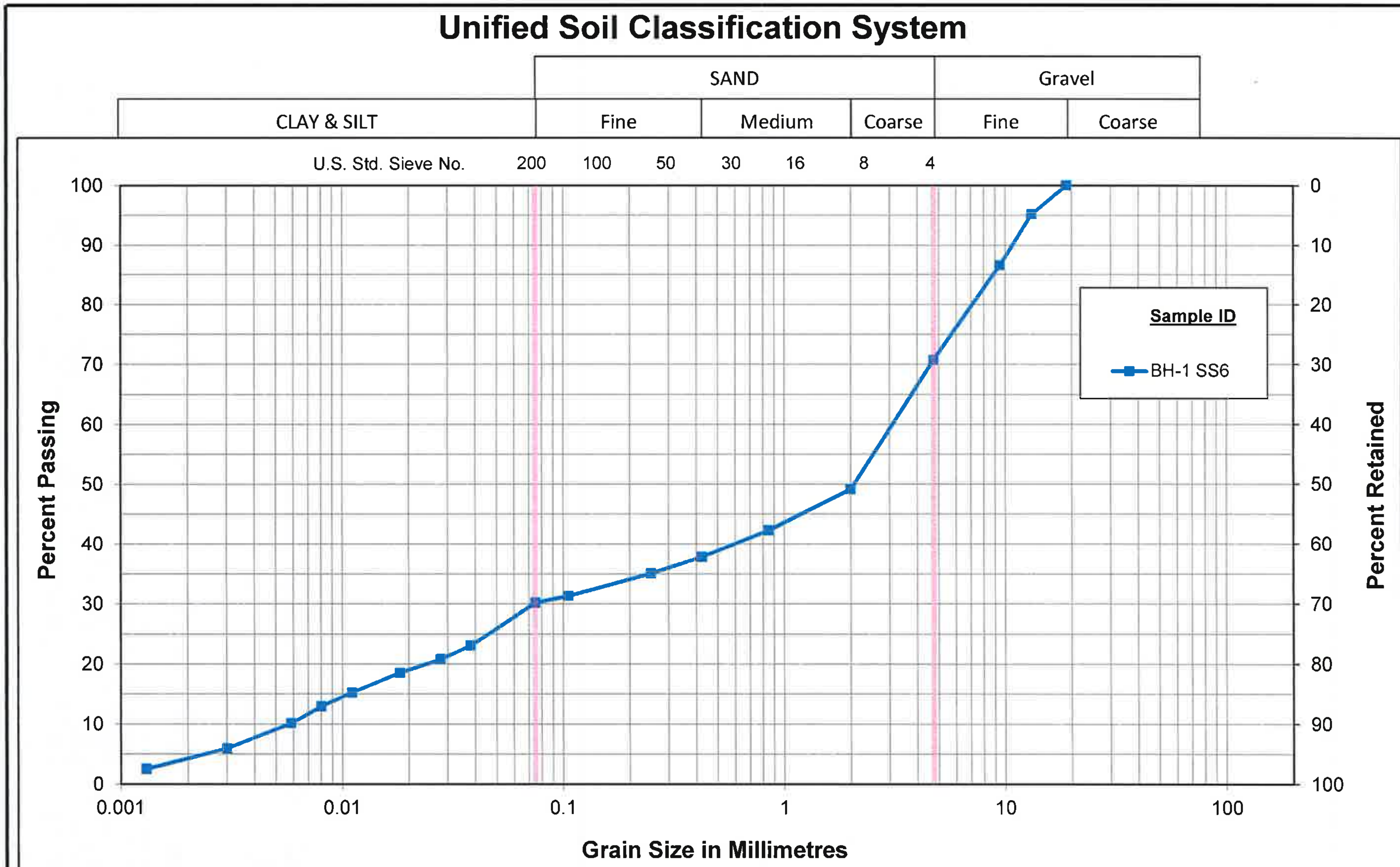
Guideline = *** = Guideline Exceedence**

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

146 Colonnade Rd. Unit 8, Ottawa, ON K2E 7Y1

Laboratory Test Results for Physical Properties



Stantec	GRAIN SIZE DISTRIBUTION Kollaard Associates Engineers, File #160749 Proposed Church Addition, 1100 Kenaston St., Ottawa	Figure No. Project No. 122410003

Stantec
2781 Lancaster Road, Suite 101
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASSHTO T 88

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #160749	Project No.:	122410003
Project:	Proposed Church Addition, 1100 Kenaston St., Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH-1	Date Sampled:	October 28, 2016
Sample No.:	SS6	Tested By:	Denis Rodriguez
Sample Depth:	12'6"-14'6"	Date Tested:	November 4, 2016

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	85.66
Sample Weight after Hydrometer and Wash (g)	33.46
Percent Passing No. 200 Sieve (%)	60.9
Percent Passing Corrected (%)	29.99

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	482.20
Sample Weight After Sieve (g)	477.60
Percent Loss in Sieve (%)	0.95

SOIL INFORMATION

Liquid Limit (LL)	N/A	
Plasticity Index (PI)	N/A	
Soil Classification		
Specific Gravity (G_s)	2.750	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	40	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	142.98
Air Dried Mass (W_a), (g)	145.03
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9859
Air Dried Mass in Analysis (M_a), (g)	86.89
Oven Dried Mass in Analysis (M_o), (g)	85.66
Percent Passing 2.0 mm Sieve (P_{10}), (%)	49.21
Sample Represented (W), (g)	174.07

SIEVE ANALYSIS

Sieve Size mm	Cum. Wt. Retained	Percent Passing
75.0		100.0
63.0		100.0
53.0		100.0
37.5		100.0
26.5		100.0
19.0	0.0	100.0
13.2	23.5	95.1
9.5	64.4	86.6
4.75	140.9	70.8
2.00	244.9	49.2
Total (C + F)[1]	477.60	
0.850	12.02	42.31
0.425	19.76	37.86
0.250	24.54	35.11
0.106	31.14	31.32
0.075	33.11	30.19
PAN	33.17	

Note 1: (C + F) = Coarse + Fine

HYDROMETER DETAILS

Volume of Bulb (V_B), (cm^3)	63.0
Length of Bulb (L_2), (cm)	14.47
Length from '0' Reading to Top of Bulb (L_1), (cm)	10.29
Scale Dimension (h_s), (cm/Div)	0.155
Cross-Sectional Area of Cylinder (A), (cm^2)	27.2
Meniscus Correction (H_m), (g/L)	1.0

START TIME 9:23 AM

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
04-Nov-16	9:24 AM	1	48.0	7.0	22.5	41.0	23.04	8.77191	9.5030	0.0129	0.0382
04-Nov-16	9:25 AM	2	44.0	7.0	22.5	37.0	20.80	9.39191	9.5030	0.0129	0.0279
04-Nov-16	9:28 AM	5	40.0	7.0	22.5	33.0	18.55	10.01191	9.5030	0.0129	0.0182
04-Nov-16	9:38 AM	15	34.0	7.0	22.0	27.0	15.18	10.94191	9.6157	0.0130	0.0111
04-Nov-16	9:53 AM	30	30.0	7.0	22.0	23.0	12.93	11.56191	9.6157	0.0130	0.0081
04-Nov-16	10:23 AM	60	25.0	7.0	22.0	18.0	10.12	12.33691	9.6157	0.0130	0.0059
04-Nov-16	1:33 PM	250	17.5	7.0	22.0	10.5	5.90	13.49941	9.6157	0.0130	0.0030
05-Nov-16	9:23 AM	1440	11.5	7.0	21.5	4.5	2.53	14.42941	9.7308	0.0130	0.0013

Remarks:

Reviewed By: Brian Prevost
Date: November 7/2016

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