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Civil • Geotechnical •
Structural • Environmental •
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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED CHURCH BUILDING ADDITIONS
1234 PRESTONE DRIVE, ORLEANS
CITY OF OTTAWA, ONTARIO

Submitted to:

St. Helen's Anglican Church
1234 Prestone Drive
Orleans, Ontario
K2S 1A3

DISTRIBUTION

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August 2013 130476

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August 27, 2013 130476

St. Helen's Anglican Church
1234 Prestone Drive
Orleans, Ontario
K2S 1A3

RE: GEOTECHNICAL INVESTIGATION
PROPOSED CHURCH BUILDING ADDITIONS
1234 PRESTONE DRIVE, ORLEANS
CITY OF OTTAWA, ONTARIO

Dear Sirs:

This report presents the results of a geotechnical investigation carried out for the above noted proposed church building additions to be added on the northwest side and south sides of the existing church building located southeast of the intersection of Prestone Drive and Kennedy Lane in the City of Ottawa, Ontario (see Key Plan, Figure 1). The purpose of the investigation was to identify the subsurface conditions at the site based on a limited number of boreholes and an augerhole. 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 subject site for this assessment is located at 1234 Prestone Drive in the City of Ottawa, Ontario. The site consists of an approximate 0.9 hectares (2.25 acres) parcel of land and is currently occupied by St. Helen's Anglican Church.

It is understood that plans are being prepared for the construction of three additions to the existing church along with an associated parking lot extension. It is understood based on a discussion with Vandenberg & Wildeboer Architects Inc. that the additions will consist of about a 213 square metre building addition on the south side, about a 50 square metre addition on the west side and about a 29 square metre addition on the north side along with an associated parking lot. The building additions are likely to be of wood frame construction with conventional concrete spread footing foundations. There is no basement or crawl space in the existing building and are no proposed basements or crawl spaces for the proposed additions.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by deposits of clay. A review of the bedrock geology map indicates that the bedrock underlying the site consists of dolomite or limestone of the Oxford Formation or limestone with some shaly partings of the Ottawa Formation.

PROCEDURE

The field work for this investigation was carried out on August 14, 2013 at which two boreholes, numbered BH1 and BH2 and one augerhole number AH1 were put down at the site using a truck mounted drill rig equipped with a hollow stem auger owned and operated by George Downing Estate Drillind Ltd. of Grenville, Quebec.

The location of the proposed church building additions were indicated to us on a site plan provided by Vandenbeg & Wilderboer Architects., Project 1324, Drawing Number A001, dated April 2013.

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 depths of about 9.1 and 9.8 metres below the existing ground surface in BH1 and BH2, respectively. The subsurface soil conditions at BH1 and BH2 were identified based on visual examination of the samples recovered and the results of the in situ vane shear testing and standard penetration tests. In situ vane shear testing was carried out in the cohesive materials encountered at both boreholes. The subsurface soil conditions at AH1 was identified based on visual examination of the upper about 2.4 metres of the open augerhole and visual and tactile examination of the recovered auger cuttings.

BH1 was continued below 6.7 metres as a probe hole using dynamic cone penetration testing. The augerhole was advanced using the same drilling equipment to a depth of about 2.4 metres below the existing ground surface.

Groundwater conditions at the boreholes and augerhole were noted at the time of drilling. A standpipe was installed at BH1 for subsequent ground water level monitoring. The boreholes and augerhole were loosely backfilled with the auger cuttings upon completion of drilling.

The field work was supervised throughout by a member of our engineering staff who located the test holes in the field, logged the test holes and cared for the samples obtained. A description of the subsurface conditions encountered at each of the boreholes and augerhole is given in the attached Record of Borehole Sheets and Table I, Record of Augerhole, respectively. The approximate locations of the test holes are shown on the attached Site Plan, Figure 2.

SUBSURFACE CONDITIONS

General

As previously indicated, a description of the subsurface conditions encountered at the test holes is provided in the attached Record of Boreholes and Table I, Record of Augerhole sheets following the text of this report. The test hole logs indicate the subsurface conditions at the specific test 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 the test pit locations may vary from the conditions encountered at the test holes.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. 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 test hole logs. 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 test holes.

Fill

From the surface at all the test holes, fill materials ranging in thickness from about 0.5 to 1.1 metres were encountered. The fill consists of topsoil, grey brown sand, clay, asphaltic concrete and gravel. The fill materials were fully penetrated at all three test hole locations.

Topsoil

About a 0.15 to 0.3 metre thick topsoil layer was encountered beneath the fill materials at all three test hole locations at depths of about 0.50 to 1.1 below the existing ground surface. The material was classified as topsoil based on the colour and the presence of organic materials. The identification of the topsoil layer is for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustainable plant growth.

Silty Sand

Beneath the fill materials and topsoil at test hole 1, a deposit of grey brown silty sand was encountered. The silty sand had a thickness of about 0.05 metres and was fully penetrated.

Silty Clay

A deposit of grey brown to grey silty clay was encountered below the fill, topsoil and silty sand at the three test hole locations. The silty clay layers were encountered at all three test holes at depths of about 1.0, 1.2 and 0.8 metres for boreholes BH1, BH2, and augerhole AH1, respectively, below existing ground surface.

The upper about 0.8 to 2.3 metre portion of the silty clay has been weathered to a stiff to very stiff grey brown crust. Beneath the grey brown crust the silty clay becomes grey and decreases to firm to stiff in consistency. The results of in situ vane shear testing carried out in the softer grey silty clay gave undrained shear strength values ranging from about 35 to 71 kilopascals.

The dynamic cone penetration tests carried out at borehole 1 gave values of weight of hammer (WH) to 7 blows per 0.3 metres to a depth of about 13.1 metres below the existing ground surface. The dynamic cone penetration test values increased with depth below 13.1 metres and ranged from 10 to 100 blows per 0.3 metres. At a depth of some 19.3 metres below the existing ground surface refusal to cone penetration was encountered. It is considered likely that the increase in blow count at about 13.1 metres depth indicates the possible presence of glacial till materials and that refusal to cone penetration possibly indicates the upper surface of the bedrock.

Groundwater

A trace to some water seepage was observed in the test holes at the time of drilling at about 2.3 and 2.4 metres, respectively, below the existing ground surface. On August 23, 2013, water was measured in a standpipe within borehole 1 at a depth of about 3.3 metres below the existing ground surface. 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 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.

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. 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 moderate corrosion rate on buried steel.

PROPOSED CHURCH BUILDING ADDITIONS

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 and topsoil, the subsurface conditions encountered at the test holes advanced during the investigation are suitable for the support of the proposed building additions on conventional spread footing foundations. 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. The excavations for the foundations should be taken through any surficial fill, topsoil or otherwise deleterious material to expose the native, undisturbed silty clay.

For the proposed building additions, strip and pad footings, a minimum 0.5 metres in width bearing on the native undisturbed silty clay or on a suitably constructed engineered pad founded on the silty clay and at a founding depth of up to 2.0 metres below the existing ground surface and above the groundwater level, a maximum allowable bearing pressure of 120 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 240 kilopascals using ultimate limit states design. The above allowable bearing pressures are suitable for an additional grade raise fill thickness adjacent to the structure of up to 0.5 metres and a maximum footing width of 1.2 metres for strip footings and 2.0 metres for pad footings.

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 25 millimetres and 15 millimetres, respectively on an adequately prepared subgrade. It is noted that most or all of the settlement will be differential with respect to the existing building.

Any fill required to raise the footings for the proposed building additions 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 additions 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. Since the source of recycled material cannot be determined, it is suggested that any granular materials used below the founding level be composed of virgin materials only.

All exterior footings and those in any unheated parts of the proposed building additions should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, 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.

Foundation Walls

The native soils at the site are considered to be highly frost susceptible. To prevent possible foundation frost jacking, the backfill against unheated walls or isolated walls or piers should consist of the 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 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. In that case any native material proposed for foundation backfill should be inspected and approved by the geotechnical engineer.

Provided everywhere the proposed finished floor surfaces are above the exterior finished grade and provided the exterior grade is adequately sloped away from the proposed building addition, no perimeter foundation drainage system is required.

Slab on Grade Support

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 area. 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. Any fill materials consisting of granular material, removed from the proposed concrete floor slab area, should be stockpiled for possible reuse with approval from the geotechnical engineer.

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. 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 about 5 metres.

Under slab drainage is not considered necessary provided that the floor slab level is everywhere above the finished exterior ground surface level. 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.

Seismic Design for the Proposed Building Additions

Based on the limited information from the test holes, for seismic design purposes, in accordance with the 2006 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class D.

Potential for Soil Liquefaction

As indicated above, the results of the test holes indicate that the native deposits underlying the site consist of firm to stiff silty clay followed by glacial till. As firm to stiff cohesive silty clay materials are not prone to liquefaction, it is considered that no damage to the proposed building additions should occur due to liquefaction of the native subgrade under seismic conditions.

Site Services

Excavation

The excavations for the site services will be carried out through fill and topsoil and into the native silty clay. The sides of the excavations in overburden materials should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Ontario Occupational Health and Safety Act. That is, open cut excavations with overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box. Groundwater seepage into the excavations, if any, should be handled by pumping from sumps in the excavation.

Pipe Bedding and Cover Materials

It is suggested that the service pipe bedding material consist of at least 150 millimetres of granular material meeting OPSS requirements for Granular A. A provisional allowance should, however, be made for sub-excavation of any existing fill or disturbed material encountered at sub-grade level. Granular material meeting OPSS specifications for Granular B Type II could be used as a sub-bedding material. The use of clear crushed stone as bedding or sub-bedding material should not be permitted.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A or Granular B Type I (with a maximum particle size of 25 millimetres).

The sub-bedding, bedding and cover materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches.

In areas where the service trench will be located below or in close proximity to existing or future roadway areas, acceptable native materials should be used as backfill between the roadway sub-grade level and the depth of seasonal frost penetrations (i.e. 1.8 metres below finished grade) in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway. Where native backfill is used, it should match the native materials exposed on the trench walls. Some of the native materials from the lower part of the trench excavations may be wet of optimum for compaction. Depending on the weather conditions encountered during construction, some drying of materials and/or recompaction may be required. Any wet materials that cannot be compacted to the required density should either be wasted from the site or should be used outside of existing or future roadway areas. Any boulders larger than 300 millimetres in size should not be used as service trench backfill. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I.

To minimize future settlement of the backfill and achieve an acceptable sub-grade for the roadways, sidewalks, etc., the trench should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. The specified density may be reduced where the trench backfill is not located or in close proximity to existing or future roadways, driveways, sidewalks, or any other type of permanent structure.

Access Roadway and Parking Area Pavements

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 access roadway and parking lot area. 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 or OPSS Granular B Type I or 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.

For pavement areas subject to cars and light trucks the pavement should consist of:

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

For the access roadway and pavement areas carrying heavy truck traffic, the subbase thickness should be increased to 400 millimetres and the asphaltic concrete thickness increased to 80 millimetres.

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.

Effects of Trees

This site is underlain by deposits of sensitive silty clay, a material which is known to be susceptible to shrinkage with a change/reduction in moisture content. Research by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada has shown that trees can cause a reduction of moisture content in the sensitive silty clays in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations bearing on or above the silty clay. Therefore, no deciduous trees should be permitted closer to the building (or any ground supported structures which may be affected by settlement) than the ultimate height of the trees. For groups of trees or trees in rows, the separation distance should be increased to 1.5 times the ultimate height of the trees.

The effects of trees should be considered in landscaping the property.

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 and to re-evaluate the guidelines provided in the report with respect to the actual project plans.

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 additions 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 foundations should be inspected to ensure that the materials used conform to the grading and compaction specifications.

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

The native topsoil and silty clay 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.

LICENSED PROFESSIONAL ENGINEER
Aug 30, 2013
S.E. deWit
100079612
PROVINCE OF ONTARIO

Dean Tataryn, B.E.S., EP

Reviewed by Steve deWit, P.Eng.

Attachments: List of Abbreviations
Record of Boreholes
Table I, Record of Augerhole
Table II, Order of Water Demand for Common Trees
Key Plan, Figure 1
Site Plan, Figure 2
Results of Chemical Laboratory Testing

File 130476

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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 millimetre 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 drill 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 remoulded shear strength

SOIL DESCRIPTIONS

Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to 10
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 to 100
Very Stiff	over 100

LIST OF COMMON SYMBOLS

c_u undrained shear strength
e void ratio
C_c compression index
C_v coefficient of consolidation
k coefficient of permeability
I_p plasticity index
n porosity
u pore pressure
w moisture content
w_L liquid limit
w_p plastic limit
ϕ^1 effective angle of friction
γ unit weight of soil
γ^1 unit weight of submerged soil
σ normal stress

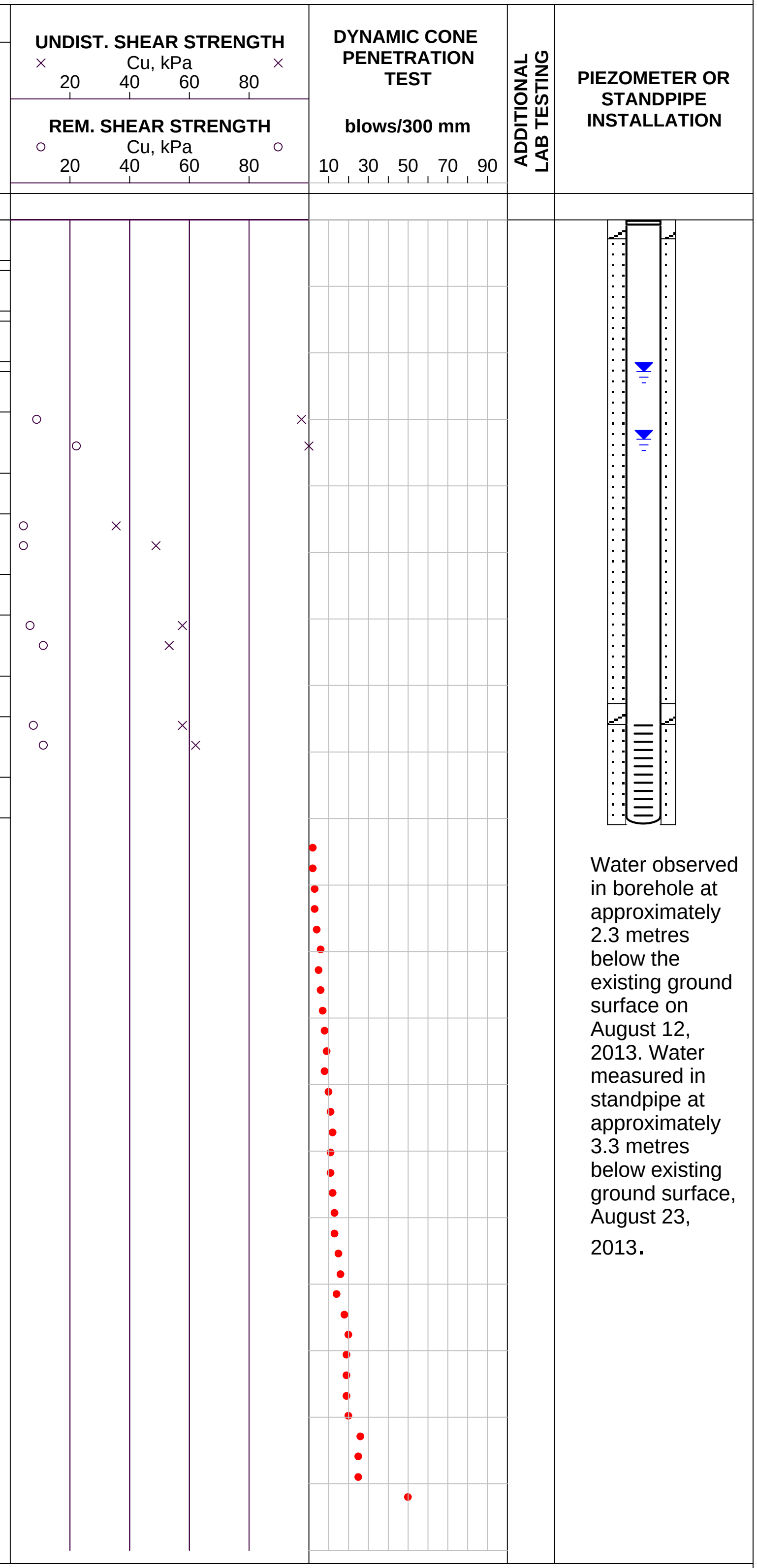
RECORD OF BOREHOLE BH1

PROJECT: Proposed Church Additions
CLIENT: St. Helen's Anglican Church
LOCATION: 1234 Prestone Drive, Ottawa
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 130476
DATE OF BORING: August 12, 2013
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (M)
	Ground Surface	
0	Topsoil, gravel, sand and clay (FILL)	0.00
0.76	TOPSOIL	0.76
	Grey brown SILTY SAND	
	Very stiff to firm red brown to grey brown SILTY CLAY	
2.29	Stiff grey SILTY CLAY	2.29
9.14	Borehole continued as Probe Hole	9.14
19.28	End of Borehole, refusal on large boulders or bedrock	19.28

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	SS	18
2	SS	16
3	SS	11
4	SS	3
5	SS	2
6	SS	1
7	SS	1
8	SS	1



Water observed in borehole at approximately 2.3 metres below the existing ground surface on August 12, 2013. Water measured in standpipe at approximately 3.3 metres below existing ground surface, August 23, 2013.

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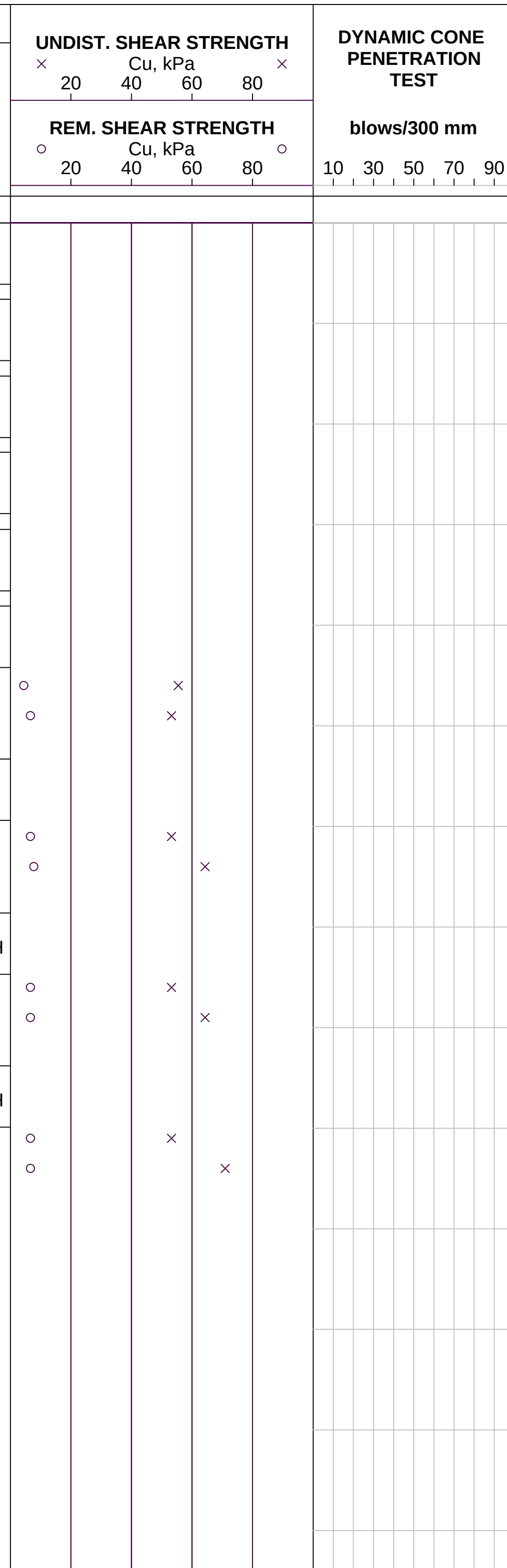
DEPTH SCALE: 1 to 75
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: DT

RECORD OF BOREHOLE BH2

PROJECT: Proposed Church Additions
CLIENT: St. Helen's Anglican Church
LOCATION: 1234 Prestone Drive, Ottawa
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 130476
DATE OF BORING: August 12, 2013
SHEET 1 of 1
DATUM:

SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (M)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m
Ground Surface				
Topsoil, sand and clay (FILL)	0.00	1	SS	10
TOPSOIL	1.06	2	SS	20
Very stiff red brown to grey brown SILTY CLAY, trace of sand layers		3	SS	11
Firm to stiff grey SILTY CLAY	2.44	4	SS	8
		4	SS	5
		5	SS	2
		6	SS	1
		7	SS	WH
		8	SS	WH
End of Borehole in SILTY CLAY	9.75			



ADDITIONAL LAB TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Water observed in borehole at approximately 2.4 metres below the existing ground surface on August 12,

Kollaard Associates Engineers

DEPTH SCALE: 1 to 75
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

TABLE I

RECORD OF AUGERHOLE
ST. HELEN'S ANGLICAN CHURCH
1234 PRESTONE DRIVE
OTTAWA, ONTARIO

AUGERHOLE NUMBER	DEPTH (METRES)	DESCRIPTION
AH1		
	0.00 – 0.13	Topsoil (FILL)
	0.13 – 0.30	Grey brown silty clay, trace of sand (FILL)
	0.30 – 0.36	Asphaltic Concrete (FILL)
	0.36 – 0.51	Grey crushed stone (FILL)
	0.51 – 0.76	TOPSOIL
	0.76 – 2.44	Very stiff red brown to grey brown SILTY CLAY
	2.44	End of test pit

Test pit dry, August 12, 2013.

TABLE II

ORDER OF WATER DEMAND FOR COMMON TREES

Some common trees in decreasing order of water demand:

Broad Leaved Deciduous

Poplar
Alder
Aspen
Willow
Elm
Maple
Birch
Ash
Beech
Oak

Deciduous Conifer

Larch

Evergreen Conifers

Spruce
Fir
Pine

KEY PLAN

FIGURE 1

NOT TO SCALE

Kollaard Associates
Engineers

Project No. 130476

Date August 2013

DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

BH1 APPROXIMATE BOREHOLE LOCATION

AH1 APPROXIMATE AUGERHOLE 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
Engineers
PO, BOX 189, 210 PRESCOTT ST
KEMPTVILLE ONTARIO
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(613) 860-0923
info@kollaard.ca

CLIENT:
VANDENBERG & WILDEBOER ARCHITECTS

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED ADDITIONS TO EXISTING CHURCH

LOCATION:
ST. HELEN'S ANGLICAN CHURCH
1234 PRESTONE DRIVE
CUMBERLAND WARD
CITY OF OTTAWA, ONTARIO

DESIGNED BY: --	*DATE:* AUGUST 16, 2013
DRAWN BY: DT	*SCALE:* N.T.S

KOLLAARD FILE NUMBER:
130476

Laboratory Test Results for Sulphate, Resistivity and pH

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:	1317653
Date Submitted:	2013-08-15
Date Reported:	2013-08-21
Project:	130362
COC #:	167371

Group	Analyte	MRL	Units	Guideline	Lab I.D.: 1050543 Sample Matrix: Soil Sample Type: Sampling Date: 2013-08-13 Sample I.D.: BH1 - 5'-7'
Agri. - Soil	Electrical Conductivity	0.05	mS/cm		0.43
	pH	2.0			7.3
General Chemistry	Cl	0.002	%		0.017
	Resistivity	1	ohm-cm		2330
	SO4	0.01	%		<0.01

Guideline = *** = Guideline Exceedence**

** = Analysis completed at 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