MOREY ASSOCIATES LTD.
CONSULTING ENGINEERS

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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
HEMPHILL STREET, RICHMOND
OTTAWA, ONTARIO

Submitted to:

Schouten Construction Ltd.
2740 Harbison Road
Richmond, Ontario
K0A 2Z0

Attention: Mr. Adrian Schouten

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January 2018 017600

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for a proposed residential development located west of Shea Road on the north side of Hemphill Street, within Part of Lot 25, Concession 4, in the Geographic Township of Goulbourn, Richmond, Ottawa, Ontario (see Key Plan, Figure 1 and Aerial Photograph, Figure 2).

The purpose of the investigation was to determine the general subsurface conditions at the site by means of a limited number of test pits, and based on an interpretation of the factual information obtained to provide engineering guidelines on the geotechnical aspects of the design of the project, including construction considerations, which could influence design decisions.

This report has been prepared in consideration of the terms and conditions noted in the report and with the assumption that the design of the project will satisfy any applicable codes and standards. Should there be any changes in the design features outlined below, which may relate to the geotechnical considerations, Morey Associates Ltd. should be advised in order to review the report recommendations.

2.0 BACKGROUND INFORMATION AND SITE GEOLOGY

For discussion purposes Hemphill Street is considered to exist at the south side of the site (see Key Plan, Figure 1).

The site consists of a rectangular ‘L’ shaped parcel of land some 0.8 hectares in plan area located west of Shea Road on the north side of Hemphill Street, Richmond, Ottawa, Ontario (see Key Plan, Figure 1).

It is understood that plans are being prepared to construct a residential development at the site consisting of some seven single family dwellings serviced by municipal sanitary sewer and private wells. It is further understood that the proposed development will be accessed by the existing Hemphill Street and a proposed cul-de-sac located just west of the intersection of Hemphill Street and Gamble Drive. Surface drainage for the proposed development will likely be by means of swales, ditches, catch basins and storm sewers. The proposed residential buildings are likely to be of wood frame construction with conventional spread footing foundations.

The site is bordered on the north and west by vacant agricultural fields, on the south by Hemphill Street with existing residential development beyond, and on the east by an existing single family dwelling with Shea Road and vacant agricultural fields beyond. A tributary to Jock River exists some 130 metres east of the subject site. The ground cover at the site consists of cultivated lands with some young to mature trees along the south property boundary.

A review of the surficial geology map for the site area indicates that the site is underlain by marine deposited clay, silt, and silty clay. The bedrock geology map indicates that the bedrock underlying the site consists of dolomite of the Oxford formation.

3.0 PROCEDURE

The field work for this investigation was carried out on November 22, 2017 at which time 4 test pits, numbered TP1 to TP4, were put down at the site. The test pits were advanced to depths of some 3.4 to 4.0 metres below the existing ground surface using a track mounted backhoe supplied and operated by the client.

The subsurface soil conditions at the test pits were identified based on visual and tactile examination of the materials exposed on the walls and bottom of the test pits and the results of in situ vane shear tests carried out in the cohesive materials encountered. Groundwater conditions in the test pits were noted at the time of excavating. Standpipes were installed at each test pit for subsequent groundwater level measurements. The test pits were loosely backfilled with the excavated material upon completion.

The field work was supervised throughout by a member of our engineering staff who located the test pits in the field, logged the subsurface conditions encountered at the test pits and cared for the samples obtained. A description of the subsurface conditions encountered at each of the test pits is given in the attached Record of Test Pit sheets. The approximate locations of the test pits are shown on the attached Site Plan, Figure 3.

The ground surface elevations at the test pits were measured by members of our engineering staff relative to a site benchmark described as a nail in the utility pole located at the southwest corner of the Hemphill Street and Shea Road intersection (see Site Plan, Figure 3). The elevation of that site benchmark is provided on the H.A.Ken Shipman Surveying Ltd., drawing reference No. GLB.-478, for file No. 16-10896D, assumed Geodetic datum.

A sample of soil obtained from the site was delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack and soil corrosion on buried concrete and steel. The results of that testing are provided in the attached Appendix A.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, a description of the subsurface conditions encountered at the test pits is provided in the attached Record of Test Pit sheets. The test pit 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 other locations than the test pit locations may vary from the conditions encountered at the test pits.

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 Morey Associates Ltd. 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 date of the observations noted in the report, and on the test pit logs. Groundwater conditions may vary seasonally, or may be affected by construction activities, agricultural tile draining activities on or in the vicinity of the site.

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

4.2 Topsoil

About a 0.3 metre thickness of topsoil was encountered from the surface at all of the test pits. The material was classified as topsoil based on colour and the presence of organic materials and is intended as identification for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustaining plant growth.

4.3 Silty Clay

Beneath the topsoil at all of the test pits a deposit of silty clay was encountered. All the test pits were terminated in the silty clay material at depths of some 3.4 to 4.0 metres below the existing ground surface.

The results of in situ vane shear strength testing carried out in the silty clay material encountered at the test pits gave undrained shear strength values ranging from about 62 to greater than 130 kilopascals, indicating that the silty clay material has been weathered to a very stiff to stiff crust to depths of at least 3.4 to 4.0 metres below the existing ground surface.

4.4 Groundwater

Groundwater seepage/groundwater flow into the test pits was observed at about 1.8 to 2.3 metres below the existing ground surface, or between about elevations 91.4 to 91.9 metres, upon the completion of excavating on November 22, 2017.

The groundwater level was measured on December 6, 2017 within the test pit standpipes at depths of some 1.1 to 1.2 metres below the existing ground surface, or between about elevations 92.4 to

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

5.0 PROPOSED RESIDENTIAL DEVELOPMENT

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 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 and have not been addressed.

5.2 Foundations for Proposed Residential Buildings

With the exception of the topsoil, the subsurface conditions encountered at the test pits advanced for this investigation are suitable for support of the proposed residential buildings on spread footing foundations. The excavations for the foundations should be taken down through the topsoil and any deleterious material to expose the native undisturbed very stiff grey brown silty clay.

Based on the results of this investigation strip and pad footings up to 0.9 metres wide and 2.0 metres square, respectively, bearing on the native undisturbed very stiff grey brown silty clay no lower than 1.0 metre below the existing ground surface and above the groundwater level may be designed using a maximum allowable bearing pressure of 95 kilopascals for serviceability limit states (SLS) and a bearing resistance for ultimate limit state (ULS) of 140 kilopascals.

The above allowable bearing pressure and bearing resistance for footings founded on the native undisturbed very stiff grey brown silty clay are suitable for a maximum 2.0 metre landscape grade raise adjacent to the proposed building foundations.

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, founded on the native undisturbed silty clay, should be less than 25 millimetres and 20 millimetres, respectively.

If grade raises adjacent to the proposed building foundations are greater than 2.0 metres or the footings are wider or founded lower than given above, the allowable bearing pressure for footings may have to be reduced. Alternatively, the use of light weight fill material, such as expanded polystyrene (EPS), could be used in conjunction with suitable native or imported backfill material (meeting the backfilling requirements outlined in this report) in order to meet the proposed finished grade level.

If any fill is required to raise the subgrade for the proposed buildings to proposed founding level, the fill should consist of imported granular material (engineered fill). The engineered fill should consist of material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular A or Granular B Type II and should be compacted in maximum 250 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 a minimum 0.3 metres horizontally out beyond the edge of the footings and then down and out from the top edge of the engineered fill at 1 horizontal to 1 vertical, or flatter. The excavations for the proposed buildings 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.

Groundwater inflow from the native soils into the basement excavations during construction, if any should be handled by pumping from suitably filtered sumps within the excavation. A suitable filter for a sump may consist of a geotextile such as Terrafix 270R or approved equivalent.

5.3 Foundation Walls

All exterior foundation elements and those in any unheated parts of the proposed buildings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated foundation elements adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum 1.8 metres of earth cover. Where less than the required depth of soil cover can be provided, the foundation elements could be protected from frost by using a combination of earth cover and extruded polystyrene rigid insulation. A typical footing frost protection insulation detail could be provided, if required.

Most of the native soils at the site may be frost susceptible. As such, to prevent possible foundation frost jacking of unheated walls and/or isolated walls and to provide drainage for the basement walls, backfill material 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 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 maximum dry density value.

For the proposed buildings a conventional, perforated perimeter drain should be provided at founding level, leading by gravity flow to a sump complete with sump pump or to a positive gravity drainage outlet. The drain should be provided with a 150 millimetre thick surround of 20 millimetre crushed stone. A suitable geotextile, such as Terrafix 270R or equivalent, should be provided between the crushed stone and any sandy backfill materials.

5.4 Seismic Design for Proposed Residential Buildings

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. That site class has been determined based on the limited investigation carried out at the site. A higher site class designation may result from additional investigation and/or in situ seismic velocity testing of the subsurface materials.

5.5 Potential for Soil Liquefaction

As indicated above the results of the test pits indicate that the native deposits underlying the site below proposed footing founding level consist of very stiff silty clay. This material is not prone to liquefaction, it is considered that no damage to the proposed residential buildings should occur due to liquefaction of the native subgrade for the expected seismic conditions in the Ottawa region.

5.6 Potential for Sulphate Attack/Corrosion on Buried Concrete and Steel

Laboratory testing carried out on a sample of soil obtained from test pit 4 gave a percent sulphate (SO_4) for the soil sample of 0.01 and an ohm-cm resistivity and pH of 6,250 and 7.1, respectively.

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 soil sample is considered to have a negligible potential for sulphate attack on buried concrete.

The resistivity and pH values for the soil sample tested indicates the soils sample has an underground corrosion rate of about 0.50 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 slightly aggressive corrosion rate on buried bare steel. The slightly aggressive corrosion rate on buried bare steel (ie: exposed ferrous metal) should be considered in the design of substructures and in the selection of pipe materials.

6.0 SITE SERVICES

6.1 Excavation

The excavations for the site services will be carried out through topsoil and 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. It is possible that some of the services excavations will extend below the water table in the silty clay. Where this occurs, some loss of ground and groundwater inflow may occur, requiring side slopes as flat as 3 horizontal to 1 vertical to be used.

Groundwater seepage into the excavations, if any, should be handled by pumping from suitably filtered sumps in the excavation. A suitable filter for a sump may consist of a geotextile such as Terrafix 270R or approved equivalent.

6.2 Pipe Bedding and Cover Material

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 subexcavation of any existing fill or disturbed material encountered at subgrade 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 a bedding, sub-bedding or cover material since fine particles from the any sandy backfill materials could potentially migrate into the voids in the clear crushed stone and cause loss of pipe support.

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.

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.

6.3 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 subgrade 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. 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 subgrade for roadways, parking areas, sidewalks, etc., the trench backfill should be compacted in maximum 250 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 within or in close proximity to existing or future roadways, driveways, sidewalks, or any other type of permanent structure.

7.0 PROPOSED HEMPHILL STREET EXTENSION ROADWAY PAVEMENT

In preparation for construction of the proposed roadway pavement for the extension of Hemphill Street associated with the proposed residential development, any topsoil and any soft, wet or deleterious materials should be removed from the proposed roadway. 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 proposed roadway granulars.

Following approval of the preparation of the subgrade, the pavement granulars may be placed. For areas of the site that require the subgrade to be raised to the proposed roadway subgrade level, it is considered that some of the drier native materials could be used for this purpose or the material could consist of OPSS select subgrade material or OPSS Granular B Type I or Type II. Any materials proposed for this use should be approved by the geotechnical engineer before placement within the proposed roadway. Materials used for raising the subgrade to proposed roadway 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 the proposed roadway extension the pavement should consist of:

40 millimetres of hot mix asphaltic concrete (HL3 or Superpave12.5) over
40 millimetres of hot mix asphaltic concrete (HL8 or Superpave19) over
150 millimetres of OPSS Granular A base over
350 millimetres of OPSS Granular B, Type II subbase

At the location(s) where the proposed roadway pavement will butt against the existing roadway pavement, the roadway subbase for the proposed roadway should be sloped up or down, as required, at 5 horizontal to 1 vertical such that the level of the underside of the proposed roadway subbase matches the level of the underside of the existing roadway subbase.

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 asphaltic concrete should be compacted in acceptance with Table 10 of OPSS 310.

The above pavement structure will be adequate on an acceptable subgrade, that is, one where any service trench backfill has been adequately compacted. If the proposed 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 proposed roadway subgrade surface and the granular subbase material.

The design of asphaltic concrete is the responsibility of the asphaltic concrete designer and/or supplier employed by the owner and is outside of the scope of this report. The above pavement design has been determined, from a geotechnical engineering point of view, to ensure that the combined strength of the pavement components (asphaltic concrete and granulars) is adequate to protect the subgrade from the expected repeated vehicle loading. The above indicated asphaltic concrete and granular thicknesses may need to be increased to meet City of Ottawa requirements.

The design life for a flexible pavement as described above is generally taken as 11 years to first major maintenance.

8.0 CONSTRUCTION CONSIDERATIONS

The geotechnical engineer should be retained to review the final grading plans for the proposed development to ensure that the proposed landscape grade raises adjacent to the proposed buildings are suitable in view of the subsurface conditions encountered at the site.

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 footing areas and any engineered fill areas for the proposed dwellings should be inspected by Morey Associates Ltd. 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 and in situ compaction testing should be carried out to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for the pavement areas and site services should be inspected and approved by Morey Associates Ltd. In situ compaction testing should be carried out on the pavement granular and service pipe bedding and backfill materials to ensure the materials meet the specifications from a compaction point of view.

The native soils at this site will be sensitive to disturbance and softening 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.

Any native material proposed to be used as earth fill below the pavement areas should be approved by Morey Associates Ltd. prior to use.

Any loose material from the backfilling of test holes encountered during construction at the site may require subexcavation, re-backfilling and suitable compaction. The geotechnical engineer should inspect and approve the re-backfilling and compaction at any encountered test holes.

9.0 REPORT CONDITIONS AND LIMITATIONS

It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document is neither intended nor authorized by Morey Associates Ltd. 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 engineering guidelines provided in this report are based on subsurface data obtained at the specific test locations only. Boundaries between zones on the logs are often not distinct but transitional and were interpreted. Experience indicates that the subsurface soil and groundwater conditions can vary significantly between and beyond the test locations. For this reason, the engineering guidelines given in this report are subject to a field verification of the subsurface soil conditions at the time of construction.

The report recommendations are applicable only to the project described in the report. Any changes in the scope of the project will require a review by Morey Associates Ltd., to ensure compatibility with the engineering guidelines contained in this report.

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 service to you, please do not hesitate to contact our office.

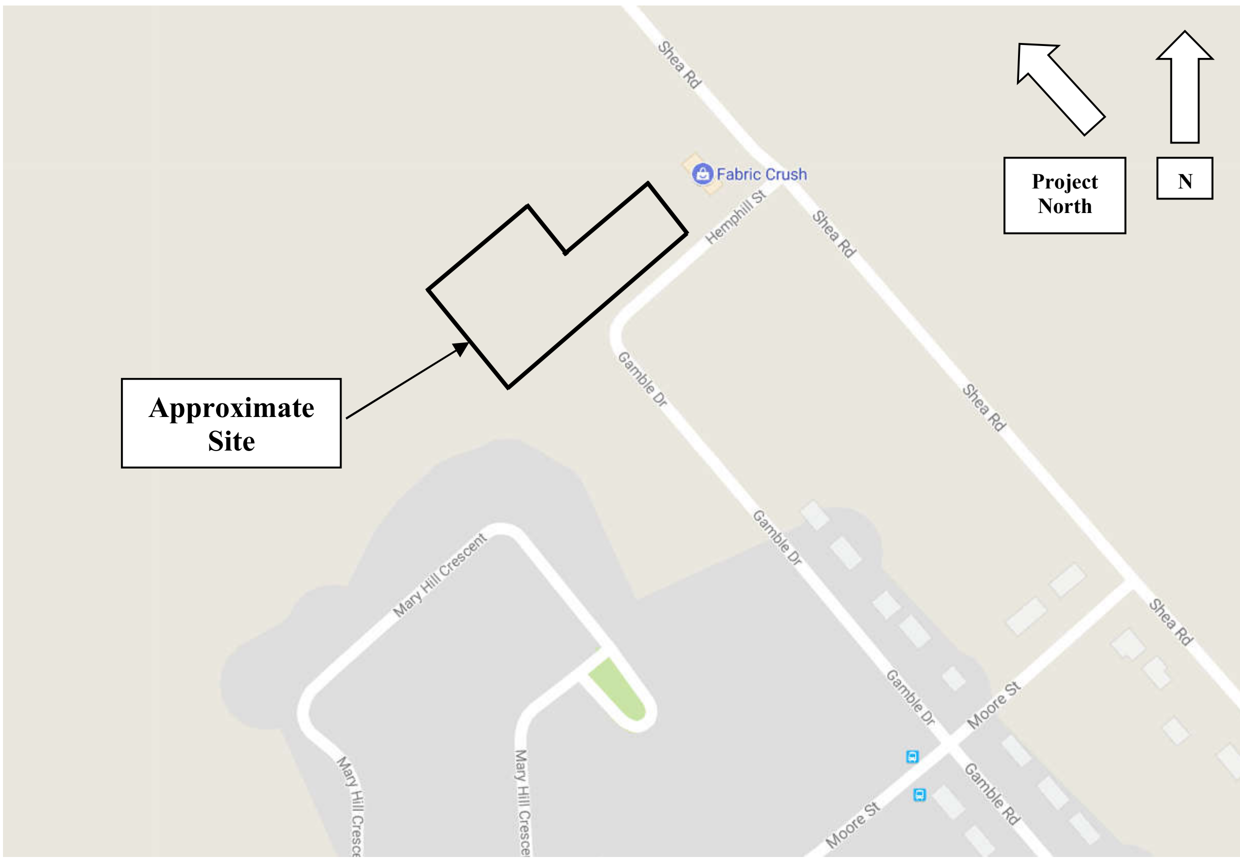
Yours truly,
Morey Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
D. G. MOREY
100208030
Jan. 19/18
PROVINCE OF ONTARIO

D. G. Morey, P. Eng.
Owner/Civil Engineer

C. R. Morey, M. Sc. (Eng.), P. Eng.
Senior Consulting Engineer

KEY PLAN	FIGURE 1



0
250m

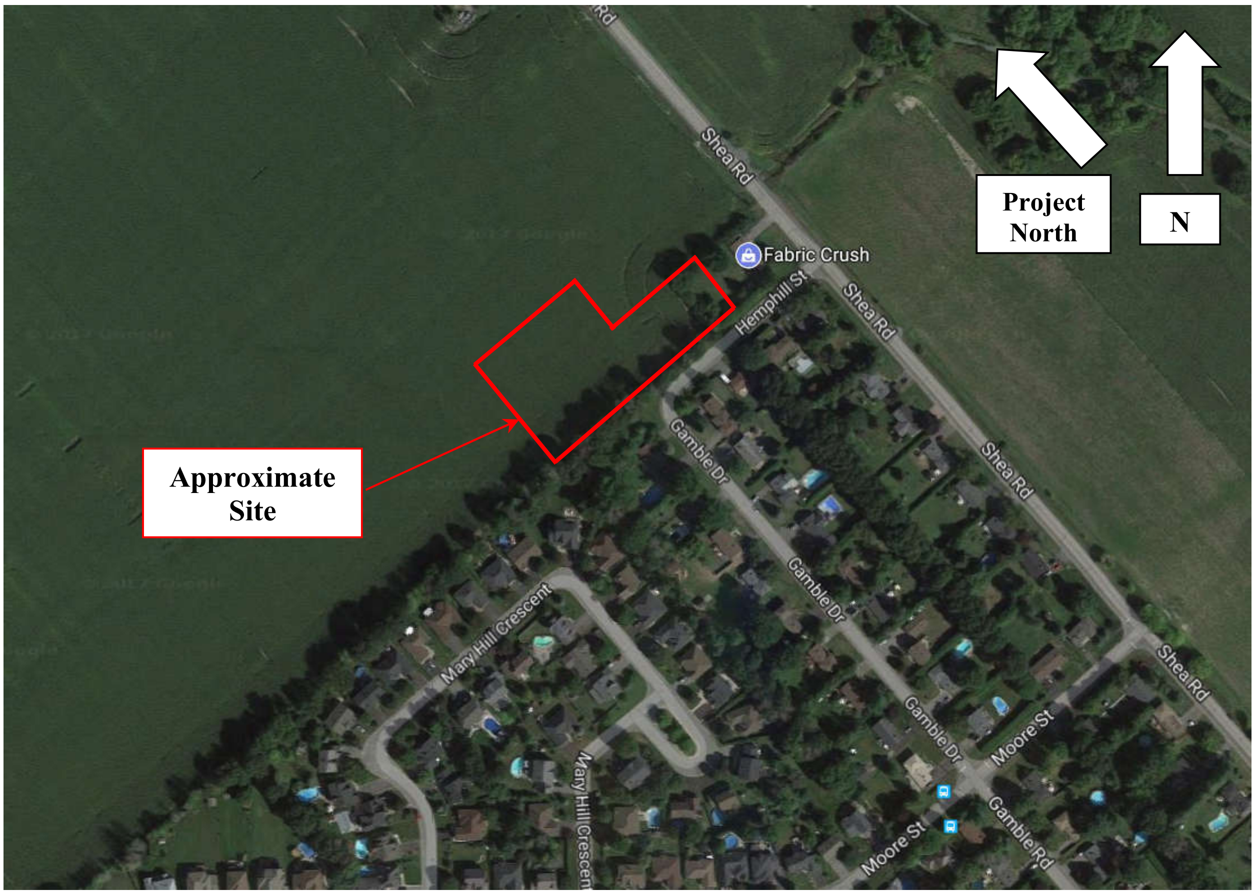
APPROXIMATE SCALE

MOREY ASSOCIATES LTD.
CONSULTING ENGINEERS

Project No. 017600
Date January 2018

AERIAL PHOTOGRAPH

FIGURE 2

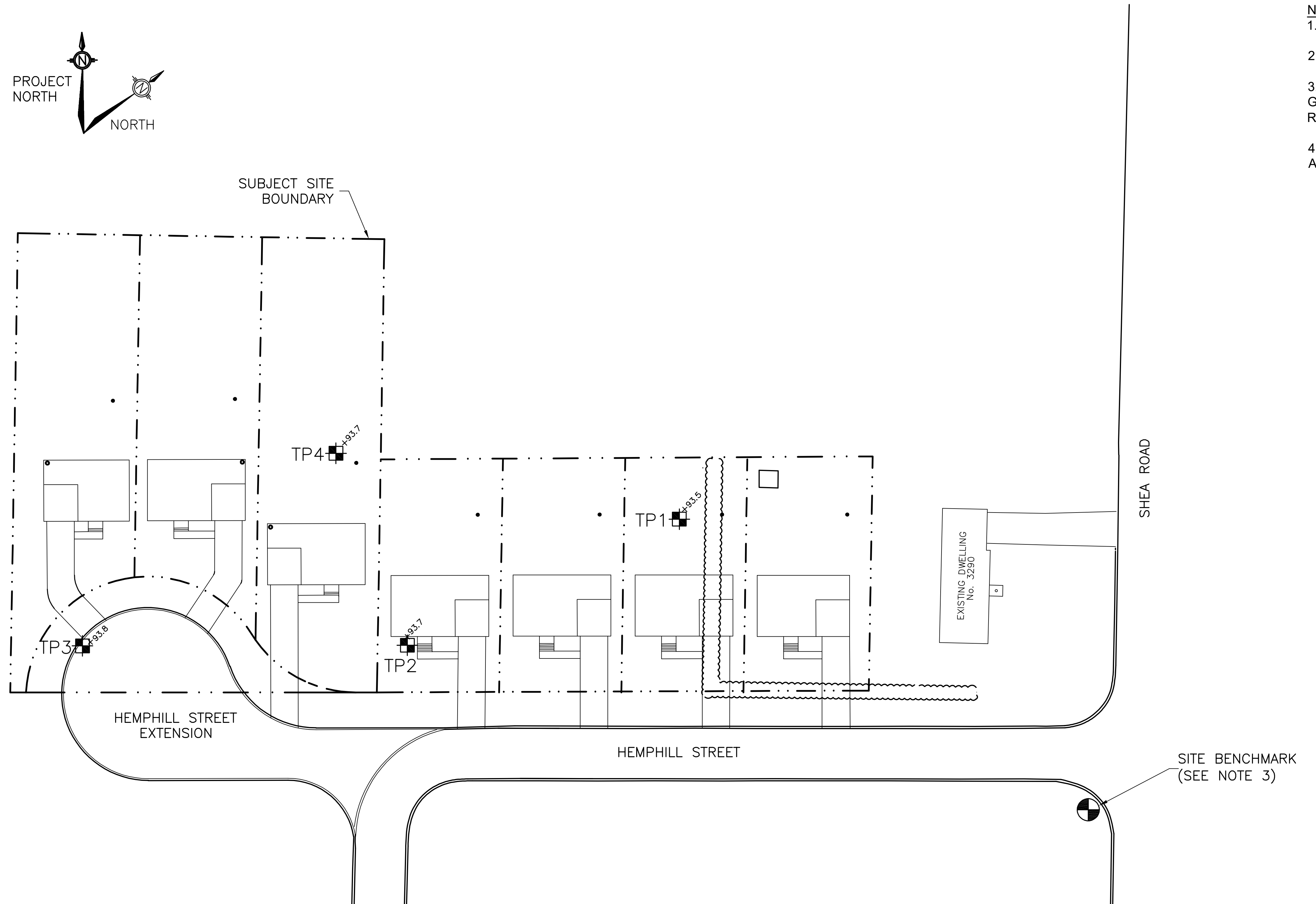


NOT TO SCALE

MOREY ASSOCIATES LTD.
CONSULTING ENGINEERS

Project No. 017600

Date January 2018



NOTES:
1. All dimensions are in metres. Do not scale drawing.

2. This drawing is to be read in conjunction with the accompanying report.

3. Site benchmark = Existing nail in utility pole, located as shown on drawing. Geodetic elevation 94.21m as per H.A.Ken Shipman Surveying Ltd. drawing Ref. No. GLB.-478 for File No. 16-10896D.

4. Any changes made to this plan must be verified and approved by Morey Associates Ltd.

LEGEND

TP1 APPROXIMATE MOREY ASSOCIATES LTD. TEST PIT

+99.9 APPROXIMATE EXISTING SPOT ELEVATION

REFERENCE:
Base plan information referenced from D. B. Gray Engineering Inc., Site Servicing Plan, Drawing C-1, dated Feb-XX-12 for Job No. 17037, Proposed 7-lot Residential Development Hemphill Street Richmond Ontario, Revision 01 dated Nov 29-17.

DRAWING
SITE PLAN, FIGURE 3

LOCATION
HEMPHILL STREET, RICHMOND
OTTAWA, ONTARIO

PROJECT
PROPOSED RESIDENTIAL DEVELOPMENT
GEOTECHNICAL INVESTIGATION

CLIENT
SCHOUTEN CONSTRUCTION LTD.

DATE	DRAWING No.	DRAWN BY	APPROX. SCALE	FILE NO.
January 2018	1 of 1	DJV	1:750	017600

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TABLE I

RECORD OF TEST PITS
PROPOSED RESIDENTIAL DEVELOPMENT
HEMPHILL STREET
OTTAWA, ONTARIO

TEST PIT NUMBER/ ELEVATION	DEPTH (METRES)	DESCRIPTION
TP1 ELEV. 93.5m	0.00 – 0.30	TOPSOIL
	0.30 – 3.35	Stiff to very stiff, grey brown SILTY CLAY
	3.35	End of test pit

Depth (m)	Strength, Cu (kPa)
0.3	>130
0.6	>130
0.9	>130
1.2	>130
1.5	>130
1.8	>130
2.1	120
2.4	105
2.7	89
3.0	65
3.3	62

Groundwater flow observed in test pit at about 2.3 metres below existing ground surface, November 22, 2017.
Water measured in standpipe at about 1.1 metres below existing ground surface, December 6, 2017.

TABLE I (CONTINUED)

TEST PIT NUMBER/ ELEVATION	DEPTH (METRES)	DESCRIPTION
TP2 ELEV. 93.7m	0.00 – 0.30	TOPSOIL
	0.30 – 4.00	Stiff to very stiff, grey brown SILTY CLAY
	4.00	End of test pit

Depth (m)	Strength, Cu (kPa)
0.3	>130
0.6	>130
0.9	>130
1.2	>130
1.5	>130
1.8	>130
2.1	120
2.4	110
2.7	90
3.0	71
3.3	67
3.6	78
3.9	84

Groundwater seepage observed in test pit at about 1.8 metres below existing ground surface, November 22, 2017. Water measured in standpipe at about 1.2 metres below existing ground surface, December 6, 2017.

TABLE I (CONTINUED)

TEST PIT NUMBER/ ELEVATION	DEPTH (METRES)	DESCRIPTION
TP3 ELEV. 93.8m	0.00 – 0.30	TOPSOIL
	0.30 – 4.00	Stiff to very stiff, grey brown SILTY CLAY
	4.00	End of test pit

Depth (m)	Strength, Cu (kPa)
0.3	>130
0.6	>130
0.9	>130
1.2	>130
1.5	>130
1.8	110
2.1	95
2.4	83
2.7	67
3.0	72
3.3	80
3.6	92

Groundwater flow observed in test pit at about 2.1 metres below existing ground surface, November 22, 2017.
Water measured in standpipe at about 1.2 metres below existing ground surface, December 6, 2017.

TABLE I (CONTINUED)

TEST PIT NUMBER/ ELEVATION	DEPTH (METRES)	DESCRIPTION
TP4 ELEV. 93.7m	0.00 – 0.30	TOPSOIL
	0.30 – 4.00	Stiff to very stiff, grey brown SILTY CLAY
	4.00	End of test pit

Depth (m)	Strength, Cu (kPa)
0.3	>130
0.6	>130
0.9	>130
1.2	>130
1.5	>130
1.8	125
2.1	100
2.4	96
2.7	90
3.0	74
3.3	80
3.6	96

Groundwater flow observed in test pit at about 2.1 metres below existing ground surface, November 22, 2017.
Water measured in standpipe at about 1.2 metres below existing ground surface, December 6, 2017.

APPENDIX A

SOIL CORROSIVITY LABORATORY TESTING

eurofins | Environment Testing

Certificate of Analysis

Client:	Morey Associates 2280 Abbott Road Kemptville, ON K0G 1J0	Report Number:	1723708
Attention:	Mr. Randy Morey	Date Submitted:	2017-12-07
PO#:	120481	Date Reported:	2017-12-13
Invoice to:	Morey Associates	Project:	017600
		COC #:	192914

Dear Randy Morey:

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:

Addrine Thomas 2017.12.13 13:52:30 -05'00'

Addrine Thomas, Inorganics Supervisor

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

Eurofins 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.

Eurofins(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.

Eurofins(Mississauga) is accredited for specific parameters by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 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. Eurofins recommends consulting the official provincial or federal guideline as required.

eurofins Environment Testing

Certificate of Analysis

Client:	Morey Associates 2280 Abbott Road Kemptville, ON K0G 1J0	Report Number:	1723708
Attention:	Mr. Randy Morey	Date Submitted:	2017-12-07
PO#:	120481	Date Reported:	2017-12-13
Invoice to:	Morey Associates	Project:	017600
		COC #:	192914

Group	Analyte	MRL	Units	Guideline	Lab I.D. 1336814 / Sample Matrix Soil / Sample Type / Sampling Date 2017-12-06 / Sample I.D. TP5 1.20m
Agri. - Soil	pH	2.0			7.1
	SO4	0.01	%		0.01
General Chemistry	Cl	0.002	%		<0.002
	Electrical Conductivity	0.05	mS/cm		0.16
	Resistivity	1	ohm-cm		6250

Guideline = *** = Guideline Exceedence**

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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

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QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 338065 **Analysis/Extraction Date** 2017-12-08 **Analyst** C F			
Method Ag Soil			
pH	4.8	100	90-110
SO4	<0.01 %	100	70-130
Method Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	85-115
Method Resistivity - soil			
Resistivity			
Run No 338116 **Analysis/Extraction Date** 2017-12-08 **Analyst** C F			
Method C CSA A23.2-4B			
Chloride		100	90-110

Guideline = *** = Guideline Exceedence**

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