Geotechnical Investigation Report
Proposed Residential Development

689 Churchill Ave North, Ottawa, ON

Project No.: 121621808

Prepared for:
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April 30, 2018

Table of Contents

LIST OF TABLES

LIST OF APPENDICES

1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) has been retained by TC United Group to carry out a geotechnical investigation for a proposed residential development located on the property at 689 Churchill Avenue North in Ottawa, Ontario as shown on the Key Plan (Drawing No. 1).

The geotechnical investigation was completed in order to determine the subsurface conditions at the site and to provide recommendations on the geotechnical design aspects of the project.

This report presents the results of the field investigation program and laboratory testing, as well as geotechnical design recommendations. Limitations associated with this report and its contents are provided in the Statement of General Conditions included in Appendix A.

2.0 SITE AND PROJECT DESCRIPTIONS

2.1 SITE AND PROJECT DESCRIPTIONS

The property currently contains a single, two-storey residential building with a partial basement located on an approximate 525 m^2 lot. In addition to the building, the lot contains three sheds, landscaped areas, a paved driveway and several trees. Based on a topographical plan of the site prepared by Stantec dated April 17, 2018, the ground surface within the site is generally flat with existing ground surface elevations varying between about 77.7 m and 78.3 m.

Stantec understands that it is planned to redevelop the subject property with a new residential building. The proposed development will consist of a 3-storey building with the floor slab of the lowest level being slightly lower than final site grades. Each floor of the building will encompass a plan area of approximately 180 m^2. The Final Floor Elevations (FFEs) (tops of slab) are understood to be 77.1 m for the below-grade/basement level and 80.36 m for the first floor. A small retaining wall (i.e. supporting a retained soil height of approximately 1 m) is required adjacent to a stairwell on the north side of the building immediately adjacent to the north property line.

2.2 GEOLOGY

Available geological maps indicate that the subsurface conditions at the site consist of clay and/or till deposits over limestone bedrock. The depth to bedrock is expected to be within about 3 m below ground surface.

3.0 INVESTIGATION METHODS

3.1 BOREHOLE INVESTIGATION

Prior to commencing the field investigation, Stantec arranged for utility clearances to be completed by a private utility locating contractor, USL-1.

A geotechnical field investigation consisting of advancing two (2) boreholes, designated as BH18-1 and BH18-2, was carried out on April 6, 2018. The approximate borehole locations are shown on the Borehole Location Plan (Drawing No. 2) in Appendix B.

The boreholes were drilled using a track-mounted drill rig equipped with 200 mm diameter, hollow-stem augers and rock coring capabilities that was supplied and operated by George Downing Estate Drilling Ltd. Boreholes were advanced in southern portion of the site; other areas of the site could not be accessed due to the existing building and attached deck, sheds, trees and/or powerlines.

The subsurface stratigraphy encountered in each borehole was recorded in the field by Stantec field personnel. Soil samples were recovered at regular intervals using a 50-mm (outside diameter) split-tube sampler by conducting Standard Penetration Tests (SPTs) in accordance with the procedures outlined in ASTM specification D1586.

Both boreholes encountered effective refusal to auger drilling on bedrock at a depth of about 1.3 m below ground surface. Coring was carried out in BH18-2 to confirm the type and engineering characteristics of the bedrock.

A standpipe piezometer was installed in BH17-2 to facilitate the measurement of the groundwater level at the site. BH17-1 was backfilled with drill cuttings mixed with bentonite.

All soil samples recovered from the boreholes were placed in moisture-proof bags. Soil and bedrock samples collected during the investigation were returned to Stantec's Ottawa laboratory for detailed classification and testing.

Borehole location information is presented on the Borehole Records in Appendix C and summarized in Table 3.1 below.

Table 3.1: Summary of Borehole Details

Borehole No.	Approximate UTM Coordinates (Zone 18T)		Approximate Ground Elevation (m)	Total Depth Drilled (m)
	Northing	Easting		
BH18-1	5025726	441449	77.9	1.3
BH18-2	5025726	441458	78	5.1

3.2 LABORATORY TESTING

The following geotechnical laboratory testing was performed on selected samples:

- Moisture contents;
- Grain size distribution/hydrometer analyses on two soil samples; and
- Unconfined compressive strength tests on two bedrock samples.

Chemical analyses related to parameters associated with the potential for corrosion or sulphate attack (i.e. pH, resistivity, and chloride and sulphate content) were completed on one sample by Paracel Laboratories Inc.

The results of the laboratory tests are discussed in the text of this report and are provided on the Borehole Records and Bedrock Core Log in Appendix C. The results of the grain size distribution tests are also included in Appendix D.

Samples remaining after testing will be stored for a period of three (3) months after issuance of the final report. Samples will then be discarded after this period unless otherwise directed.

4.0 SUBSURFACE CONDITIONS

4.1 GENERAL

Detailed descriptions of the subsurface soil and bedrock conditions are presented on the Borehole Records, Bedrock Core Log, and Rock Core Photographs provided in Appendix C. Documents providing explanations of the symbols and terms used on the borehole records are also provided in Appendix C. Laboratory test results are shown on the borehole records as well as Figure D1 in Appendix D.

The stratigraphic boundaries on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil and bedrock types rather than exact boundaries between geological units. The borehole records depict conditions at the particular locations and at the particular times indicated. The subsurface soil, bedrock and groundwater conditions will vary between boreholes and/or at locations away from the boreholes.

The information provided in the following sections is intended to summarize the conditions encountered; however, the borehole records provided in Appendix C should be used as the primary source of the subsurface information for the site.

In general, the subsurface stratigraphy encountered at the borehole locations consists of surficial fill materials including asphalt overlying limestone bedrock at a depth of 1.3 m below ground surface. A summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2 ASPHALTIC CONCRETE

A surficial asphaltic concrete layer was encountered at ground surface in both BH18-1 and BH18-2. The thickness of the asphalt varied between 55 mm and 70 mm at the borehole locations.

4.3 FILL

Fill materials comprised of silty sand with gravel to clayey sand with gravel were encountered beneath the asphaltic concrete layer in BH18-1 and BH18-2. The fill materials extended to a depth of about 1.3 m below ground surface in both boreholes.

An approximately 100 mm thick layer of buried topsoil was encountered at a depth of 0.6 m below ground surface in BH18-1. The topsoil was underlain by a layer of gravel and/or cobbles that extended to a depth of 0.8 m. Brick pieces were noted within the fill in BH18-2.

Standard Penetration Test (SPT) penetration resistances of 7 and 8 per 0.3 m of penetration were measured within the upper zone of the fill to a depth of 0.6 m indicating that these materials are in a loose state. SPT penetration resistances of 50 blows per 0.1 m of penetration and 50 blows per 0.08 m of penetration were measured within the lower zone of granular fill indicating that these materials are in a very dense state.

Laboratory testing conducted on samples of the fill measured natural moisture contents of between 5 and 17%, expressed as a percentage of the dry weight of the soil.

Grain size distribution tests were completed on two (2) samples of the fill. The results of the tests are plotted on Figure D1 in Appendix D and summarized in Table 4.1.

Table 4.1: Grain Size Distribution Results – FILL

Borehole	Sample	Depth (m)	Unified Soil Classification System	% Gravel	% Sand	% Silt and Clay
BH18-1	SS1	0.07 to 0.6	SILTY SAND (SM) with gravel	22	46.5	31.5
BH18-2	SS2	0.8 to 1.4	CLAYEY SAND with gravel (SC)	26.5	24.5	49

4.4 BEDROCK AND AUGER REFUSAL

The bedrock surface was encountered at a depth of approximately 1.3 m below ground surface corresponding to an elevation of about 76.7 m in BH18-2. The presence of bedrock was confirmed by coring at this location. A detailed description of the rock core retrieved from this borehole is provided on the Bedrock Core Log in Appendix C. Rock core photographs are also provided in Appendix C.

The bedrock core obtained from BH18-2 consisted predominantly of grey, slightly weathered limestone with shale laminations to interbeds. An approximately 70 mm thick zone of fractured rock or void was encountered at a depth of about 3.1 m depth. Rock Quality Designation (RQD) values of between 70 % and 100 % were recorded within the core runs indicating the bedrock is of fair to excellent quality.

Compressive strength tests conducted on two (2) rock core samples collected from depths of about 1.5 m and 2.7 m indicated that the compressive strength of the samples tested were 207 MPa and 148 MPa, respectively. The test results indicate that the bedrock is very strong.

Auger refusal was also encountered at a depth of 1.3 m in BH18-1. As this is the same depth as the bedrock was encountered in BH18-2, the refusal in inferred to be a result of encountering bedrock.

4.5 GROUNDWATER CONDITIONS

No groundwater seepage was noted within the overburden materials during drilling and no accumulation of water was observed within the boreholes immediately after completion of drilling, in BH18-1 and BH18-2.

A groundwater monitoring well, with a well screen located at a depth of 2.1 m to 3.7 m below ground surface, was installed in BH18-2. The groundwater level in this well was recorded at approximately 3.1 m below ground surface on April 13, 2018.

Perched groundwater conditions may develop within and above the fill materials and bedrock. Therefore, it should be noted that groundwater levels at the site will be subject to fluctuations due to seasonal changes and precipitation events.

4.6 CHEMICAL ANALYSIS

Chemical testing related to the potential for corrosivity and sulphate attack was completed on one a selected soil sample from BH18-2. Table 4.2 below summarizes the test results. The laboratory test report is provided in Appendix E.

Table 4.2: Summary of Chemical Testing Results

Borehole No.	Sample No./Depth	Physical Characteristics				
		% Solids (by Wt.)	pH	Resistivity (Ohm-m)	Chloride (ug/g)	Sulphate (ug/g)
BH18-2	SS2/1.1m	100	7.34	5.95	20	1960

5.0 DISCUSSION AND RECOMMENDATIONS

This section provides engineering input related to the geotechnical design aspects of the proposed development based on our interpretation of the available subsurface information described herein and our understanding of the project requirements.

The discussion and recommendations presented in the following sections of this report are intended to provide the designers with preliminary information for planning and design purposes only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities.

5.1 SEISMIC SITE CLASS

The seismic Site Class value, as defined in Section 4.1.8.4 of the 2012 Ontario Building Code (OBC), contains a seismic analysis and design methodology which uses a seismic site response and site classification system defined by the shear stiffness of the upper 30 metres of the ground below the foundation level. There are six site classes (from A to F), decreasing in stiffness from A (hard rock) to E (soft soil); Site Class F denotes problematic soils for which a site-specific evaluation is required.

Based on the results of the Stantec field investigation, it is appropriate to classify the existing ground conditions at the subject site as a Site Class of C. We note that a building founded on the bedrock at this site can likely be designed with a better site class (i.e. a Site Class of A or B); however, the OBC requires measurement of shear wave velocities in the bedrock be carried out before these site classes can be used in design.

A copy of the NBC Seismic Hazard Calculation Data sheet is provided in Appendix F for reference.

5.2 FROST PENETRATION

The design frost penetration depth for the Ottawa area is 1.8 m. All foundations founded on frost-susceptible materials should be provided with a minimum of 1.8 metres of earth cover or equivalent insulation for frost protection purposes. However, footings bearing on competent, undisturbed limestone bedrock are not required to be founded below the frost penetration depth.

Exterior slabs-on-grade or slabs-on-grade within unheated area will be subject to the risk of heave due to frost. Consideration could be given to the use rigid insulation to protect structures against frost action; however appropriate frost tapers would need to be incorporated at the ends of the insulation.

It is to be noted that the above frost penetration depth is applicable only to foundation design. Short period deeper frost penetrations, which would have little impacts on foundations, may occur. The typical soil cover for frost protection of watermains and services is 2.4 m below ground surface in the City of Ottawa.

5.3 SITE PREPARATION

5.3.1 Grade Raise Restrictions

The final site grades in the area of the proposed building are understood to vary between approximately 77.7 m and 78.4 m which are close to existing site grades. Based on this information, significant grade raises are not planned at the site.

The native subsurface materials present at the site consist of a thin layer of fill overlying limestone bedrock which is not highly compressible. Therefore, grade raises, if required, are not anticipated to result in settlements of the underlying soil/bedrock that would adversely affect the performance of the proposed facilities.

5.3.2 Site Preparation and Floor Slab Construction

In preparation for construction of the building foundations and floor slab, all vegetation and tree stumps/roots, organic soil (including topsoil), existing fill materials, existing infrastructure (e.g. foundations, floor slabs and services for the existing building) and any loose, wet, and/or otherwise disturbed native material should be removed from within the footprint of the proposed building and any other settlement sensitive areas.

A portion of the existing residential building contains a basement that would extend below the level of both the floor slab and typical founding elevations for the proposed building. To provide consistent subgrade conditions, all below-grade portions of the existing building as well as the basement wall backfill materials should be removed to expose the surrounding bedrock.

Following removal of the above noted materials, the prepared subgrade will require inspection by geotechnical personnel to verify all unsuitable material has been removed. Where removal of unsuitable materials extends below the floor slab subgrade level, the grade beneath the building floor slab should be raised/reinstated to design subgrade level using Structural Fill consisting of Ontario Provincial Standard Specification (OPSS) Granular B Type I or II materials that are placed in lifts no thicker than 300 mm and compacted to at least 100% of the material’s Standard Proctor Maximum Dry Density (SPMDD).

The floor slab for the lowest level of the proposed building is understood be located below the final exterior grades. This level should either be designed to be waterproof/watertight or an underslab drainage system should be provided to prevent hydrostatic pressure build-up beneath the floor due to fluctuations in the water table and/or infiltration of surface water. At least 300 mm of free draining material, such as 16 mm clear crushed stone, should be provided beneath the base of the slab. These materials should be lightly-compacted to provide a level surface and improve trafficability during construction. Subdrains consisting of 100 mm diameter perforated pipes should be provided at approximately 6 m spacings within the floor slab bedding and should be connected to a frost-free gravity outlet or a sump form which the water is pumped. The requirements for a underslab vapour barrier should be in accordance with the requirements of the Ontario Building Code.

As noted later in this report, the proposed building is recommended to be supported on shallow foundations bearing on the limestone bedrock. In the area of the existing basement, the new building foundations will need to be lowered to found on the undisturbed bedrock. If, in other areas, fill materials are present beneath the proposed founding elevation (e.g. areas where the bedrock was previously excavated for service construction) or if the bedrock surface is found to be irregular, all fill materials and/or loose rock should be removed to expose the competent bedrock surface and the grade brought up to the founding level by placing 5 MPa concrete; the limits of the concrete placement should be determined on site by a geotechnical engineer.

Inspection and testing services will be critical to ensure that all fill is removed, and that new engineered fill and concrete used is suitable and is placed competently.

5.4 FOUNDATION DESIGN INPUT

Based on the subsurface conditions encountered at the site and the proposed finished floor slab level of the proposed building, the preferred foundation option from a geotechnical perspective for this site is the use of shallow strip and/or spread footings founded on the limestone bedrock.

5.4.1 Foundation Design Parameters - Shallow Footings

Shallow foundations bearing directly on competent limestone bedrock can be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 1 MPa. The factored geotechnical bearing resistance at ULS incorporates a resistance factor of 0.5. The settlement of footings sized using the above factored bearing resistance are expected to be less than 15 mm and, therefore, Serviceability Limit States (SLS) are not anticipated to control design for footings bearing on the bedrock at this site.

As discussed in the site preparation section of the report, the foundations will need to be lowered in the vicinity of the portion of the existing building that contains a basement in order to be founded on the bedrock. Stepping of the foundations should be carried out in accordance with the requirements of the Ontario Building Code.

All footings should be founded on above a relatively level rock surface. All soil, and broken, fractured and/or loose bedrock should be removed to expose the competent bedrock surface. The bedrock surfaces beneath all footings **must be inspected by qualified geotechnical personnel** prior to placing concrete to verify assumed foundation bearing conditions and integrity.

The unfactored horizontal resistance to sliding of the spread foundations may be calculated using the following unfactored coefficient of friction:

0.65 between fractured limestone bedrock and cast-in-place concrete

In accordance with Table 8.1 of the Canadian Foundation Engineering Manual 4th Edition (CFEM), a resistance factor (ϕ) against sliding (for frictional materials) of 0.8 should be applied to obtain the factored resistance at ULS.

5.4.2 Foundation Wall Backfill

The soils/fill materials encountered at the site are susceptible to frost heave and should not be used as backfill against exterior, unheated, or well insulated foundation elements within the depth of frost penetration. To avoid problems with frost adhesion and heaving, foundation walls in these areas should be backfilled with non-frost susceptible granular fill meeting the gradation requirements of OPSS Granular B Type I materials. The fill should be placed in maximum 300-millimetre thick lifts and should be compacted to at least 95 percent of the material's SPMDD using suitable vibratory compaction equipment.

In areas where hard surfacing (e.g., concrete slabs, sidewalks) surround the building, differential frost heaving will occur between the granular fill backfill zone and other areas. To reduce this differential heaving, a frost taper of the granular backfill is recommended. The frost taper should extend up from 1.5 metres below finished exterior grade (at the foundation wall) at a slope of 3 horizontal to 1 vertical, or flatter, to the base of the pavement subgrade level.

Exterior grades should be sloped away from the building to prevent ponding of water around the buildings. As the lowest floor slab level is below the final exterior grades, the basement wall backfill should be drained using a perimeter drainage system (e.g. perforated subdrain) which is provided positive drainage to storm sewer or to a sump from which water is pumped similar to the underslab drainage system discussed in section 5.3.2.

5.5 EARTH PRESSURES

Earth pressures will need to be considered in the design of the basement walls and the small retaining wall around the stairwell at the north end of the building. The total active (P_A), passive (P_P) and at-rest (P_O) thrusts can be calculated using the following equations:

$$P_A = \tfrac{1}{2} K_a \gamma H^2$$
$$P_P = \tfrac{1}{2} K_p \gamma H^2$$
$$P_O = \tfrac{1}{2} K_o \gamma H^2$$

where;
H = height of the wall
γ = unit weight of the backfill soil

Values for K_a, K_p, K_o and γ are provided in the table below. These values are based on the assumption that a horizontal back slope will be utilized behind the wall system. At-rest earth pressures should be used in the design of walls that are restrained from movement. The thrust acts at a point one third up the height of the wall.

Table 5.1: Non-Seismic Lateral Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B - Type I	Existing Fill Materials
Bulk Unit Weight, γ (kN/m^3)	22	20
Effective Friction Angle	32°	30°
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.5
Coefficient of Active Earth Pressure (K_a)	0.31	0.33
Coefficient of Passive Earth Pressure (K_p)	3.25	3.0

Total active and passive thrusts under earthquake conditions can be calculated using the following equations:

$P_{AE} = ½ K_{AE} \gamma H^2$

$P_{PE} = ½ K_{PE} \gamma H^2$

where;

K_{AE} = active earth pressure coefficient (combined static and seismic)
K_{PE} = passive earth pressure coefficient (combined static and seismic)
H = height of wall
γ = total unit weight

The recommended seismic earth pressure parameters are provided in Table 5.3 below. The angle of friction between the soil and the wall has been assumed to be 0° to provide a conservative estimate.

Table 5.2: Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B - Type I	Existing Fill Materials
Bulk Unit Weight, γ (kN/m^3)	22	21
Effective Friction Angle	32°	30°
K_{AE} (Non-Yielding Wall)	0.51	0.54
Height of Application of P_{AE} from base as a ratio of wall height, (H) – Non Yielding Wall	0.44	0.44
Active Earth Pressure (K_{AE}) – Yielding Wall	0.39	0.42
Height of Application of P_{AE} from base as a ratio of wall height, (H)	0.39	0.39
Passive Earth Pressure, (K_{PE})	3	2.75
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.31	0.31

In order to use the coefficients of pressure for the granular materials presented in the tables above, the granular backfill must be provided within a wedge extending from the base of the wall at 45 degrees (or smaller) to the horizontal.

5.6 EXCAVATIONS AND BACKFILL

5.6.1 Temporary Excavations

All temporary excavations should be carried out in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. Care should be taken to direct surface water away from the open excavations.

The excavation side slopes should be protected from precipitation or surface runoff to prevent further softening that could lead to additional sloughing and caving. If sloughing and cave-in are encountered in the excavation, the slopes should be further flattened to achieve a stable configuration.

5.6.1.1 Excavations in Soil

Excavations required for the building construction are expected to typically be less than 2 m in depth although localized, deeper excavations could be required (e.g. for service connection tie-ins). The building footprint is offset a minimum of 4 m from adjacent structures but is located within 1.5 m of the driveway for the structure on the lot to the south of the site.

Based on the boreholes advanced within the site, shallow excavations within the overburden at the site are anticipated to extend through fill materials varying in composition from silty sand with gravel to clayey sand that are located above the groundwater table. Conventional hydraulic excavating equipment is considered suitable for developing excavations in these materials.

The existing fill materials are above the water table and would be classified as Type 3 soils as defined by Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. Within Type 3 soils, temporary open cut excavations must be sloped at 1 horizontal to 1 vertical from the base of the excavation per the requirements of OHSA. The excavation sideslopes would need to be flattened and/or appropriate groundwater control measures implemented if excavations are carried out in overburden materials below the water table.

The excavations must be developed in a manner to ensure that adequate support is provided for any existing structures, utilities or underground services located adjacent to the excavations. Where there is insufficient space to develop open cuts without resultant loss of support for existing features (e.g. guy wire foundations for the hydro pole located near the north property line) or encroaching into adjacent properties, the installation of a shoring system meeting the requirements of the OHSA would be required. Given the shallow depth to bedrock, a cantilevered soldier pile and lagging system could likely be used. All shoring systems should be designed and approved by a qualified Professional Engineer. The excavation support system should be designed to resist loads from traffic.

5.6.1.2 Excavations in Bedrock

The bedrock surface was encountered at an approximate depth of 1.3 m, in both BH18-1 and BH18-2. For shallow depths of bedrock excavation, rock removal could be accomplished using mechanical methods (such as hoe ramming in conjunction with closely-spaced line drilling), although this method may be slow and hindered by the presence of thick beds/slabs of very strong limestone bedrock. Excavations extending significantly into the rock (if required) will more-efficiently be carried out using drill and blast procedures, if permitted by the City of Ottawa. All blasting should be conducted by a licensed blasting contractor with sufficient experience on projects of similar scale and detail.

Bedrock excavations with near vertical sidewalls should stand unsupported for the construction period, at least for moderate depths (i.e., total excavation depths of less than about 3 metres). However, blast damage to the bedrock walls must be avoided or else rock reinforcement would be required. Special construction procedures such as line drilling at close spacing prior to blasting or pre-shearing of the excavation limits using controlled blasting may be required to minimized blast-related bedrock damage. Furthermore, all loose rock must be removed from the sidewalls of such excavations and the sidewalls should be inspected by experienced to geotechnical personnel prior to worker-entry into the excavation to assess the sidewall conditions and confirm that additional support measures are not required.

Where new/replacement service lines will be located below the bedrock surface, consideration could be given to replacing the pipes within the existing trenches in order reduce the amount of bedrock excavation required. If the existing trench widths within the bedrock are not wide enough, excavation into the sidewalls of the existing trenches could be carried out to increase the trench width. This method of excavation will result in less bedrock removal and decreased vibrations in comparison to excavating new trenches in the bedrock.

5.6.2 Vibration Considerations and Precondition Surveys

The required construction activities for the proposed building will generate vibrations that will be perceptible to nearby residents. The vibrations are expected to be greatest if bedrock excavation by blasting or mechanical methods is carried out. A pre-construction survey should also be carried out on all of the surrounding structures and utilities prior to excavation works.

Significant precautions should be exercised if blasting is allowed/planned to be carried out due to the close proximity of existing buildings. The blasting and rock excavation activities should be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be avoided. This will require blast designs to be prepared by a specialist in this field. If practical, blasting should commence at the furthest points from the closest structure or service to assess the ground vibration attenuation characteristics and to confirm the anticipated ground vibration levels based on the contractor's blasting methods.

It is recommended that Contractor be required to submit a detailed blasting or rock excavation design plan and monitoring proposal prepared by a vibrations specialist prior to commencement of blasting or rock excavation. The construction vibrations should generally be limited to the maximum, frequency dependent peak particle velocities outlined below.

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

Should there be structures in the area sensitive to vibrations, more stringent specifications should be developed by a vibration specialist. For instance, the peak particle velocities may need to be limited further if there are any historic buildings in the area. Vibration monitoring should be carried out prior to and throughout the construction period.

5.6.3 Temporary Dewatering Considerations

The groundwater level measured in the piezometer installed in BH18-2 was at a depth of about 3.1 m below ground surface at the time of the field investigation which is below the bottom of the overburden materials. However, perched groundwater may be encountered at the time of excavation and may require dewatering. Groundwater inflows into excavations developed within the overburden should be possible to be handled by pumping from filtered sumps within the excavation areas.

Increased water inflows would occur if excavations extend below the groundwater table in fractured bedrock. Control of groundwater into shallow excavations into the bedrock is expected to be able to be handled by pumping from sumps in the excavations; however, further investigation would be required to assess potential inflow volumes for these conditions.

5.7 PIPE BEDDING AND BACKFILL

OPSS Granular A materials should be placed below sewer and water pipes as bedding material. The bedding should have a minimum thickness of 150 mm or more if required to meet City of Ottawa standards. Where unavoidable disturbance to the subgrade surface does occur, it may be necessary to thicken the bedding layer or provide a subbedding layer of compacted Granular B Type II materials. Pipe backfill and cover materials should also consist of OPSS Granular A material. A minimum of 300 mm vertical and side cover should be provided. These materials should be compacted to at least 95% of the material's SPMDD in lifts no greater than 300 mm. Clear crushed stone backfill should not be permitted as pipe bedding materials.

Where the pipe trenches will be covered with hard surfaced areas, the type of native material placed in the frost zone (i.e. between subgrade level and the top of the pipe cover materials) should match the soil exposed on the trench walls for frost heave compatibility. A 3H:1V frost taper is recommended in order to minimize the effects of differential frost heaving if materials different than those present in excavation sidewalls are used as backfill.

Trench backfill above the pipe/duct cover materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 98 % of the material's SPMDD using suitable vibratory compaction equipment.

The existing fill materials that are free of organic matter and other deleterious materials, may be considered suitable for reuse as trench backfill or as general site grade fill (i.e. materials used to raise the site grade to the design elevations). The ability to compact these materials to the required levels is dependent on the moisture content of the materials; thus, the amount of re-useable material will be dependent on the natural moisture content, weather conditions and the construction techniques at the time of excavation and placement. In addition, any boulders or cobbles with dimensions greater than 150 mm should be removed from these materials prior to placement.

Any imported fill materials proposed for use as bedding or trench backfill should be tested and approved by a geotechnical engineering firm prior to delivery to the site.

Materials testing and inspection should be carried out during construction to ensure the materials meet the project specifications and required level of compaction.

5.8 ADVERSE WEATHER CONSTRUCTION

Additional precautions, effort, and measures may be required, when and where construction is undertaken during late fall, winter, and/or early spring (i.e. when the temperature and climatic conditions can have an adverse influence on the standard construction practices) or during periods of inclement weather. With respect to all earthworks activities undertaken during the late fall through to late spring, when less-than-ideal weather and construction conditions may prevail, the following comments are provided:

1. Foundations shall be constructed on non-frozen ground only; where non-frozen ground includes the material at surface and all underlying soils. The non-frozen nature of the ground must be confirmed by a geotechnical inspection within 1 hour of concrete placement.
2. Similarly, concrete for floor slabs should not be placed on frozen ground. Test pits or other measures should be undertaken to confirm that the soils beneath the slab(s) are frost-free prior to slab construction.
3. Following construction of footings, protection measures must be provided to prevent freezing of the foundation subgrade/bearing soils and for protection of the concrete during curing.
4. Engineered fills including pipe bedding and cover, are recommended to consist of imported granular materials, including OPSS Granular A or B materials. The use of non-granular fill materials may be considered for use as trench backfill but obtaining suitable compaction of such materials could be extremely problematic, and these materials should only be used if large, post-construction settlement of the trench backfill is deemed acceptable.

5. Fill placement should be inspected by qualified field personnel on a full-time basis under the supervision of a geotechnical engineer, with the authority to stop the placement of fill at any time when conditions are considered to be unfavourable.
6. Backfill materials, including imported materials, that contain ice, snow, or any frozen material should not be accepted for use.
7. Overnight frost penetration may occur, even in granular fill materials, where precipitation and ground surface runoff pools and accumulates, and freezing temperatures exist. The on-site clayey soils are prone to frost heave due to ice lensing. Any frozen materials should be removed prior to placing subsequent lifts of engineered fill. Breaking the frost in-situ is not considered acceptable.
8. It may be necessary to stop the placement of engineered fill during periods of cold, where ambient temperatures are -5° C or less exist.

Appropriate scheduling of the work may also require specific consideration and revision from that typically adopted. The scope of work intended may have to be reduced or adjusted, and/or only select construction activities be undertaken during specific climatic conditions. The areas of planned fill placement may have to be reduced on a daily basis, and the extent of excavations may have to be limited.

5.9 CEMENT TYPE AND CORROSION POTENTIAL

One (1) test was conducted on a selected soil sample to determine the water soluble sulphate content of the site soils. The sulphate concentration in the sample was 1960 ug/g as shown in Table 4.2. The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations higher than 1000 µg/g generally indicate that a sulphate attack may be expected on concrete in contact with soil and groundwater. High Sulphate-Resistant Portland cement is therefore recommended for use in concrete at this site.

The test results provided in Table 4.2 should be used by the designers in assessing the potential for corrosion of steel elements and may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The soil pH result of 7.3 is within what is considered the normal range for soil pH of 5.5 to 9.0. The pH level of the tested soil does not indicate a highly corrosive environment. The reported resistivity of 5.95 (ohm-m) suggests a severe corrosive environment.

6.0 CLOSURE

Not all details related to the proposed development are known at this time. In this regard, all geotechnical comments provided in this report should be reviewed and, if necessary, revised once the final plans become available. Stantec should be retained to review the final drawings and specifications to confirm that the geotechnical input provided herein has been adequately addressed.

This report has been prepared for the sole benefit of TC United Group and their agents, and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and TransCanada Pipelines Limited. Any use, which a third party makes of this report, is the responsibility of such third party. Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of TC United Group, who is identified as “the Client” within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying or unexpected site conditions
- Planning, design or construction

We trust the above information meets with your present requirements. Should you have any questions or require further information, please contact us. This report has been prepared by Ramy Saadeldin, Ph.D., P.Eng. and reviewed by Kevin Nelson, P.Eng.

Thank you for the opportunity to be of service to you.

Respectfully submitted,

STANTEC CONSULTING LTD.

LICENSED PROFESSIONAL ENGINEER
R. SAADELDIN
100506710
30 April 2018
PROVINCE OF ONTARIO

Ramy Saadeldin, PhD, P.Eng.
Geotechnical Engineering

Kevin Nelson, P.Eng.
Principal, Geotechnical Engineering

APPENDIX A

Statement of General Conditions

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec Consulting Ltd.'s present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec Consulting Ltd. is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec Consulting Ltd. will not be responsible to any party for damages incurred as a result of failing to notify Stantec Consulting Ltd. that differing site or sub-surface conditions are present upon becoming aware of such conditions.

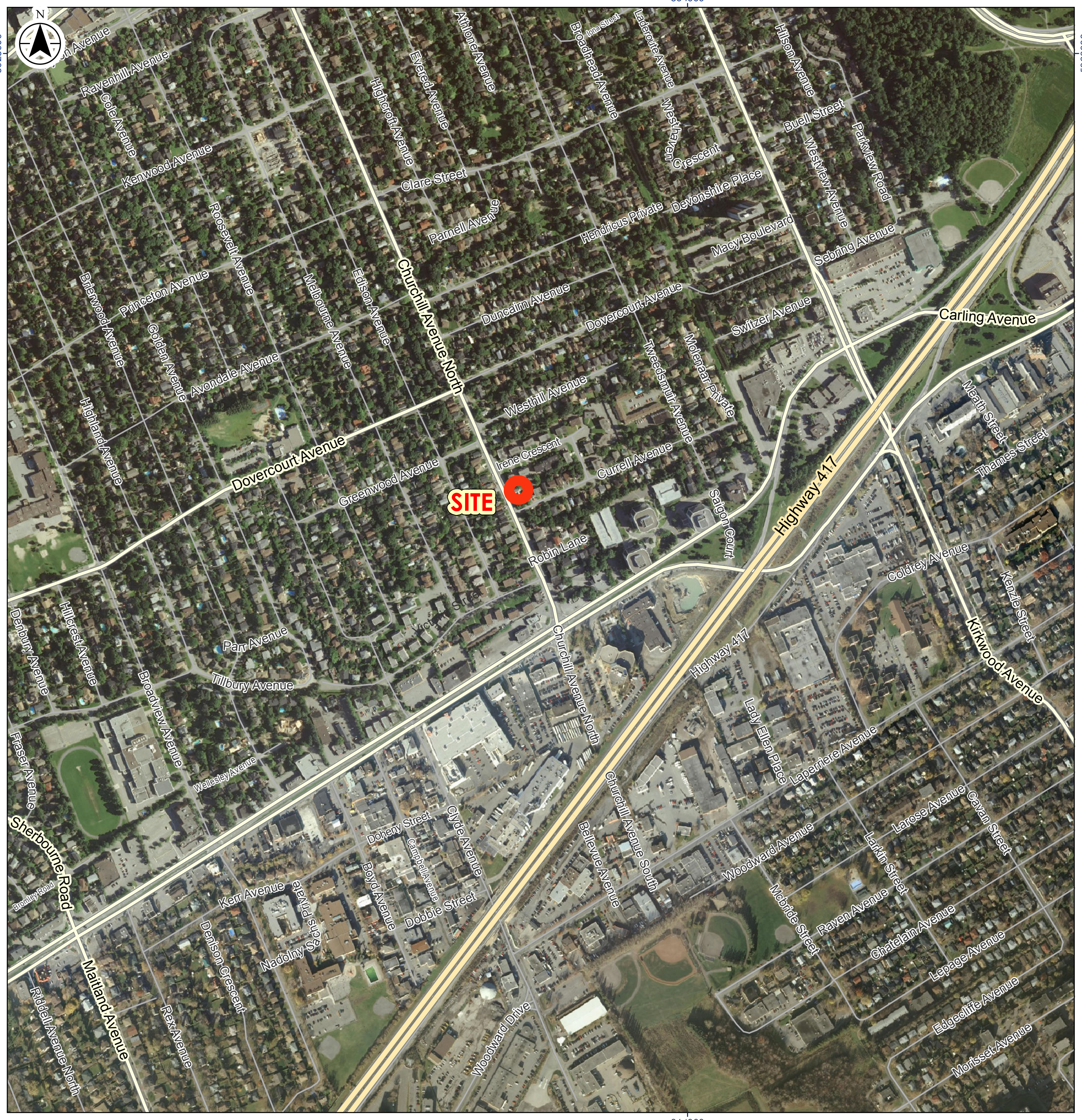
PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec Consulting Ltd., sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec Consulting Ltd. cannot be responsible for site work carried out without being present.

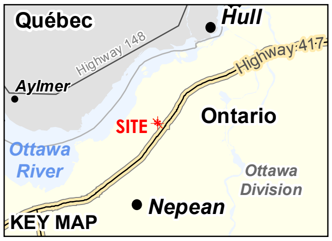
APPENDIX B

Drawing No. 1 – Key Plan

Drawing No. 2 – Borehole Location Plan

T:\Autocad\Drawings\Project Drawings\2018\121621808\Key Plan (GIS)\121621808_Key Plan.mxd Revised: 2018-04-12 By: gbriones





Legend

- Highway
- Major Road
- Minor Road

0 200 400 metres

1:10,000 (at original document size of 8.5x11)

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Project Location 689 Churchill Avene North Ottawa, Ontario	Project No. 163621808 Prepared by Gliceria Briones on 2018-04-12

Client/Project

TC UNITED GROUP
GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING, 689 CHURCHILL AVENUE N.

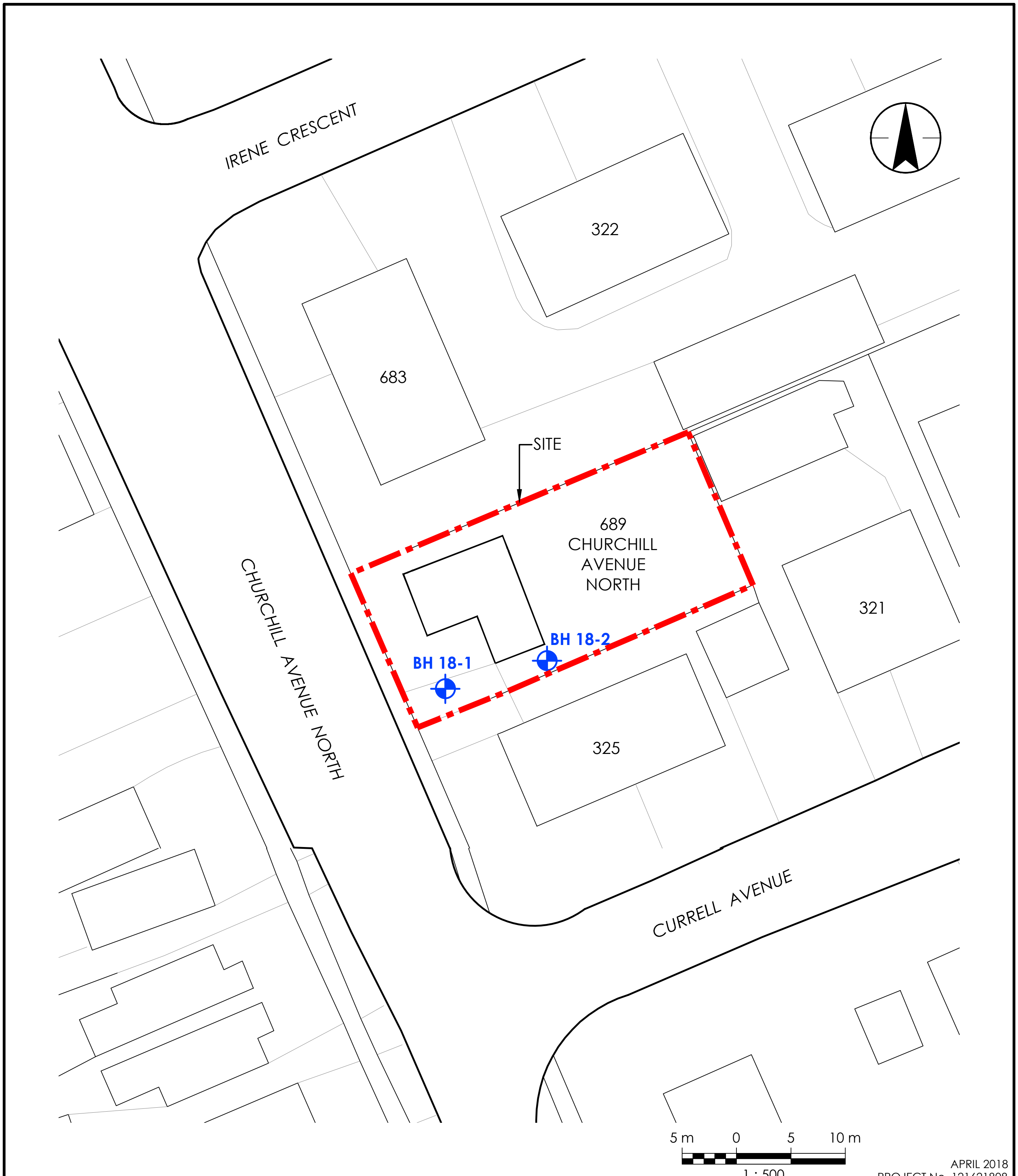
Drawing No.
1

Title
Key Plan

Notes

1. Coordinate System: NAD 1983 MTM Zone 9N.
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © Queen's Printer for Ontario, 2016.
3. Imagery provided by Frist Base Solutions 2018.

Disclaimer: Stantec assumes no responsibility for data supplied in electronic format. The recipient accepts full responsibility for verifying the accuracy and completeness of the data. The recipient releases Stantec, its officers, employees, consultants and agents, from any and all claims arising in any way from the content or provision of the data.



T:\Autocad\Drawings\Project Drawings\2018\121621808\121621808_Borehole Locations.dwg
2018/04/24 8:22 AM By: Briones, Gliceria

APRIL 2018
PROJECT No. 121621808

LEGEND

APPROXIMATE BOREHOLE LOCATION

Stantec

400 - 1331 Clyde Avenue
Ottawa, ON, Canada K2C 3G4
www.stantec.com

NOTES

1. COORDINATE SYTEM: NAD 1983 MTM ZONE 9.
2. BASE FEATURES PRODUCED UNDER LICENSE WITH THE ONTARIO MINISTRY OF NATURAL RESOURCES © QUEEN'S PRINTER FOR ONTARIO, 2013.

Client/Project

TC UNITED GROUP

GEOTECHNICAL INVESTIGATION, PROPOSED BUILDING, 689 CHURCHILL AVE. N., OTTAWA, ON.

Drawing No.

2

Title

BOREHOLE LOCATION PLAN

APPENDIX C

Symbols & Terms Used on the Borehole Records
Borehole Records
Bedrock Core Log and Photographs

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Rootmat	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor Quality*
25-50	*Poor Quality*
50-75	*Fair Quality*
75-90	*Good Quality*
90-100	*Excellent Quality*

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	*Crushed*
Severely Fractured	*Shattered or Very Blocky*
Fractured	*Blocky*
Moderately Jointed	*Sound*
Intact	*Very Sound*

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

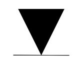
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.

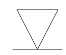
Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Igneous Bedrock	Meta-morphic Bedrock	Sedi-mentary Bedrock

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

 measured in standpipe, piezometer, or well

 inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_U	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

Stantec

BOREHOLE RECORD

N: 5 025 726 E: 441 449

BH18-1

CLIENT: TC United Group
LOCATION: 689 Churchill Ave N
DATES: BORING April 6, 2018 WATER LEVEL N/A

BOREHOLE No. BH18-1
PROJECT No. 121621808
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
0	77.90							
	77.8	70 mm ASPHALTIC CONCRETE			SS	1		7
	77.4	FILL: grey, loose, SILTY SAND with gravel						
	77.3	100 mm TOPSOIL						
	77.2	100 mm Gravel/Cobbles			SS	2		50/ 102mm
1	76.6	FILL: grey, very dense, CLAYEY SAND with gravel						
		End of Borehole - Auger refusal on inferred bedrock at 1.3 m - No groundwater seepage noted during drilling						
2								
3								
4								
5								
6								
7								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

N: 5 025 726 E: 441 458

BH18-2

CLIENT TC United Group

LOCATION 689 Churchill Ave N

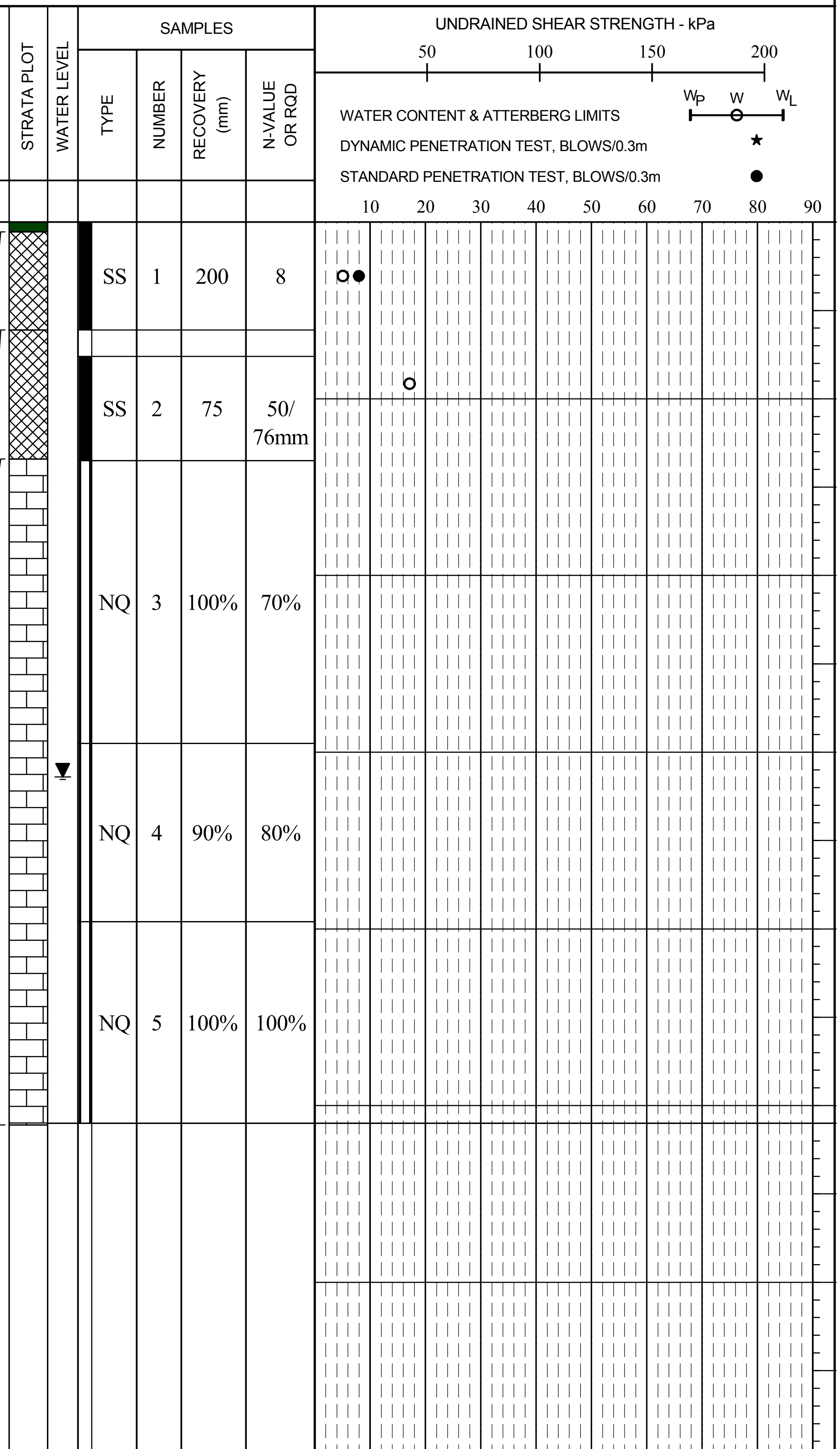
DATES: BORING April 6, 2018 WATER LEVEL 3.1 m on 13 April 2018

BOREHOLE No. BH18-2

PROJECT No. 121621808

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	78.00					
	77.9	55mm ASPHALTIC CONCRETE				
		FILL: grey, loose, SILTY SAND with gravel	SS	1	200	8
	77.4	- contains broken red brick pieces				
1		FILL: grey, very dense, CLAYEY SAND with gravel	SS	2	75	50/ 76mm
	76.7	- with some brick pieces				
		Fair to excellent quality, very strong, grey fresh to slightly weathered LIMESTONE with shale laminations to interbeds				
2			NQ	3	100%	70%
3		- brown silty clay seam at 3.1 m, followed by a 70 mm fractured zone or void	NQ	4	90%	80%
4			NQ	5	100%	100%
5	72.9	End of Borehole				
		Well Details:				
		Screen from 3.7 to 2.1 m bgs				
		Bentonite hole plug from 5.1 to 3.7 m bgs				
6		Silica sand from 5.1 to 1.8 m bgs				
		Bentonite hole plug from 1.8 to 1.1 m bgs				
		Drill cuttings and bentonite hole plug from 1.1 to 0 m bgs				
7						



Inferred Groundwater Level

Groundwater Level Measured in Standpipe

Field Vane Test, kPa

Remoulded Vane Test, kPa

Pocket Penetrometer Test, kPa

Bedrock Core Log

Stantec

Client:	TC United Group	**Project No.:**	121621808
Project:	689 Churchill Ave North	**Date:**	6-Apr-18
Contractor:	George Downing Estate Drilling Limited	**Borehole No.:**	BH18-2
		Logger:	JHJ

DEPTH FROM	RUN NO.	% CORE RECOVERY	% RQD	DEPTH TO	GENERAL DESCRIPTION (Rock Type/s, %, Colour, Texture, etc.)	STRENGTH	WEATHERING	DISCONTINUITIES: NO. OF SETS	TYPE/S	ORIENTATION	SPACING	ROUGHNESS	APERTURE	FILLING	OCCASIONAL FEATURES	DRILLING OBSERVATIONS
1.4 m	3	100%	73%	3 m	Fair to excellent quality, very strong, grey, fresh to slightly weathered, LIMESTONE with laminations to interbeds (up to 150 mm in thickness) of dark grey SHALE	R5	W2	1	JN	F	VC	RU/RP	O	Si	Unconfined compressive strengths of 207 MPa and 148 MPa measured on core samples collected from depths of 1.5 m and 2.7 m, respectively.	
									BD							
3 m	4	90%	80%	4 m		R5	W1 W2	1	JN	F	C - M	RU/RP	O	NC	At ~3.1 m depth, silty clay coating/infilling noted followed by ~0.07 m fractured zone or void.	
									BD							
4 m	5	100%	100%	5.1 m		R5	W1 W2	1	JN	F	C	RU/RP	O	Si		
									BD							

STRENGTH (MPa)

Grade/Classification	Est. Strength (MPa)
R0 Extremely Week	0.25 - 1.0
R1 Very Weak	1.0 - 5.0
R2 Weak	5.0 - 25.0
R3 Medium Strong	25.0 - 50.0
R4 Strong	50.0 - 100.0
R5 Very Strong	100.0 - 250.0
R6 Extremely Strong	>250.0

JOINT TYPE

BD = Bedding
JN = Joint
FOL = Foliation
CON = Contact
FLT = Fault
VN = Vein

ORIENTATION

F = Flat = 0-20^{0}
D = Dipping = 20-50^{0}
V = n-Vertical = >50^{0}

FILLING

T = Tight, Hard
O = Oxidized
SA = Slightly Altered, Clay Free
S = Sandy, Clay Free
Si = Sandy, Silty, Minor Clay
NC = Non-softening Clay
SC = Swelling, Soft Clay

WEATHERING

Grade/Classification	Description
W1 Fresh	No Visible Signs of Weathering
W2 Slightly	Discoloration, Weathering on Discontinuities
W3 Moderately	<50% of Rock Material is Decomposed, Fresh Core Stones
W4 Highly	>50% Decomposed to soil: Fresh Core Stones
W5 Completely	100% Decomposed to Soil: Original Structure Intact
W6 Residual Soil	All Rock Converted to Soil, Structure and Fabric Destroyed

DISCONTINUITY SPACING

Spacing (mm)	
EW = >6000	Extremely Wide
VW = 2000 - 6000	Very Wide
W = 600 - 2000	Wide
M = 200 - 600	Moderate
C = 60 - 200	Close
VC = 20 - 60	Very Close
EC = <20	Extremely Close

JOINT ROUGHNESS

Jr	Description
4	DJ = Discontinuous Joints
3	RU = Rough, Irregular, Undulating
1.5	SU = Smooth, Undulating
1.5	LU = Slickensided, Undulating
1.0	RP = Rough or Irregular, Planar
0.5	SP = Smooth, Planar
2	LP = Slickensided, Planar

Stantec	Project No.: 121621808	Rockcore Photographs
	Project Name: 689 Churchill Ave North	

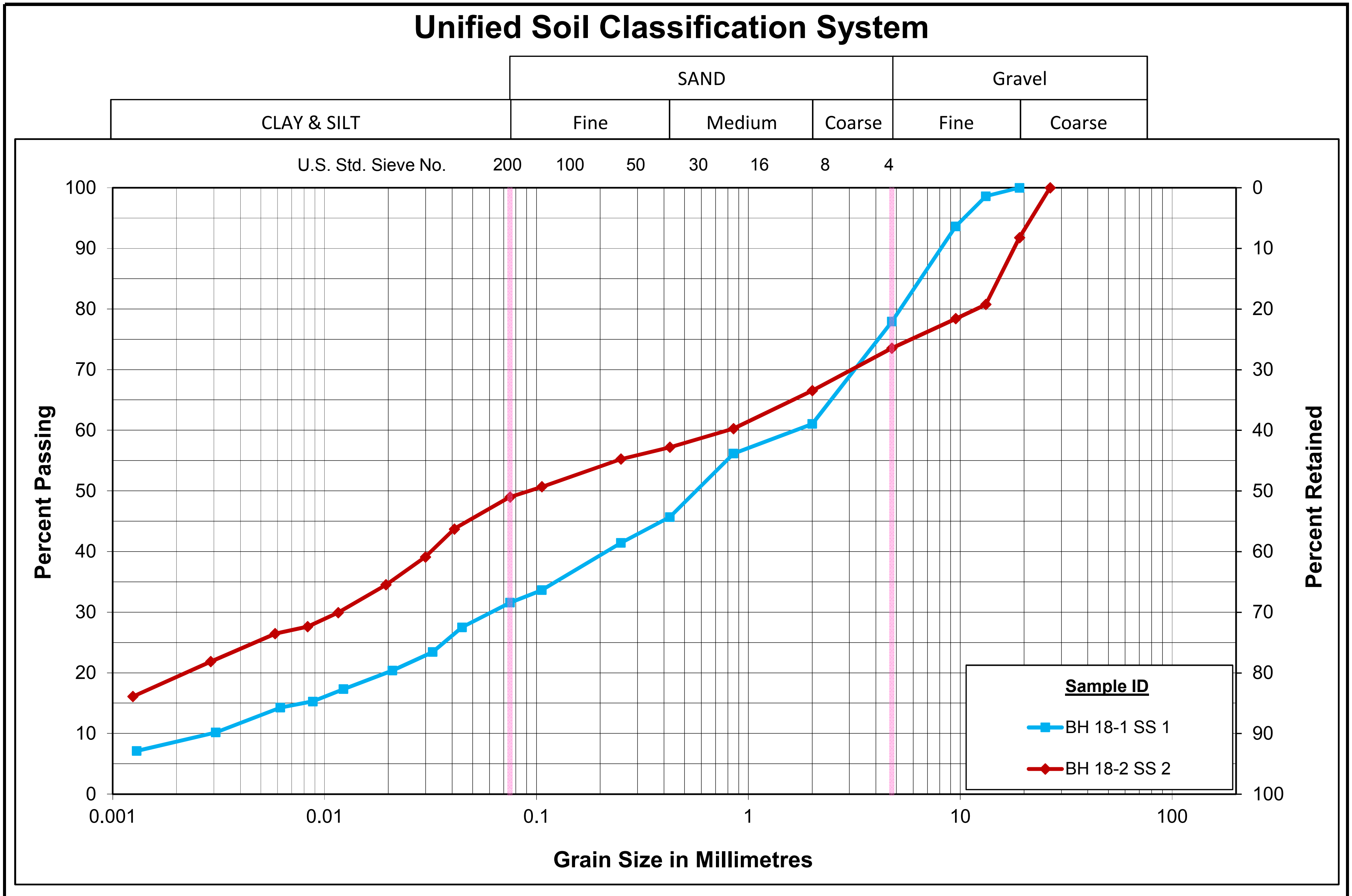
Rock Core Photo No.:	1	Borehole:	BH 18-2	Depth:	1.3 to 3.9 m

Stantec	Project No.: 121621808 Project Name: 689 Churchill Ave North	Rockcore Photographs

Rock Core Photo No.:	2	Borehole:	BH 18-2	Depth:	3.9 to 5.1 m

APPENDIX D

Laboratory Test Results
Figure D1: Grain Size Distribution Plot (FILL)



Stantec	GRAIN SIZE DISTRIBUTION SILTY SAND to CLAYEY SAND (FILL) 689 Churchill Ave. N	Figure No. D1 Project No. 121621808

APPENDIX E

Laboratory Chemical Analysis Results

PARACEL LABORATORIES LTD. | TRUSTED. RESPONSIVE. RELIABLE.

300 - 2319 St. Laurent Blvd
Ottawa, ON, K1G 4J8
1-800-749-1947
www.paracellabs.com

Certificate of Analysis

Stantec Consulting Ltd. (Ottawa)

2781 Lancaster Road, Suite 101
Ottawa, ON K1B 1A7
Attn: Rami Saadeldin

Client PO:
Project: 121621808
Custody:

Report Date: 19-Apr-2018
Order Date: 13-Apr-2018

Order #: 1815507

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1815507-01	BH 18-02, SS 2, 2 1/2 - 4 1/2 ft.

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Any use of these results implies your agreement that our total liabilty in connection with this work, however arising, shall be limited to the amount paid by you for this work, and that our employees or agents shall not under any circumstances be liable to you in connection with this work.

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	17-Apr-18	17-Apr-18
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	17-Apr-18	17-Apr-18
Resistivity	EPA 120.1 - probe, water extraction	18-Apr-18	18-Apr-18
Solids, %	Gravimetric, calculation	18-Apr-18	18-Apr-18

PARACEL
LABORATORIES LTD.

Order #: 1815507

Certificate of Analysis

Client: Stantec Consulting Ltd. (Ottawa)

Client PO:

Report Date: 19-Apr-2018

Order Date: 13-Apr-2018

Project Description: 121621808

	Client ID:	BH 18-02, SS 2, 2 1/2 - 4 1/2 ft.	-	-	-
	Sample Date:	04/06/2018 09:00	-	-	-
	Sample ID:	1815507-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	100	-	-	-
General Inorganics					
pH	0.05 pH Units	7.34	-	-	-
Resistivity	0.10 Ohm.m	5.95	-	-	-
Anions					
Chloride	5 ug/g dry	20	-	-	-
Sulphate	5 ug/g dry	1960	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	19.8	5	ug/g dry	20.2			2.1	20	
Sulphate	1960	5	ug/g dry	1960			0.1	20	
General Inorganics									
pH	7.94	0.05	pH Units	7.92			0.3	10	
Resistivity	11.7	0.10	Ohm.m	11.6			0.9	20	
Physical Characteristics									
% Solids	78.2	0.1	% by Wt.	78.2			0.1	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	115	5	ug/g	20.2	94.7	78-113			
Sulphate	2050	5	ug/g	1960	88.2	78-111			

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

APPENDIX F

Seismic Hazard Calculation Sheet

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

April 10, 2018

Site: 45.3825 N, 75.7479 W   User File Reference: 689 Churchill Ave N.

Requested by: , Stantec Consulting Ltd.

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.438	0.513	**0.431**	0.328	**0.233**	**0.116**	**0.055**	**0.015**	**0.0054**	**0.276**	**0.193**

Notes. Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. ***These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.043	0.144	0.241
Sa(0.1)	0.059	0.182	0.293
Sa(0.2)	0.054	0.157	0.249
Sa(0.3)	0.043	0.122	0.191
Sa(0.5)	0.031	0.087	0.136
Sa(1.0)	0.015	0.044	0.069
Sa(2.0)	0.0060	0.020	0.032
Sa(5.0)	0.0012	0.0047	0.0080
Sa(10.0)	0.0006	0.0019	0.0032
PGA	0.032	0.099	0.159
PGV	0.021	0.067	0.109

References

National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français