Geotechnical Investigation Report
Proposed Residential Development

17-19 Robinson Avenue, Ottawa, ON

Project No.: 121622042

Prepared for:
TC United Group
800 Industrial Ave, Unit 9
Ottawa, ON K1G 4B8

Prepared by:
Stantec Consulting Ltd.
1331 Clyde Avenue
Ottawa, Ontario

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1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) has been retained by TC United Group to prepare a Geotechnical Investigation Report for the proposed residential development located on the properties at 17-19 Robinson Avenue in Ottawa, Ontario at the location shown on the Key Plan (refer to Drawing No. 1).

Stantec previously conducted a geotechnical investigation and prepared an investigation report for a previously planned development at the site in 2010 which was reported under Stantec Project No. 163808203. As the development plans and building code requirements have changed since the submittal of the previous report, this report provides updated geotechnical design input for the proposed development. TC United Group has received approval from the former property owner to reference/use the subsurface information contained in that report and provided authorization to Stantec to use this information as part of the current assignment.

This report provides recommendations on the geotechnical design aspects of the project. Limitations associated with this report and its contents are provided in the Statement of General Conditions included in Appendix A.

2.0 BACKGROUND INFORMATION

2.1 SITE AND PROJECT DESCRIPTIONS

The proposed development site contains three former properties (17, 19, and 23 Robinson Avenue) that have been combined to form a single property. Each of the former properties currently contains an individual residential structure located in the south (front) portion of the property.

Based on the information provided by TC United Group, Stantec understands that it is planned to construct a three-storey apartment building with a basement level that has a floor slab located approximately 1 m below exterior site grades at the site. The proposed building will encompass a plan area of approximately 258 m^2 and is planned to be constructed at the location shown on Drawing No. 1 in Appendix B.

Based on a topographical plan of the site prepared by Stantec dated July 10, 2009, the ground surface within the site is generally flat with ground surface elevations varying between about 60.8 m and 61.5 m.

2.2 PREVIOUS INVESTIGATION

A total of eight boreholes designated as BH10-01 to BH10-07 and MW10-05A were drilled at the site as part of the previous investigation. The boreholes were advanced to depths of between 2.5 m and 11.6 m below ground surface. Bedrock coring was conducted in BH10-4. A monitoring well was installed at the location of MW10-05A.

3.0 SUBSURFACE CONDITIONS

3.1 GENERAL

Detailed descriptions of the subsurface soil, bedrock and groundwater conditions encountered during the 2010 investigation at the site 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 from the previous investigation are shown on the borehole records and are included in Appendix D for reference.

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 consisted of surficial materials including topsoil, asphalt and fill underlain by a native deposit of glacial till and then by shale bedrock. A summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

3.2 SURFICIAL MATERIALS

Surficial/near-surface materials consisting of topsoil, asphalt and predominantly granular fill materials, varying in composition from silty sand to gravelly sand/sand and gravel to gravel with asphalt, were encountered at all borehole locations. These materials extended to depths of approximately 0.035 m to 0.9 m below ground surface.

In addition to the fill encountered during the investigation, fill materials of greater thickness are expected to be present within the backfill zones of the existing residential structures and site services.

A deposit of dark brown silty sand was encountered below the fill at a depth of 0.9 m in BH10-02. The silty sand extended to a depth of 2.1 m below ground surface.

3.3 GLACIAL TILL

The near-surface materials described above are underlain by a native glacial till deposit that was encountered at depths of approximately 0.1 m to 1.6 m below ground surface (corresponding to elevations of approximately 59.2 m to 61.1 m).

The glacial till typically consists of silty sand with gravel. Cobbles were noted throughout the till deposit at the borehole locations. The glacial till in Ottawa is typically comprised of cobbles and boulders set in a matrix of finer-grained material (i.e. gravel, sand, silt and clay); larger boulders (e.g. in excess of 1.0 m) are common. The till is typically unsorted and without stratification, but in places contains discontinuous layers or irregular shaped masses of sand and silt. In this regard, where glacial till deposits are identified, cobbles and boulders will be present throughout the deposits and permeable layers of sand and/or silt may also randomly be present due to the unsorted and unstratified nature of the glacial till.

Standard Penetration Test (SPT) ‘N’ values measured within the glacial till ranged between 1 to > 50 per 0.3 m of penetration indicating that the glacial till is in a loose to very dense state. However, the lower ‘N’ values are inferred to be influenced by disturbance of the glacial till during drilling while the higher ‘N’ values are inferred to be influenced by the presence of cobbles and boulders.

Laboratory testing conducted on samples of the glacial till measured natural moisture contents of between 7 % and 13 %, expressed as a percentage of the dry weight of the soil.

Grain size distribution tests were completed on three (3) samples of the glacial till. The results of the tests are displayed on the figures in Appendix D and are summarized in Table 3.1. In accordance with the Unified Soil Classification System, the samples tested can be classified as SILTY SAND with gravel (SM).

Table 3.1: Grain Size Distribution Results – SILTY SAND TILL (SM)

Borehole	Sample	Depth (m)	Unified Soil Classification System	% Gravel	% Sand	% Silt and Clay
BH10-3	SS3	1.5	SILTY SAND (SM) with gravel	22	44	34
BH10-04	SS11	7.6		20	44	36
BH10-5A	SS4	5.3		20	45	35

3.4 BEDROCK

The bedrock surface was encountered at depths of approximately 9.1 m to 9.5 m below ground surface corresponding to elevations of about 51.5 m and 52.1 m, respectively.

Coring was conducted in BH10-4 to confirm the presence and type of bedrock and to provide information on the engineering characteristics of the bedrock. The first core run at this location retrieved predominantly glacial till materials with fractured shale bedrock present within the lower portion of the core run. A detailed description of the rock core is provided on the Bedrock Core Log in Appendix C.

The bedrock core obtained from BH10-04 consisted predominantly of slightly weathered, black shale. Rock Quality Designation (RQD) values of 9% and 82% were recorded within the core runs indicating that the bedrock is of very poor to good quality. The lower RQD value in the lowest core run is inferred to have been influenced by the drilling process.

Unconfined compressive strength tests on rock core samples yielded strengths of approximately 56 and 70 MPa.

3.5 GROUNDWATER CONDITIONS

A groundwater monitoring well was installed in MW10-05A on August 4, 2010. The groundwater level in the well was recorded at approximately 3.7 m below ground surface, corresponding to an elevation of 57.5 m, one week after completion of drilling.

It should be noted that groundwater level at the site will be subject to fluctuations due to seasonal changes and precipitation events as well as water level in the nearby Rideau River. The water level at the site may have changed since the time of 2010 investigation.

4.0 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.

4.1 SEISMIC DESIGN CONSIDERATIONS

4.1.1 Seismic 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 average 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.

As part of the Seismic Site Class assessment, publicly available information from a nearby site where vertical seismic profile (VSP) testing was carried out to determine the shear wave velocity of the subsurface materials was reviewed. The results of this testing indicate that shear wave velocities in excess of 1,000 m/s were measured within till deposits that had similar gradations and strength characteristics to those measured at the subject site. Based on the results of the current field investigation and the above noted VSP testing, it is appropriate to classify the existing ground conditions at the subject site as a Site Class C.

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

4.1.2 Liquefaction Potential

The potential for soil liquefaction was evaluated by comparing the cyclic stress ratio (CSR) caused by the design earthquake with the soil resistance expressed in terms of the cyclic resistance ratio (CRR). The evaluation follows the analysis methodology suggested by Idriss and Boulanger (2008) and is based on the following:

- SPT 'N' values from boreholes and available information on the shear wave velocity of till soils noted above.
- A Site Adjusted PGA of 0.28 g.
- An earthquake magnitude M_w of 6.2.

The assessment indicated that the site soils are generally not considered susceptible to liquefaction taking into account the available information on the shear wave velocities in the till and that the lowest SPT ‘N’ values measured within the till are considered to have been influenced by drilling activities.

4.2 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.

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.

Exterior slabs-on-grade or slabs-on-grade within unheated areas will also be subject to the risk of heave and deformation/cracking 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.

4.3 SITE PREPARATION

4.3.1 Grade Raise Restrictions

It is understood that significant grade raises are not planned at the site. The native subsurface materials present at the site consist predominantly of silty sand till overlying shale bedrock. These materials are not considered to be highly compressible. Therefore, grade raises of less than 1 m, if required, are not anticipated to result in settlements of the underlying soil/bedrock that would adversely affect the performance of the proposed facilities.

4.3.2 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 buildings) 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. To provide consistent subgrade conditions, all below-grade portions of the existing buildings as well as basement wall backfill materials should be removed to expose the native glacial till. Following removal of the above noted materials, the prepared subgrade will require inspection by geotechnical personnel to verify all unsuitable material has been removed.

The existing basements and foundations of the existing structures at the site are anticipated to extend below the basement floor slab level for the proposed building. Where removal of existing structures and/or unsuitable materials extends below the floor slab subgrade level, the grade beneath the new building floor slab should be raised/reinstated to the 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 basement/lowest level of the proposed building is understood to 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 geotextile encapsulated, 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 from which the water is pumped. The requirements for an underslab vapour barrier should be determined 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 native silty sand till deposit. If existing fill materials or structures are present beneath the proposed founding elevations, all such fill materials and structures should be removed from beneath the footprint of the building, the footings and the zone of influence of all footings, to expose the native glacial till surface. The zone of influence is defined by a line drawn at 1 horizontal to 1 vertical, outward and downward from the edge of the footings. The grade should be raised back up to the founding level using Structural Fill as discussed above.

Inspection and testing services will be critical to ensure that all fill, existing structures and unsuitable materials are removed beneath the proposed building, and that new engineered fill and concrete used is suitable and is placed competently.

4.4 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 for this site is the use of shallow strip and/or spread footings bearing on either the undisturbed native till deposits or compacted Structural Fill placed above the undisturbed till.

4.4.1 Foundation Design Parameters - Shallow Footings

Shallow foundations bearing directly on undisturbed native silty sand till or on Structural Fill placed above the native silty sand till can be designed using factored geotechnical resistance values presented in Table 4.1 below.

Table 4.1: Geotechnical Resistance for Shallow Footings

Footing Type and Width (m)	Minimum Footing Embedment (m) Below Basement Floor Slab Surface	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa)
Square Footings			
1			225
2	0.8	300	200
2.5			175
Strip Footings			
0.5 to 1.5	0.8	200	175

Notes:
The geotechnical resistances in the above table are provided for the range of footing widths and the minimum footing embedment depths (below the basement floor slab surface) listed in the above table. Additional input should be provided by the geotechnical engineer if the foundation sizes or embedment depths are outside of the ranges outlined above.

The factored geotechnical bearing resistance at ULS incorporates a resistance factor of 0.5. The post-construction total and differential settlements of footings sized using the above SLS bearing pressure should be less than about 25 and 20 millimetres, respectively, provided that the soil at or below founding level is not disturbed during construction.

The native soils are highly susceptible to disturbance by construction activity especially during wet or freezing weather. Care should be taken to preserve the integrity of the materials as bearing strata. It is essential that the founding level for the footings be inspected by the geotechnical engineer prior to placing concrete. If the concrete for the footings on the native soil cannot be placed immediately after excavation and inspection, it is recommended that a working mat of lean concrete be placed in the excavation to protect the integrity of the bearing stratum.

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

- 0.55 between OPSS Granular A or B Type II materials and cast-in-place concrete
- 0.45 between silty sand till 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.

4.4.2 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 surface 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 understood to be 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 a storm sewer or to a sump from which water is pumped similar to the underslab drainage system discussed in section 4.3.2.

4.5 EARTH PRESSURES

Earth pressures will need to be considered in the design of the basement walls. 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 4.2: Non-Seismic Lateral Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B - Type I
Bulk Unit Weight, γ (kN/m^3)	22
Effective Friction Angle	32º
Coefficient of Earth Pressure at Rest (K_o)	0.47
Coefficient of Active Earth Pressure (K_a)	0.31
Coefficient of Passive Earth Pressure (K_p)	3.25

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 4.3 below. The angle of friction between the soil and the wall has been assumed to be 0° to provide a conservative estimate.

Table 4.3: Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B - Type I
Bulk Unit Weight, γ (kN/m^3)	22
Effective Friction Angle	32º
K_{AE} (Non-Yielding Wall)	0.51
Height of Application of P_{AE} from base as a ratio of wall height, (H) – Non Yielding Wall	0.44
Active Earth Pressure (K_{AE}) – Yielding Wall	0.4
Height of Application of P_{AE} from base as a ratio of wall height, (H) - Yielding Wall	0.39
Passive Earth Pressure, (K_{PE})	2.99
Height of Application of P_{PE} from base as a ratio of wall height, (H)	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 out from the base of the wall at 45 degrees (or smaller) to the horizontal.

4.6 EXCAVATIONS AND BACKFILL

4.6.1 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.

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).

Shallow excavations within the overburden at the site are anticipated to extend through fill materials varying in composition from silty sand with gravel to sand and gravel and the native silty sand till deposit. Conventional hydraulic excavating equipment is considered suitable for developing excavations in these materials recognizing that additional effort will be required to remove cobbles and boulders within the glacial till. Boulders larger than 0.3 metres in size should be removed from the excavation side slopes.

The existing fill materials and the native glacial till deposit that are above the water table 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 side slopes 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 or encroaching into adjacent properties, the installation of a shoring system meeting the requirements of the OHSA would be required. 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 and adjacent building foundations.

4.6.2 Temporary Dewatering Considerations

The groundwater level measured in the piezometer installed in MW10-05A was measured to be at a depth of about 3.7 m below ground surface in 2010. Control of groundwater into shallow excavations into the glacial till deposit is expected to be able to be handled by filtered sumps within the excavation areas.

More significant groundwater inflows should be expected for deeper excavations that extend below the groundwater level. More extensive dewatering systems (e.g. external dewatering system using well points or other dewatering wells) could be required for such conditions. Depending on the depth of excavations, dewatering activities may require either registration in the Ministry of the Environment and Climate Change (MOECC) Environmental Activity and Sector Registry (EASR) or obtaining a Permit to Take Water (PTTW) from the MOECC depending on the anticipated groundwater removal rates. A separate hydrogeological assessment, should be completed to confirm these requirements before such excavations are undertaken. This assessment should include measurement of the in situ hydraulic conductivity of the site soils.

4.7 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 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 sub-bedding layer of compacted Granular B Type II materials. Pipe backfill and cover materials should also consist of OPSS Granular A material. A minimum thickness of 300 mm of these materials should be provided as vertical and side cover beside and over top of the pipes. 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 or cover 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 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 and the native glacial till 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 levels of compaction.

4.8 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 or above 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 native 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.

4.9 CEMENT TYPE AND CORROSION POTENTIAL

The results of two (2) tests conducted on selected soil samples to determine the water soluble sulphate content of the site soils completed during the previous investigation at the site are summarized in Table 4.4 below. The results are provided to aid in the selection of coatings and corrosion protection systems for items such as steel pipe in contact with the soil and groundwater at the site.

Table 4.4: Chemical Analysis Results

Borehole No.	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
BH10-3	SS5	3.1 m – 3.7 m	7.82	22	145	50.6
BH10-6	SS5	3.1 m – 3.7 m	7.87	6	138	63.5

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. The soluble sulphate concentrations for the samples were 138 µg/g and 145 µg/g. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with the soil and groundwater. General Use (GU) Portland cement is appropriate for use at the site.

The test results provided in the Table 4.4 may also be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The pH, resistivity, and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH values were 7.82 and 7.87, which are within what is considered a normal range for soil pH of 5.5 to 9.0. The pH levels of the tested soils do not indicate a highly corrosive environmental. The reported resistivity levels of 50.6 Ohm-m and 63.5 Ohm-m suggest a moderate degree of corrosiveness for steel.

5.0 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 TC United Group. 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.

Ramy Saadeldin, PhD, P.Eng.
Senior Geotechnical Engineer

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.

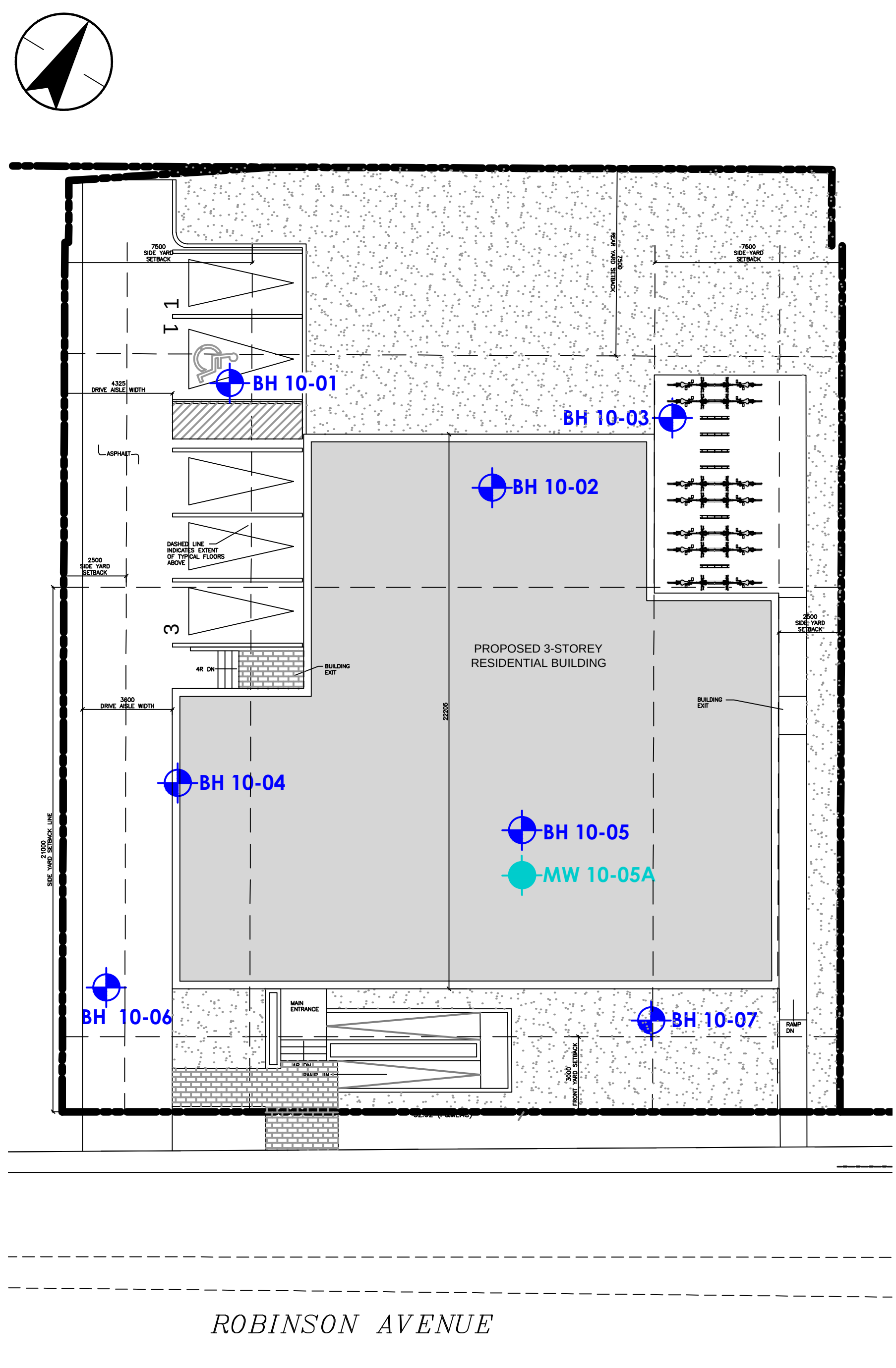
INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications 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.

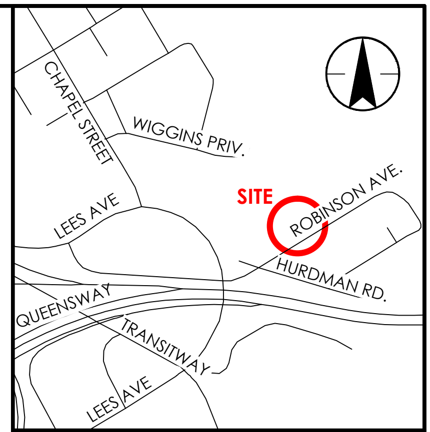
VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial 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 – Borehole Location Plan





KEY PLAN SCALE 1 : 12 500

NOTES

1. COORDINATE SYTEM: NAD 1983 MTM ZONE 9.
2. BASEPLAN FROM A PDF PLAN PROVIDED BY TC UNITED GROUP (FILENAME: 1821 - TCU Robinson1719_SitePlan.pdf).

3 0 3 m 6 m

1 : 300

JULY 2018
Project No. 121622042

T:\Autocad\Drawings\Project Drawings\2018\121622042\121622042_Boreholes_MTM9.dwg
2018/07/25 2:54 PM By: Briones, Gliceria

LEGEND

PROPERTY BOUNDARY
APPROXIMATE BOREHOLE LOCATION
APPROXIMATE MONITORING WELL LOCATION

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

Client/Project
TC UNITED GROUP
GEOTECHNICAL INVESTIGATION - PROPOSED BUILDING, 17-19 ROBINSON ROAD, OTTAWA, ON

Drawing No.
1

Title
BOREHOLE LOCATION PLAN

APPENDIX C

Symbols & Terms Used on the Borehole Records
2010 Borehole Records
2010 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.

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

BH10-01

CLIENT Robinson Park Development Corp. BOREHOLE No. BH10-01
LOCATION 17 Robinson Ave, Ottawa, ON PROJECT No. 163808203
DATES: BORING July 27, 2010 WATER LEVEL DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD	SPT, BLOWS/0.3m
0	60.83	Topsoil, trace organics			SS	1	230	6	6
	60.2	Dark brown silty sand with gravel FILL			SS	2	300	6	6
	59.2	Loose to dense, brownish grey silty sand with gravel (SM) TILL, wet			SS	3	420	40	40
					SS	4	350	15	15
					SS	5	460	12	12
					SS	6	450	11	11
					SS	7	475	9	9
					SS	8	400	13	13
		- occasional cobbles			SS	9	190	22	22
					SS	10	120	39	39
					SS	11	210	11	11
					SS	12	320	50/ 610mm	
	52.0	End of Borehole							

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 ●

Depth scale: 0–11 m; N scale: 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

App'd ______
Date ______

Stantec

BOREHOLE RECORD

BH10-02

CLIENT Robinson Park Development Corp.
LOCATION 17 Robinson Ave, Ottawa, ON
DATES: BORING July 27, 2010 WATER LEVEL

BOREHOLE No. BH10-02
PROJECT No. 163808203
DATUM Geodetic

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)

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	61.10					
	60.9	Topsoil, trace organics	SS	1	475	40
	60.5	360 mm - ASPHALT				
	60.2	FILL: Crushed gravel with asphalt				
1		Dark brown silty sand	SS	2	375	4
			SS	3	25	12
2	59.0	Loose to dense grey silty sand with gravel (SM) TILL, wet				
			SS	4	475	10
3			SS	5	450	4
4			SS	6	320	21
5			SS	7	400	4
			SS	8	490	7
6		- occasional cobbles	SS	9	450	24
7			SS	10	350	35
8			SS	11	300	41
	52.3		SS	12	610	40
9		End of Borehole				
10						
11						

∇ Inferred Groundwater Level
▼ Groundwater Level Measured in Standpipe

▣ Field Vane Test, kPa
□ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

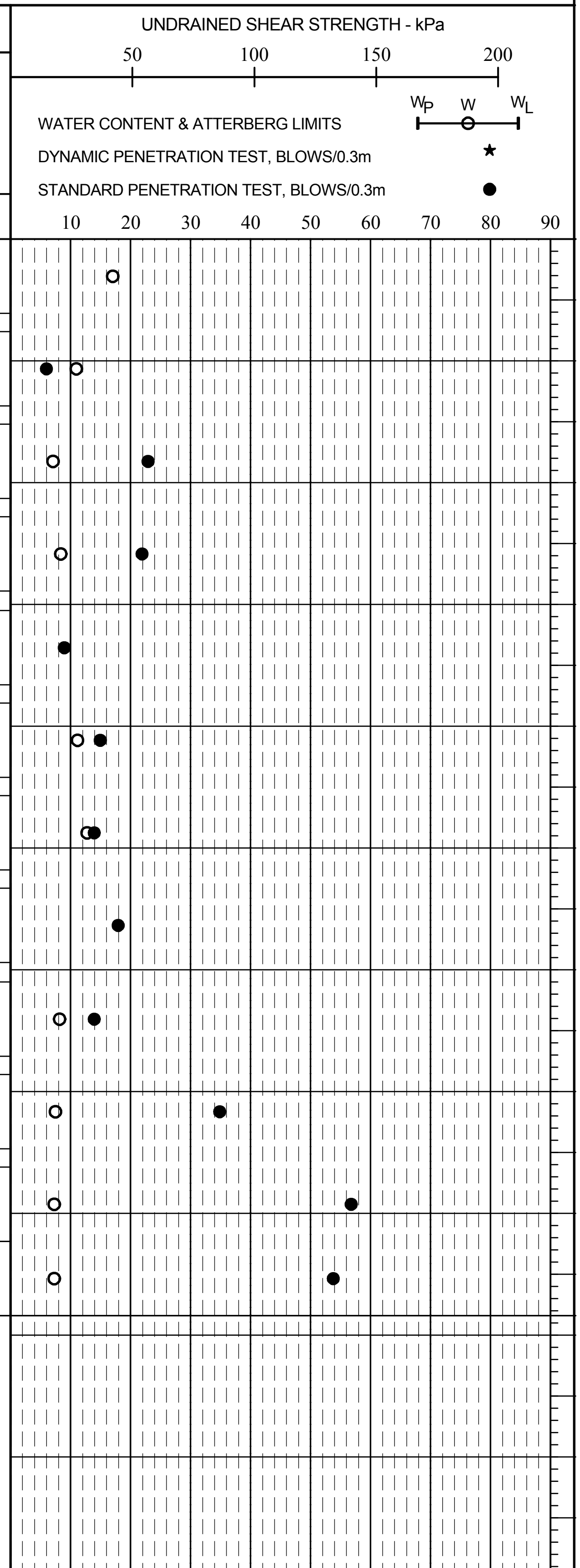
App'd ________
Date ________

BOREHOLE RECORD

BH10-03

CLIENT Robinson Park Development Corp.
LOCATION 17 Robinson Ave, Ottawa, ON
DATES: BORING July 27, 2010 WATER LEVEL
BOREHOLE No. BH10-03
PROJECT No. 163808203
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	61.22					
	61.2	40mm - ASPHALT	BS	1		
	60.6	FILL: Dark brown gravelly sand, trace organics				
1		Loose to very dense, grey silty sand with gravel (SM) TILL, moist	SS	2	340	6
2			SS	3	300	23
			SS	4	380	22
3			SS	5	250	9
4			SS	6	150	15
5			SS	7	420	14
			SS	8	25	18
6		- occasional cobbles	SS	9	50	14
7			SS	10	525	35
8			SS	11	450	57
	52.4		SS	12	560	54
9		End of Borehole				
10						
11						



Inferred Groundwater Level
Groundwater Level Measured in Standpipe

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

App'd
Date

Stantec

BOREHOLE RECORD

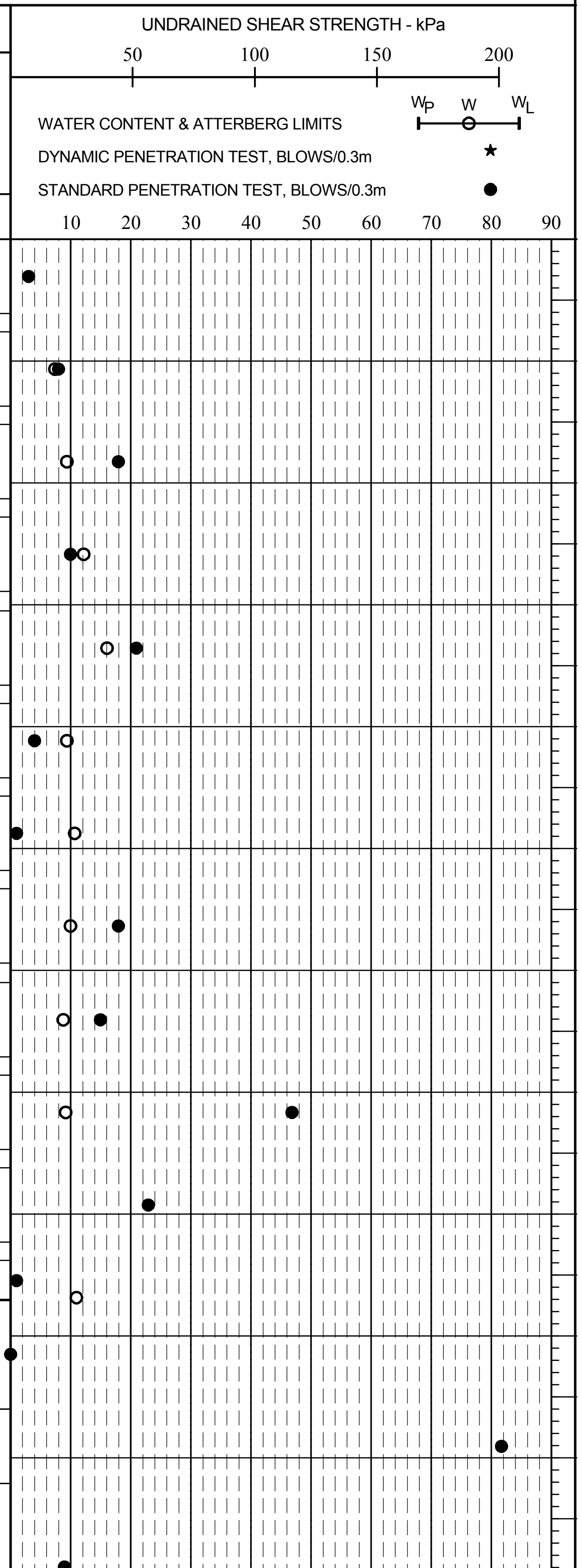
BH10-04

CLIENT: Robinson Park Development Corp.
BOREHOLE No.: BH10-04
LOCATION: 17 Robinson Ave, Ottawa, ON
PROJECT No.: 163808203
DATES: BORING July 26, 2010 WATER LEVEL
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION
0	60.94	
	60.9	FILL: crushed gravel
		FILL: Dark brown sand and gravel
	60.0	Very loose to compact, greyish brown silty sand with gravel (SM) TILL, wet
	54.8	Compact to dense, grey silty sand with gravel (SM) TILL, wet Frequent cobbles and boulders
	51.5	Fair to good grey SHALE: Bedrock - RQD and REC values for NQ15 are low due to equipment drilling

SAMPLES

TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
SS	1	100	3
SS	2	375	8
SS	3	400	18
SS	4	375	10
SS	5	400	21
SS	6	400	4
SS	7	240	1
SS	8	380	18
SS	9	400	15
SS	10	150	47
SS	11	410	23
SS	12	330	1
NQ	13	67%	0%
NQ	14	100%	82%
NQ	15	58%	9%



Inferred Groundwater Level
Groundwater Level Measured in Standpipe

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

App'd
Date

Stantec

BOREHOLE RECORD

BH10-04

CLIENT Robinson Park Development Corp.
LOCATION 17 Robinson Ave, Ottawa, ON
DATES: BORING July 26, 2010 WATER LEVEL

BOREHOLE No. BH10-04
PROJECT No. 163808203
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
11	49.4							
		End of borehole						

Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

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

App'd
Date

Stantec

BOREHOLE RECORD

BH10-05

CLIENT: Robinson Park Development Corp.
LOCATION: 17 Robinson Ave, Ottawa, ON
DATES: BORING July 28, 2010 WATER LEVEL

BOREHOLE No. BH10-05
PROJECT No. 163808203
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	61.21					
	61.2	35mm - ASPHALT	BS	1		
	61.1	Dark brown gravelly sand with organics				
1		Compact greyish brown silty sand with gravel (SM) TILL	SS	2	360	16
2			SS	3	500	13
	58.7		SS	4	240	50/ 910 mm
		End of Borehole Refusal on Inferred Boulder				

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

App'd
Date

Stantec

BOREHOLE RECORD

BH10-06

CLIENT Robinson Park Development Corp.

LOCATION 17 Robinson Ave, Ottawa, ON

DATES: BORING July 26, 2010 WATER LEVEL

BOREHOLE No. BH10-06

PROJECT No. 163808203

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	61.04					
	60.9	FILL: Crushed sandy gravel	BS	1		
		Compact brown silty sand with gravel (SM) TILL				
1			SS	2	240	13
2			SS	3	500	19
	58.7	Compact to dense, brownish grey silty sand with gravel, occasional cobbles (SM) TILL	SS	4	610	27
3			SS	5	400	18
4			SS	6	610	17
5			SS	7	400	18
6			SS	8	610	19
			SS	9	280	35
7			SS	10	430	23
8			SS	11	300	10
			SS	12	400	45
9	52.2	End of Borehole				
10						
11						

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

App'd ________

Date ________

Stantec

BOREHOLE RECORD

BH10-07

CLIENT Robinson Park Development Corp.
BOREHOLE No. BH10-07
LOCATION 17 Robinson Ave, Ottawa, ON
PROJECT No. 163808203
DATES: BORING July 28, 2010 WATER LEVEL
DATUM Geodetic

Ground surface elevation: 61.19

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	61.2	25mm - ASPHALT	BS	1		
	61.1	FILL: Dark brown gravelly sand				
1		Loose to very dense, grey silty sand with gravel (SM) TILL, moist	SS	2	320	30
2			SS	3	440	24
			SS	4	375	17
3			SS	5	475	11
4			SS	6	300	4
5			SS	7	380	7
			SS	8	530	8
6		- occasional cobbles	SS	9	450	10
7			SS	10	390	27
8			SS	11	610	28
	52.3		SS	12	500	73
9		End of Borehole				
10						
11						

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 ●

Scale: 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

App'd ______
Date ______

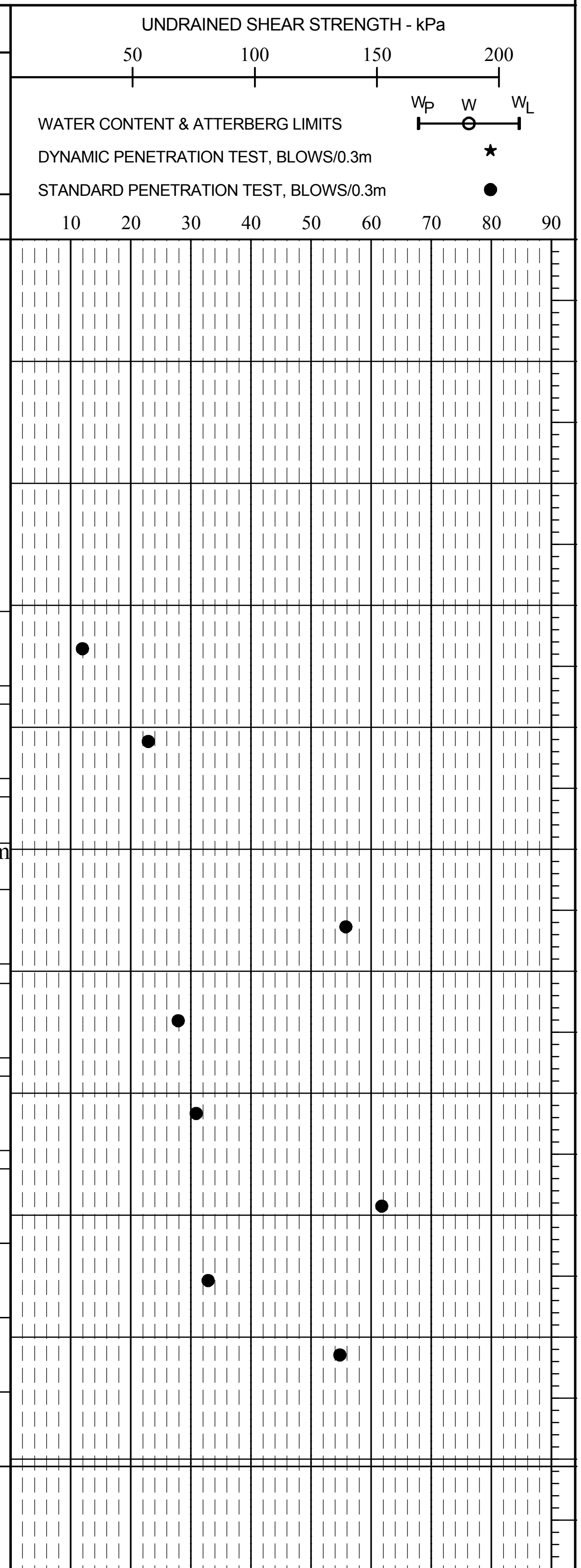
Stantec

MONITORING WELL RECORD

MW10-05A

CLIENT Robinson Park Development Corp.
LOCATION 17 Robinson Ave, Ottawa, ON
DATES: BORING July 28, 2010 WATER LEVEL August 4, 2010
BOREHOLE No. MW10-05A
PROJECT No. 163808203
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	61.21 / 61.2	35mm - ASPHALT				
		Silty sand with gravel (SM): TILL Refer to BH10-05 for soil description				
3	58.2	Compact to dense, grey silty sand with gravel (SM) TILL	SS	1	300	12
			SS	2	525	23
			SS	3	300	50/ 910mm
			SS	4	460	56
			SS	5	260	28
			SS	6	400	31
			SS	7	470	62
			SS	8	500	33
9	52.1	Weathered shale : Bedrock	SS	9	610	60/ 1520
			SS	10	250	95/ 1830
10	51.2	End of Borehole Refusal on Bedrock				



Inferred Groundwater Level
Groundwater Level Measured in Standpipe

Field Vane Test, kPa
Remoulded Vane Test, kPa
Pocket Penetrometer Test, kPa

App'd ________
Date ________

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Bedrock Core Log

Client:	Robinson Park Development Corp.	**Project No.:**	163808203
Project:	17 Robinson Avenue	**Date:**	27-Jul-10
Contractor:	Marathon Drilling	**Borehole No.:**	BH18-04
		Logger:	LEB

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
8.7 m	13	67%	0%	9.6 m	Upper portion of core consisted of Silty Sand with gravel, cobbles and boulders (TILL) , Heavily Fractured SHALE Bedrock encountered below 9.5 m											
9.6 m	14	100%	82%	10.2 m	SHALE Bedrock	R5	W3		BD	F	C	SP		SA	Unconfined compressive strengths of 56 MPa and 70 MPa measured on core samples REC and RQD values are low due to drilling disturbance	
10.2 m	15	58%	9%	11.6m	SHALE Bedrock	R5	W3		BD	F	C	SP		SA		

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

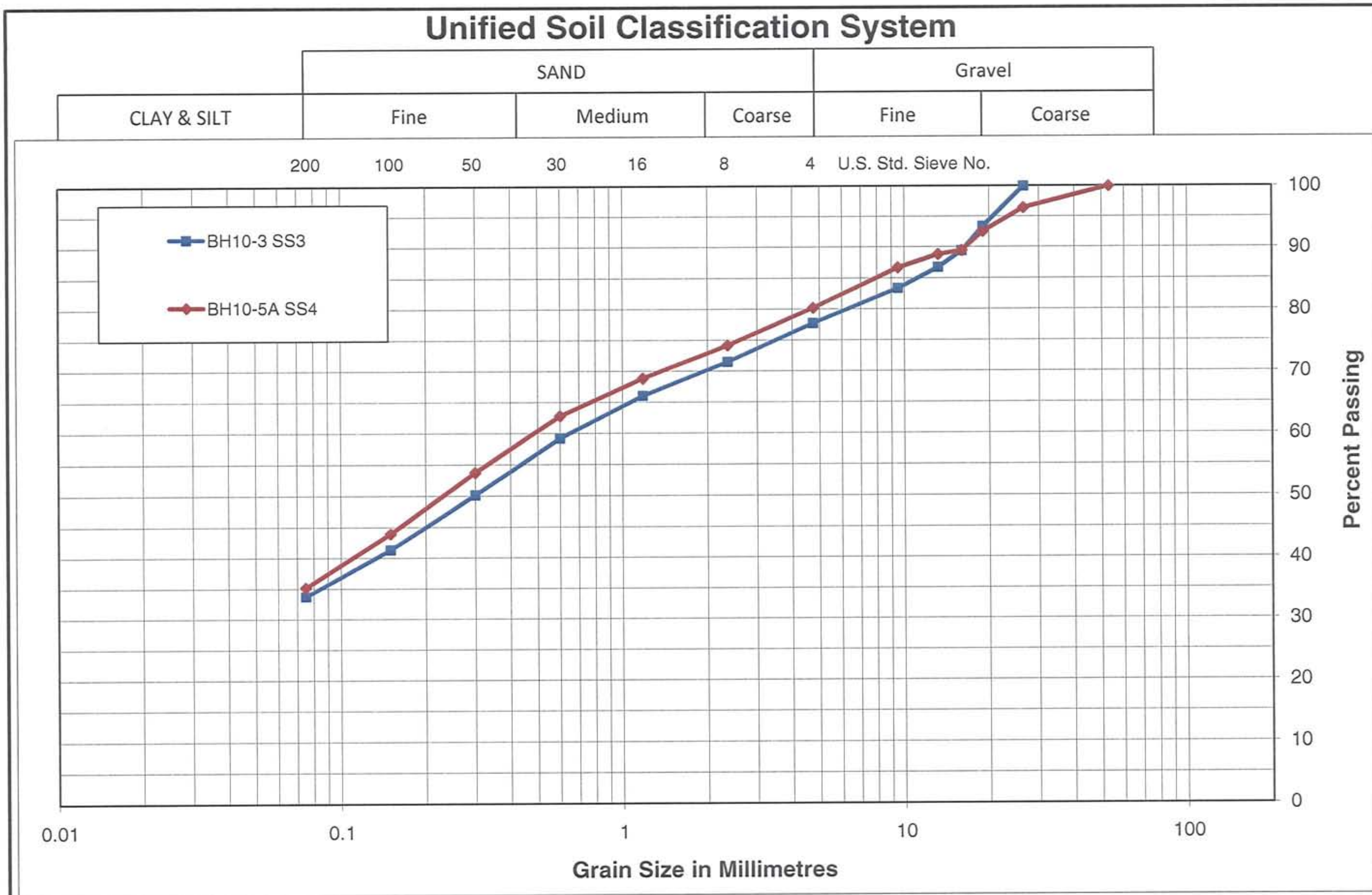
Photo No. 1: BH10-04 9.60m – 11.58m

Photo No. 2: BH10-04 9.60m – 11.58m

APPENDIX D

Laboratory Test Results

2010 Grain Size Distribution Plots



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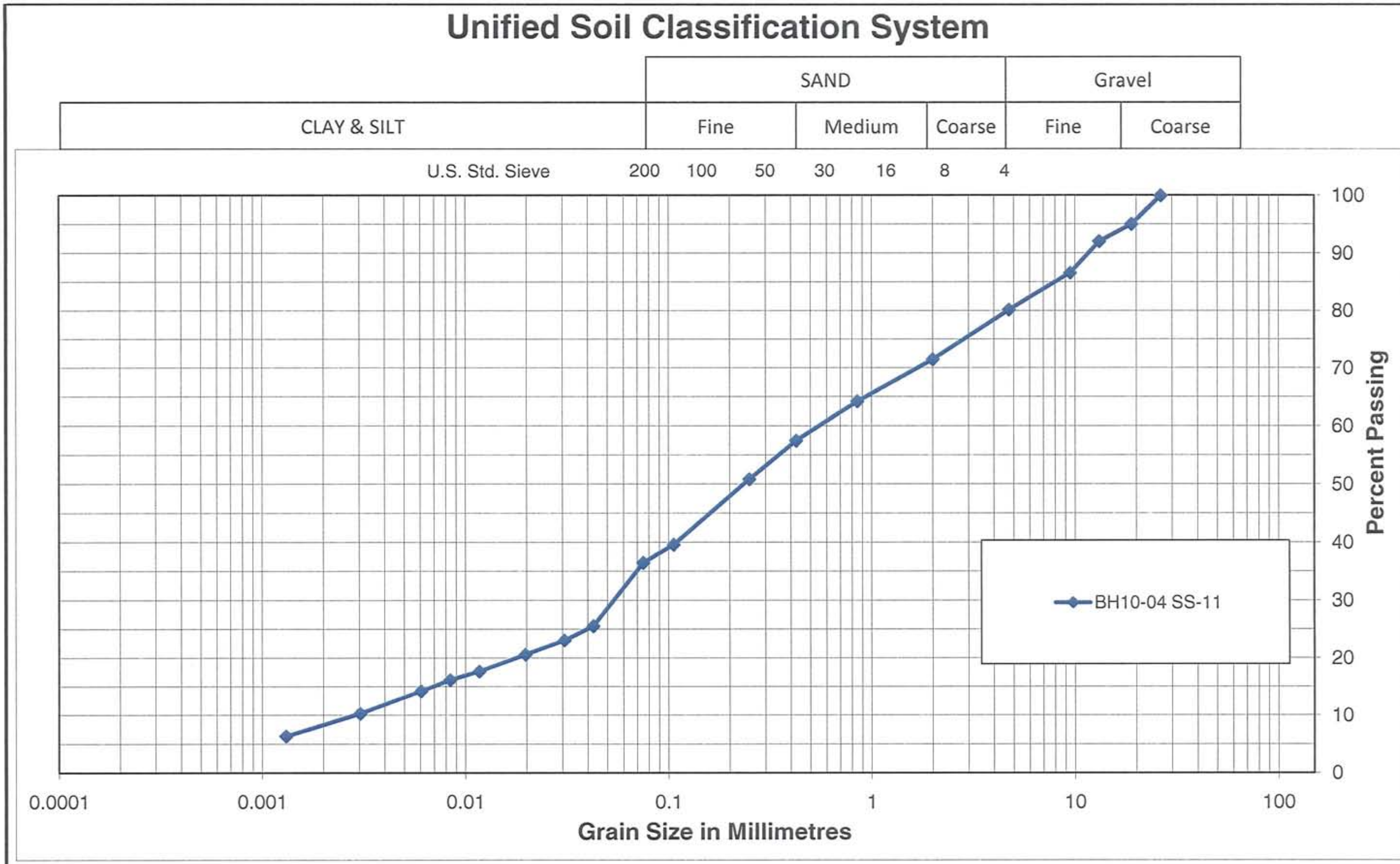
GRAIN SIZE DISTRIBUTION

Figure No. 1

Project No. 163808203



GRAIN SIZE DISTRIBUTION

Figure No. 2

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APPENDIX E

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

July 17, 2018

Site: 45.418 N, 75.6673 W   User File Reference: 17 Robinson Avenue, Ottawa, ON

Requested by: RS, 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)**
0.449	0.526	**0.441**	0.335	**0.238**	**0.118**	**0.056**	**0.015**	**0.0054**	**0.282**	**0.197**

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)	0.044	0.149	0.249
Sa(0.1)	0.061	0.187	0.301
Sa(0.2)	0.055	0.162	0.256
Sa(0.3)	0.044	0.125	0.196
Sa(0.5)	0.031	0.088	0.139
Sa(1.0)	0.015	0.044	0.070
Sa(2.0)	0.0061	0.021	0.033
Sa(5.0)	0.0012	0.0047	0.0081
Sa(10.0)	0.0006	0.0019	0.0032
PGA	0.033	0.102	0.164
PGV	0.021	0.068	0.111

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