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Geotechnical Investigation and Hydrogeological Review

Proposed Multi-Storey Building
36 Robinson Avenue
Ottawa, Ontario

Prepared For

Robinson Village LPIV Limited Partnership
c/o TC United Development

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Report PG5231-1- Revision 2

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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1.0 Introduction

Paterson Group (Paterson) was commissioned by TC United Development on behalf of Robinson Village LPIV Limited Partnership to prepare a geotechnical investigation and hydrogeological review report for the proposed multi-storey building located at 36 Robinson Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

- ☐ determine the subsurface soil and groundwater conditions by means of boreholes
- ☐ review available subsoil and groundwater information previously prepared by others for the subject site.
- ☐ provide geotechnical recommendations pertaining to the design of the proposed development, including construction considerations which may affect the design.
- ☐ Undertake a hydrogeological review to assess and manage groundwater conditions.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. This report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was carried out as a separate program and is reported under separate cover.

2.0 Proposed Development

It is understood that the proposed development will consist of a multi-storey building with two and one half levels of underground parking with the floor slab for P-3 level being at elevation 50.2 m. It is expected that the proposed structure will occupy the entire boundary of the subject site. The finished floor elevation at grade is currently set at elevation 58.92 m.

It is further understood that the proposed development will be municipally serviced with water and sewer.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field portion of the geotechnical investigation and hydrogeological review was conducted on February 21, 2020. At that time, a total of 6 test pits were completed across the subject site to a maximum depth of 8.2 m to provide general coverage of the proposed development and confirm subsoil and groundwater conditions. The test pits were conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

Relevant test holes (12 boreholes) completed by others as part of the previous subsoil and groundwater investigations have been included as part of the current geotechnical report. The approximate location of the test holes are presented on Drawing PG5231-1 - Test Hole Location Plan included in Appendix 2.

Groundwater

Monitoring wells were installed by others in 9 boreholes during the previous geotechnical investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

The test pit samples from the current geotechnical investigation will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless otherwise directed.

3.2 Field Survey

The test pit locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at each test hole location was extrapolated from the geotechnical report prepared by others which are considered approximate geodetic elevation based on the shoring drawing.

3.3 Laboratory Testing

All soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

4.0 Observations

4.1 Surface Conditions

The subject site consists of 5 contiguous properties identified as 36 Robinson Avenue. The properties are occupied by residential structures that will be demolished prior to commencing the re-development of the site. The site is relatively flat with the grade sloping downwards approximately 1 m from west to east towards the Rideau River.

The subject site is bordered to the north by Robinson Avenue, to the east, west and south by residential units.

4.2 Subsurface Profile

Overburden

Generally, the soil conditions encountered at the test hole locations consist of a fill material consisting of a mixture of silty sand, gravel, topsoil and debris.

The native soil consists of stratified sandy silt and clayey silt layers. Underlying this stratification layer is a glacial till deposit consisting of a sandy silt matrix along with occasional cobbles and boulders with trace levels of gravel. The glacial till deposit extended to the bedrock surface at approximate depths of 5.6 to 7.9 m below the existing grade.

Bedrock

Bedrock, consisting of a dark grey almost black limestone from the Eastview Formation, was cored by others at several locations during the previous investigation to a maximum depth of 16.5 m below the existing grade. The recovery values and RQD values for the bedrock cores were calculated by others during the previous investigation with recovery values varying between 50 to 100% and RQD values ranging from 0 and 100%. Based on these results, the bedrock quality varies from very poor to fair.

Some of the more recent test pits extended to bedrock to confirm the soundness of the bedrock. The hydraulic shovel was not able the to penetrate the black limestone surface.

4.3 Groundwater

Stabilized groundwater levels were measured on December 13, 2019 by others in the monitoring wells installed during the previous geotechnical field investigation. The measured groundwater level readings ranged from 1.4 to 3 m below the existing grade in the overburden wells while the bedrock wells had groundwater levels ranging from 2.8 to 4.7 m below the existing grade. It should be noted that surface water can become trapped within a backfilled boreholes that can lead to higher than typical groundwater level observations.

Based on our review of the historical monitoring wells installed at the subject site, general knowledge of the areas geology, experience with similar development projects in the immediate area in conjunction with the drawdown effect of the nearby Rideau River, it is expected that the long-term groundwater is located approximately 4 to 5 m below existing ground surface. However, it should be noted that a perched groundwater conditions was encountered in the overburden with water being trapped in the stratified layers of sandy silt and clayey silt.

The test pits excavated in February of 2020, did not identify any significant water infiltration issues within the overburden. The exception was encountered in TP-2 where minor water infiltration was noticed at the overburden and bedrock interface.

5.0 Discussion

5.1 Geotechnical Assessment

Based on the results of the geotechnical investigation, the subject site is considered satisfactory for the proposed development. The proposed multi-storey building will be founded on conventional spread footings placed within the bedrock unit.

Due to the depth of the proposed underground parking garage, a water suppression system is recommended to lessen the volume of water infiltration over the long term during post-construction.

Bedrock removal will be required to complete the lower portion of the excavation, dependent on the specific founding depths of the proposed building and elevator pits. This portion is discussed further in Subsection 5.2.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

It is expected that all the building will be demolished and all overburden will be excavated to the bedrock surface for the entire building footprint to accommodate two levels of underground parking.

Bedrock Removal

As noted above, bedrock removal will be required for the lower portion of the excavation dependent on the final founding depths of the proposed building. Hoe ramming is an option where the bedrock is weathered and/or where only small quantities of bedrock need to be removed. Where large quantities of bedrock need to be removed, line drilling and controlled blasting is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 50 mm/s during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

Construction operations could be the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

The following construction equipments could be the source of vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system will require the use of this type of equipment. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause or the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the permissible vibrations, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

If fill placement is required for grading beneath the proposed building to support the floor slab, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be approved prior to delivery to the site. The granular material should be placed in lifts no greater than 300 mm thick and compacted with suitable compaction equipment for the lift thickness. Fill placed beneath the buildings should be compacted to a minimum of 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at a minimum compacted by the heavy equipment tracks to minimize voids. If these materials are to build up the subgrade level for areas to be paved, the material should be compacted in thin lifts to a minimum density of 95% of the SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Non-specified existing fill and site-excavated soil are not suitable as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

It's expected that a mass excavation will take place and the bottom of the excavation will be relatively uniform to accept a concrete mud slab. Footings will be poured over this concrete mud slab which will also be acting as a horizontal hydraulic barrier for the water suppression system.

Concrete Hydraulic Barrier

To create a horizontal hydraulic barrier at depth, it's recommended that a concrete mud slab be placed on the bedrock surface which has been subexcavated to accommodate this additional concrete thickness. The bearing surface should be inspected by the geotechnical engineer prior to concrete placement. The concrete mud slab should consist of a 150 mm thick layer with a minimum 25 MPa compressive strength.

Bearing Resistance Values

Footings placed on the concrete mud slab overlying a sound bedrock surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **2,000 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **3,500 kPa**, incorporating a geotechnical resistance factor of 0.5.

The above noted bearing resistance value at SLS will be subjected to negligible total and differential settlements.

5.4 Design for Earthquakes

A site specific shear wave velocity test was conducted by GHD in December of 2018. According to the results of the shear wave velocity test, in our opinion and interpretation of the data, the average shear wave velocity of the 30 m profile for foundations placed on the sound bedrock surface was calculated to be greater than 1,500 m/s. Therefore, a seismic **Site Class A** is applicable for the proposed building founded directly on the bedrock surface as per Table 4.1.8.4.A of the OBC 2012. The results of the shear wave velocity test are provided in Appendix 1.

5.5 Basement Slab

For the parking garage portion, the rigid pavement structure provided in Subsection 5.8 will be used in the lower level. Fill used to backfill over the concrete mud slab to the underside of the pavement structure will consist of an OPSS Granular B Type II.

For the finished lower basement areas such as locker rooms and mechanical rooms, it’s recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone for better drainage. All backfill material within the proposed building footprint should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soil that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\, K_o\, \gamma\, H^2$, where $K_o = 0.5$ for the soil conditions noted above.
The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

The anchor should be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed building, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of shale ranges between about 40 and 50 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Grouted Rock Anchor Parameters

Parameters used to calculate grouted rock anchor lengths are provided in Table 1.

Table 1 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Shale Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Shale	60 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths are provided in Table 2. The factored tensile resistance values provided are based on a single anchor with no group influence effects.

Table 2 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2	0.8	2.8	450
	2.6	1	3.6	600
	3.2	1.2	4.4	750
	4.5	2	6.5	1000
125	1.6	0.6	2.2	600
	2	1	3	750
	2.6	1.4	4	1000
	3.2	1.8	5	1250

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Structure

For design purposes, the pavement structures presented in the following tables are recommended, where required.

Table 3 - Recommended Pavement Structure - Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over glacial till deposit.	

Table 4 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over concrete mud slab/bedrock.	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Hydrogeological Review

A test pit program was recommended and carried out on February 21, 2020 to assess groundwater conditions and infiltration volumes. Test pits were excavated to the bedrock surface to expose the overburden and determine infiltration rates and volumes that could potentially be expected during the construction phase. Based on this test pit program, two scenarios were evaluated:

☐ Management of groundwater infiltration volumes during the excavation and foundation construction phase.

☐ Long term management of groundwater infiltration at post construction.

Our findings indicate that groundwater infiltration volumes during the excavation program will be mostly encountered at the bedrock/overburden interface. With the excavation extending into the bedrock, water infiltration will continue and the groundwater will be depressurized in the bedrock until steady state is achieved . With the groundwater flow being at steady state, the groundwater entering the excavation sidewalls of the bedrock will be somewhat less than the overburden. The building design will incorporate a water suppression system that will consist of a horizontal concrete hydraulic barrier at the base of the excavation and a waterproofing membrane for the vertical surfaces. The water suppression system will reduce significantly water infiltration volumes at post construction which can be managed by the building sump pit system. The detailed information below addresses the hydrogeological issues.

6.1 Foundation Drainage and Backfill

Water Suppression System and Foundation Drainage

To manage and control groundwater water infiltration over the long term, the following water suppression system is recommended to be installed for the exterior foundation walls and underfloor drainage (refer to Figure 3 - Water Suppression System in Appendix 2 for an illustration of this system cross-section):

☐ The concrete mud slab will create a horizontal hydraulic barrier to lessen the water infiltration at the base of the excavation and will consist of a 150 mm thick layer of 25 MPa compressive strength concrete. The 150 mm minimum thickness is required to enable the support of construction traffic until the footings are poured and the area is backfilled.

❏ A waterproofing membrane will be required to lessen the effect of water infiltration for the underground parking levels starting at 3 m below finished grade (which is approximately 1 m above the high groundwater table). The waterproofing membrane will consist of bentonite panels fastened to the soldier pile and timber lagging shoring system and the grinded bedrock surface. The membrane should extend to the bottom of the excavation at the founding level of the proposed footings over the concrete mud slab. Consideration can be given to doubling the bentonite panels in the lower P2 and P3 levels where minor hydrostatic pressure will be created.

❏ A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the bottom of the foundation wall. It's expected that 150 mm diameter sleeves placed at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area. Water infiltration will result from two sources. The first will be water infiltration from the upper 3 m which is above the vertical waterproofed area. The second source will be groundwater breaching the waterproofing membrane.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration below the lowest underground parking level slab that breaches the horizontal hydraulic barrier (minimum 150 mm thick concrete mud slab). For design purposes, it's recommended that a 150 mm diameter perforated pipe be placed in each bay over the concrete hydraulic barrier. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Water Infiltration Volumes

During the construction phase, it's expected that water infiltration should have a steady state volume of less than 150,000 L/day plus any surface water infiltration following a precipitation event. The initial influx will be greater once the excavation extends below the long term groundwater level. The zone of influence associated with the temporary dewatering during the construction excavation for 2.5 levels of underground parking will be approximately 10 m.

With the water suppression system in place, it's expected that long term groundwater infiltration will be significantly reduced during post-construction. With a properly implemented water suppression system, it's expected that post-construction volumes will be less than 20,000 L/day. The zone of influence associated with the long term dewatering at post construction for 2.5 levels of underground parking will be less than 5 m.

Foundation Backfill

Where required, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I or OPSS Granular A granular material, should otherwise be used for this purpose.

Adverse Effects from Dewatering on Adjacent Structures

The temporary dewatering program during construction will have a limited zone of influence of less than 10 m from the foundation perimeter and less than 5 m at post construction. The underlying native soil below the groundwater table at the subject site is a glacial till deposit with a varying soil matrix. The dewatering of the glacial till deposit during the excavation and construction stage will not be susceptible to further consolidation since the material is compact to dense and has cobbles and boulders.

In our opinion, no adverse effects to surrounding structures and infrastructure within the nearby roadway right of way are expected.

Implementation of the water suppression system recommended above is expected to limit the drawdown of the local groundwater table over the long term and in a limited area. Therefore, in our opinion, no adverse effects to nearby structures and infrastructure are expected over the long term.

6.2 Groundwater Control

Groundwater Control for Building Construction

Given the depth of the proposed excavation below the groundwater level and the predominantly sandy soils encountered overlying the bedrock, groundwater infiltration into the excavation is anticipated to be moderate to high. It is therefore recommended that the shoring system consist of a secant pile wall which is socketed into the bedrock in order to act as a cofferdam.

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes, being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater breaching the waterproofing system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (less than 20,000 L/day) which includes higher volumes during peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

7.0 Design and Construction Precautions

7.1 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover, or a minimum of 0.6 m of soil cover in conjunction with foundation insulation, should be provided.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

It is expected that the foundations will generally not require protection against frost action due to the founding depth. However, unheated structures, such as the access ramp, may require insulation against the deleterious effect of frost action.

7.2 Excavation

It’s expected that temporary shoring will be required due to the proposed depth for the underground parking levels. Furthermore, it’s expected that the foundation walls will be blind poured against the shoring system.

Excavation Side Slopes for Servicing and Shallow Excavations

The subsoil at this site is considered to be mainly a Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below the groundwater level.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.
It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Temporary shoring will be required due to the depth of the excavation, the proximity of the adjacent structures and underground services. Due to the glacial till deposit and the excavation below the bedrock surface, it's assumed that the temporary shoring will consist of drilled soldier piles and timber lagging system. Temporary shoring will be required to support the overburden for the entire perimeter of the excavation.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration, a full hydrostatic condition which can occur during significant precipitation events.

For design purposes, the temporary system will most likely consist of a drilled soldier pile and timber lagging system. Drilled soldier piles will be required to penetrate through expected occasional boulders and bedrock. These systems can be anchored or braced. Generally, it is expected that the shoring system will be provided with tie-back anchors to ensure their stability and greater safety.

Typical Geotechnical Parameters

Generally, it is expected that the shoring systems will be provided with tie-back anchors to ensure their stability.

The geotechnical design of grouted anchors is based upon two possible failure modes. The anchor can fail either by shear failure along the grout interface or by pullout of a 60 to 90 degree cone with the apex of the cone near the middle of the bonded length of the anchor.

The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 5 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible.

The dry unit weight should be used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of $0.65 \cdot K \cdot \gamma \cdot H$ for strutted or anchored shoring, or a triangular earth pressure distribution with a maximum value of $K \cdot \gamma \cdot H$ for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible. The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

It should be noted that some relief of hydrostatic pressure is anticipated with the implementation of the above noted water suppression system.

7.3 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil/bedrock subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

7.4 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The excavations may be completed in proximity of existing structures which could be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions which could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

7.5 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of an aggressive to very aggressive corrosive environment.

8.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation services program including the following aspects be performed by the geotechnical consultant.

❑ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.

❑ Observe and approve the installation of the water suppression system.

❑ Review proposed waterproofing and foundation drainage design and requirements.

❑ Observation of all bearing surfaces prior to the placement of concrete.

❑ Sampling and testing of the concrete and fill materials used.

❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

❑ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.

❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

9.0 Statement of limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Robinson Village LPIV Limited Partnership and TC United Development or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 12, 2020
C. P. DA SILVA
PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng., ing., QP_{ESA}

Report Distribution

- ☐ TC United Development (3 copies)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS AND NOTES ON BOREHOLES BY OTHERS

MULTI-CHANNEL ANALYSIS AND SURFACE WAVE RESULTS BY OTHERS

LABORATORY TESTING RESULTS BY OTHERS

CORE LOG PHOTOGRAPHS BY OTHERS

BUILDING CODE SEISMIC HAZARD CALCULATIONS BY OTHERS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
36 Robinson Avenue
Ottawa, Ontario

DATUM Approximate elevations obtained from others.

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 February 21

FILE NO. PG5231

HOLE NO. TP 1-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	59.40
FILL: Brown silty sand, trace gravel, organics and debris							
						1	58.40
1.40		G	1				
GLACIAL TILL: Brown silty sand, occasional cobbles and obulders, some clay, trace gravel		G	2				
						2	57.40
						3	56.40
- grey by 3.2 m depth							
		G	3				
						4	55.40
						5	54.40
						6	53.40
6.20							
End of Test Pit Refusal to excavation on bedrock surface @ 6.2m depth (TP dry upon completion)							

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

Piezometer Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
36 Robinson Avenue
Ottawa, Ontario

DATUM Approximate elevations obtained from others.

REMARKS

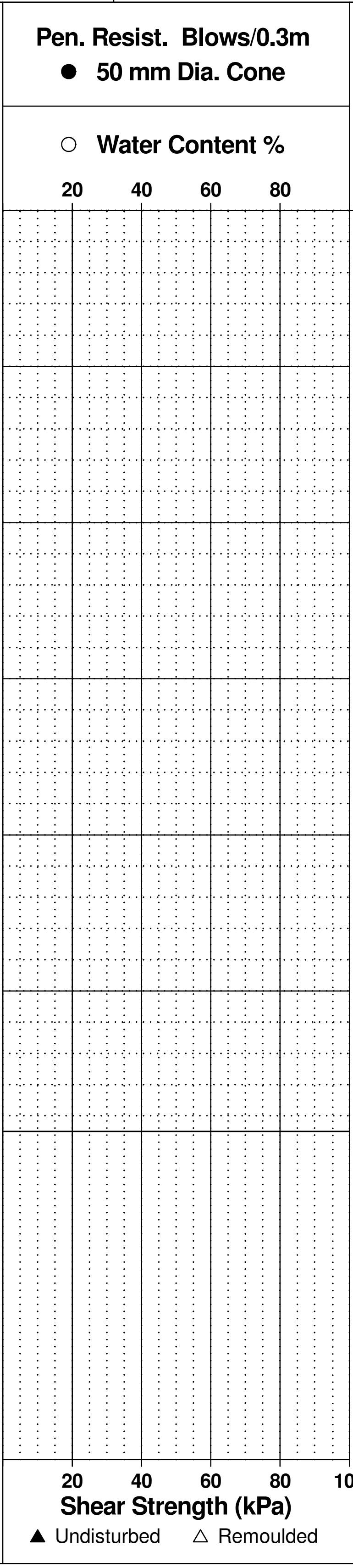
BORINGS BY Hydraulic Shovel

DATE 2020 February 21

FILE NO. PG5231

HOLE NO. TP 2-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	58.55	
FILL: Brown silty sand, trace organics, gravel and debris						1	57.55	
		G	1					
2.00						2	56.55	
GLACIAL TILL: Brown sandy silt, some clay, trace gravel, occasional cobbles and boulders		G	2					
- grey by 2.9m depth						3	55.55	
		G	3					
						4	54.55	
						5	53.55	
								∇
5.90								
End of Test Pit Refusal to excavation on bedrock surface @ 5.9m depth (Open hole GWL @ 5.4m depth)								



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
36 Robinson Avenue
Ottawa, Ontario

DATUM Approximate elevations obtained from others.

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 February 21

FILE NO. PG5231

HOLE NO. TP 3-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	58.60		
FILL: Brown silty sand, trace gravel, organics and brick						1	57.60		
		G	1						
2.00						2	56.60		
GLACIAL TILL: Brown silty clay, some sand and gravel		G	2						
2.30									
End of Test Pit (TP dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
36 Robinson Avenue
Ottawa, Ontario

DATUM Approximate elevations obtained from others.

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 February 21

FILE NO. PG5231

HOLE NO. TP 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	58.30		
FILL: Brown silty sand, trace gravel, organics and debris						1	57.30		
		G	1						
2.10						2	56.30		
GLACIAL TILL: Brown clayey silt, some sand, trace gravel 2.30		G	2						
End of Test Pit (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
36 Robinson Avenue
Ottawa, Ontario

DATUM Approximate elevations obtained from others.

FILE NO. **PG5231**

REMARKS

HOLE NO. **TP 5-20**

BORINGS BY Hydraulic Shovel

DATE 2020 February 21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	58.95		
FILL: Brown silty sand, trace gravel, organics and debris	▨					1	57.95		
	▨	G	1						
2.00						2	56.95		
GLACIAL TILL: Brown clayey silt, some sand and gravel 2.20	^^^	G	2						
End of Test Pit (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
36 Robinson Avenue
Ottawa, Ontario

DATUM Approximate elevations obtained from others.

FILE NO. PG5231

REMARKS

HOLE NO. TP 6-20

BORINGS BY Hydraulic Shovel **DATE** 2020 February 21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	58.90		
FILL: Brown silty sand, trace gravel and debris						1	57.90		
1.50									
GLACIAL TILL: Brown clayey silt, some sand and gravel		G	1			2	56.90		
2.40									
GLACIAL TILL: Grey sandy silt, some clay, trace gravel		G	2			3	55.90		
						4	54.90		
						5	53.90		
5.80		G	3						▽
End of Test Pit Refusal to excavation on bedrock surface @ 5.8m depth (Open hole GWL @ 5.74m depth)									

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation 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 layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

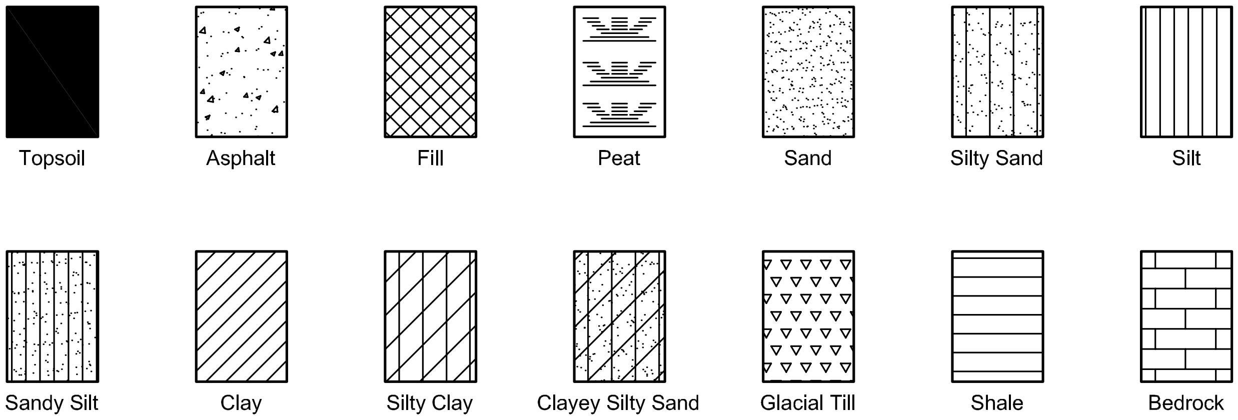
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

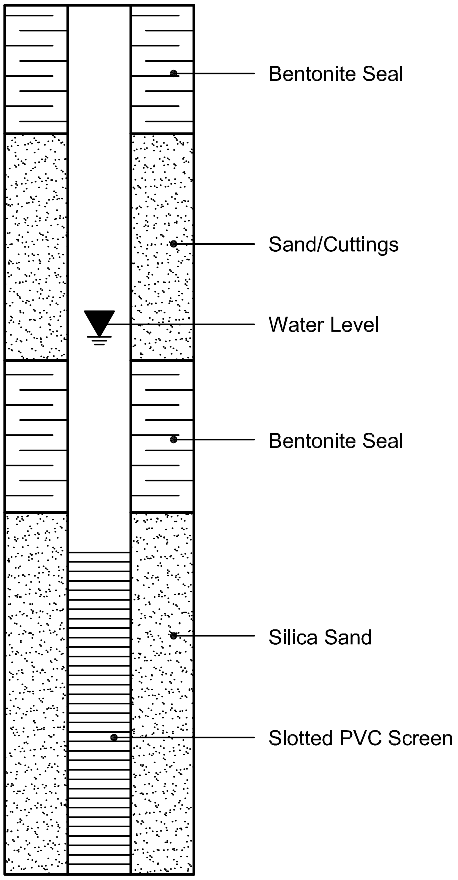
SYMBOLS AND TERMS (continued)

STRATA PLOT

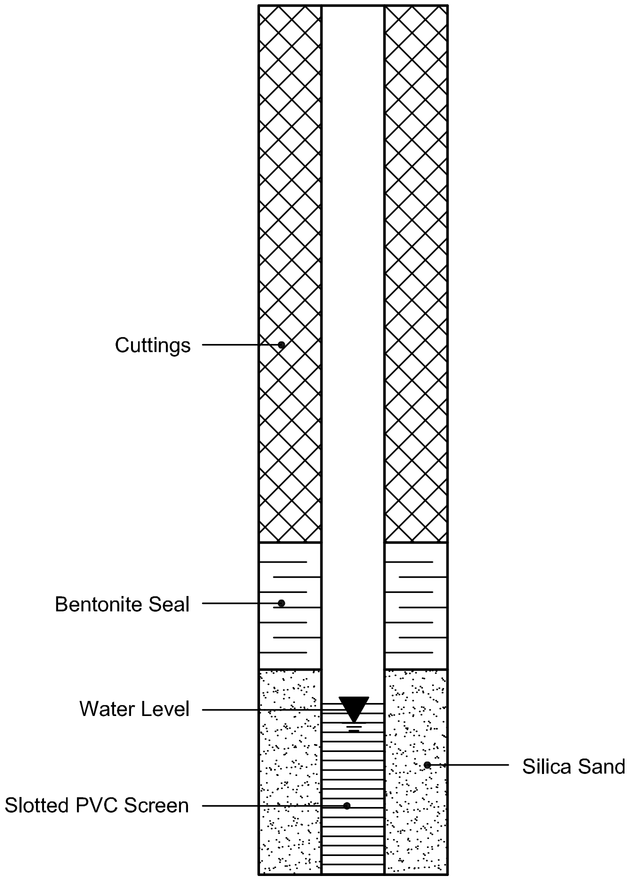


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



REFERENCE No.: 11186719

ENCLOSURE No.: 1

BOREHOLE No.: BH1

ELEVATION: 58.30 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

DESCRIBED BY: R. Vanden Tillaart

CHECKED BY: B. Vazhbakht

DATE (START): 21 January 2019

DATE (FINISH): 21 January 2019

LEGEND

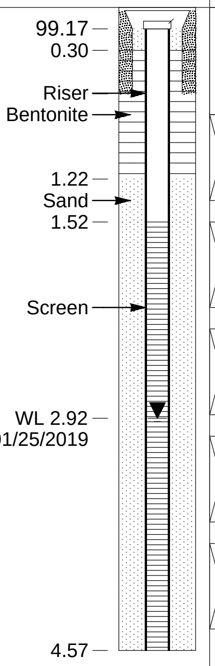
☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
	S	Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

STRATIGRAPHY

Depth BGS (meters)	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK
	58.30	GROUND SURFACE
		FILL - Silty sand, some clay, trace gravel, brown, moist, loose Upper 0.6 m frozen
	56.6	**SILTY SAND**- some clay, some gravel, brown, moist to saturated, compact to dense
	55.3	**SANDY SILT**- some clay, trace gravel, grey, saturated, compact
	53.7	Borehole terminated at 4.6 mbgs

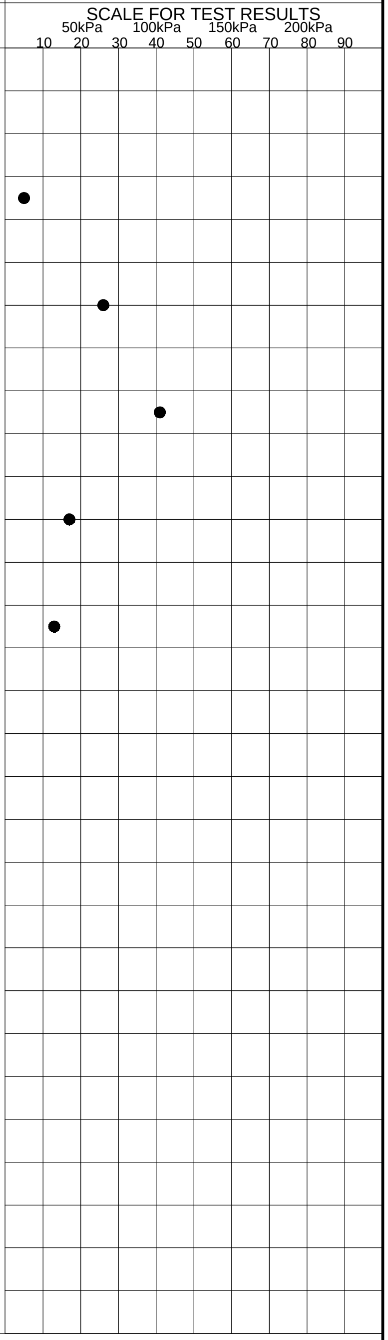
Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0

MONITOR WELL



SAMPLE DATA

State	Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
☒	SS1	25	11	5
☒	SS2	63	25	26
☒	SS3	100	11	41
☒	SS4	100	12	17
☒	SS5	71	9	13



NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

REFERENCE No.: 11186719 ENCLOSURE No.: 2

BOREHOLE No.: **BH2**

ELEVATION: 58.60 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

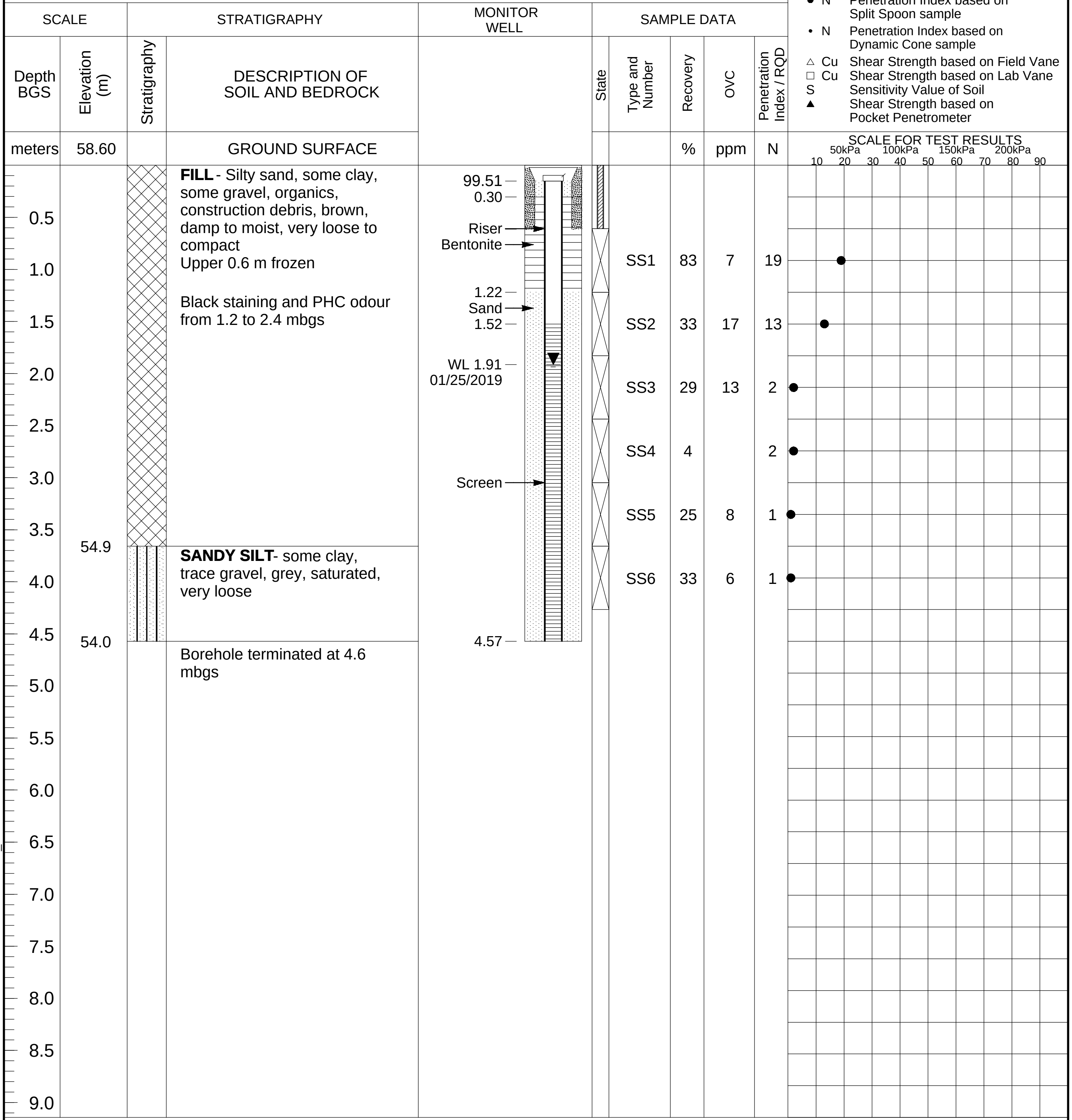
LOCATION: 36 Robinson Avenue

DESCRIBED BY: R. Vanden Tillaart CHECKED BY: B. Vazhbakht

DATE (START): 21 January 2019 DATE (FINISH): 21 January 2019

LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
	S	Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer



Depth BGS	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK
meters	58.60	GROUND SURFACE
		FILL - Silty sand, some clay, some gravel, organics, construction debris, brown, damp to moist, very loose to compact Upper 0.6 m frozen Black staining and PHC odour from 1.2 to 2.4 mbgs
	54.9	**SANDY SILT**- some clay, trace gravel, grey, saturated, very loose
	54.0	Borehole terminated at 4.6 mbgs

Type and Number	Recovery	OVC	Penetration Index / RQD
	%	ppm	N
SS1	83	7	19
SS2	33	17	13
SS3	29	13	2
SS4	4		2
SS5	25	8	1
SS6	33	6	1

NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

REFERENCE No.: 11186719 ENCLOSURE No.: 3

BOREHOLE No.: BH3

ELEVATION: 59.40 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

DESCRIBED BY: R. Vanden Tillaart CHECKED BY: B. Vazhbakht

DATE (START): 22 January 2019 DATE (FINISH): 22 January 2019

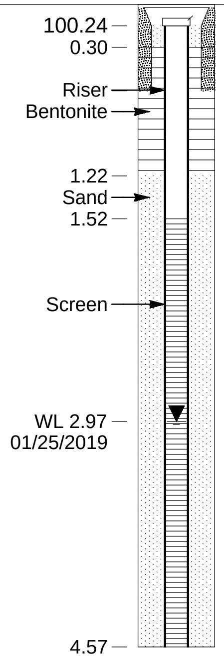
LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
⊢⊣		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
	S	Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

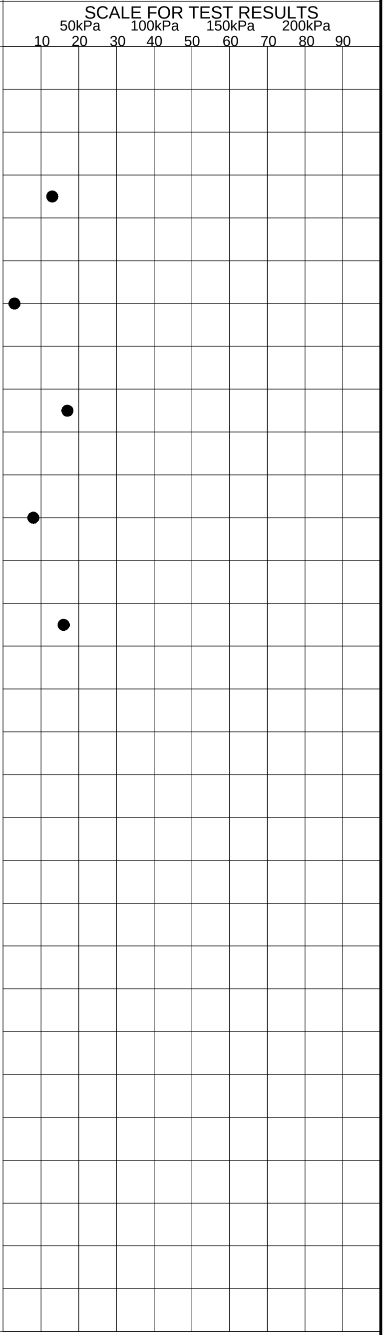
SCALE		STRATIGRAPHY		SAMPLE DATA				
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	OVC	Penetration Index / RQD
meters	59.40		GROUND SURFACE			%	ppm	N
		☒	**FILL** - Silty sand, some gravel, brown, moist, very loose to compact Upper 0.6 m frozen					
1.0				☒	SS1	67	28	13
2.0				☒	SS2	4		3
	57.1		**SILTY SAND**- some clay, some gravel, grey to brown, saturated, loose to compact					
2.5				☒	SS3	58	42	17
3.5				☒	SS4	58	26	8
4.0				☒	SS5	71		16
4.5	54.8		Borehole terminated at 4.6 mbgs					

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0

MONITOR WELL



SCALE FOR TEST RESULTS



NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

BOREHOLE LOG 11186719-A1-BOREHOLE LOGS.GPJ INSPEC_SOL.GDT 16/12/19

REFERENCE No.: 11186719 ENCLOSURE No.: 4

BOREHOLE No.: BH4

ELEVATION: 58.70 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

DESCRIBED BY: R. Vanden Tillaart CHECKED BY: B. Vazhbakht

DATE (START): 22 January 2019 DATE (FINISH): 22 January 2019

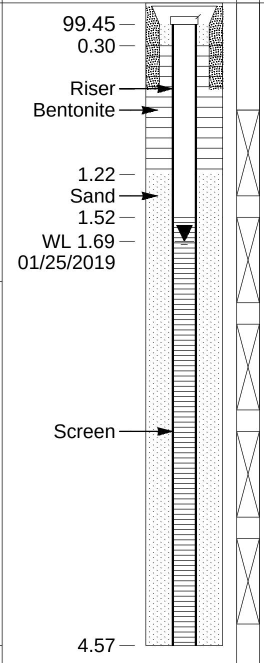
LEGEND

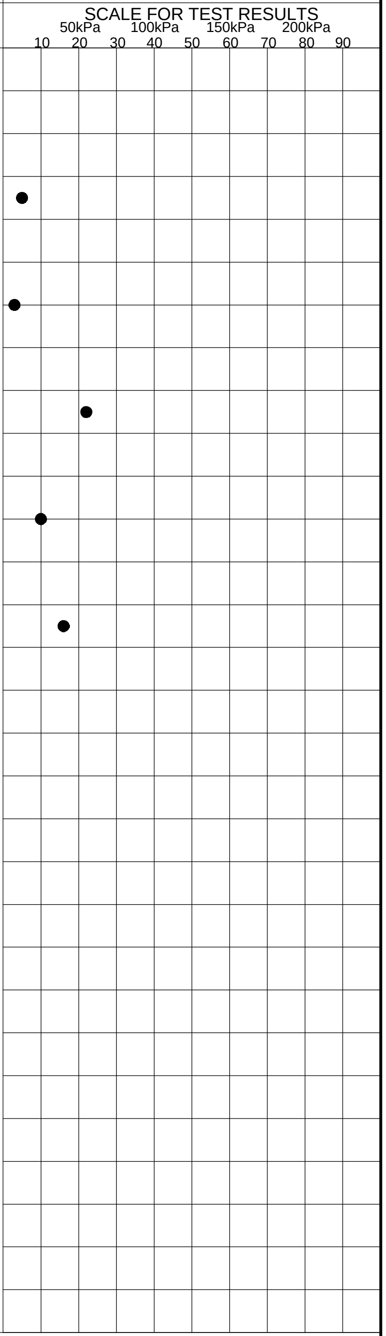
☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY		SAMPLE DATA				
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	OVC	Penetration Index / RQD
meters	58.70		GROUND SURFACE			%	ppm	N
		▨	**FILL** - Silty sand, some gravel, brown, moist, very loose to loose					
				☒	SS1	4		5
			Black staining, PHC odour at 1.5 mbgs	☒	SS2	54	19	3
	56.7	▥	**SILTY SAND**- some gravel, some clay, grey to brown, moist to saturated, compact					
				☒	SS3	63	28	22
				☒	SS4	58	17	10
				☒	SS5	88	14	16
	54.1		Borehole terminated at 4.6 mbgs					

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0

MONITOR WELL





NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

REFERENCE No.: 11186719

ENCLOSURE No.: 5

BOREHOLE No.: BH5

ELEVATION: 58.30 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

DESCRIBED BY: S. Wheeler

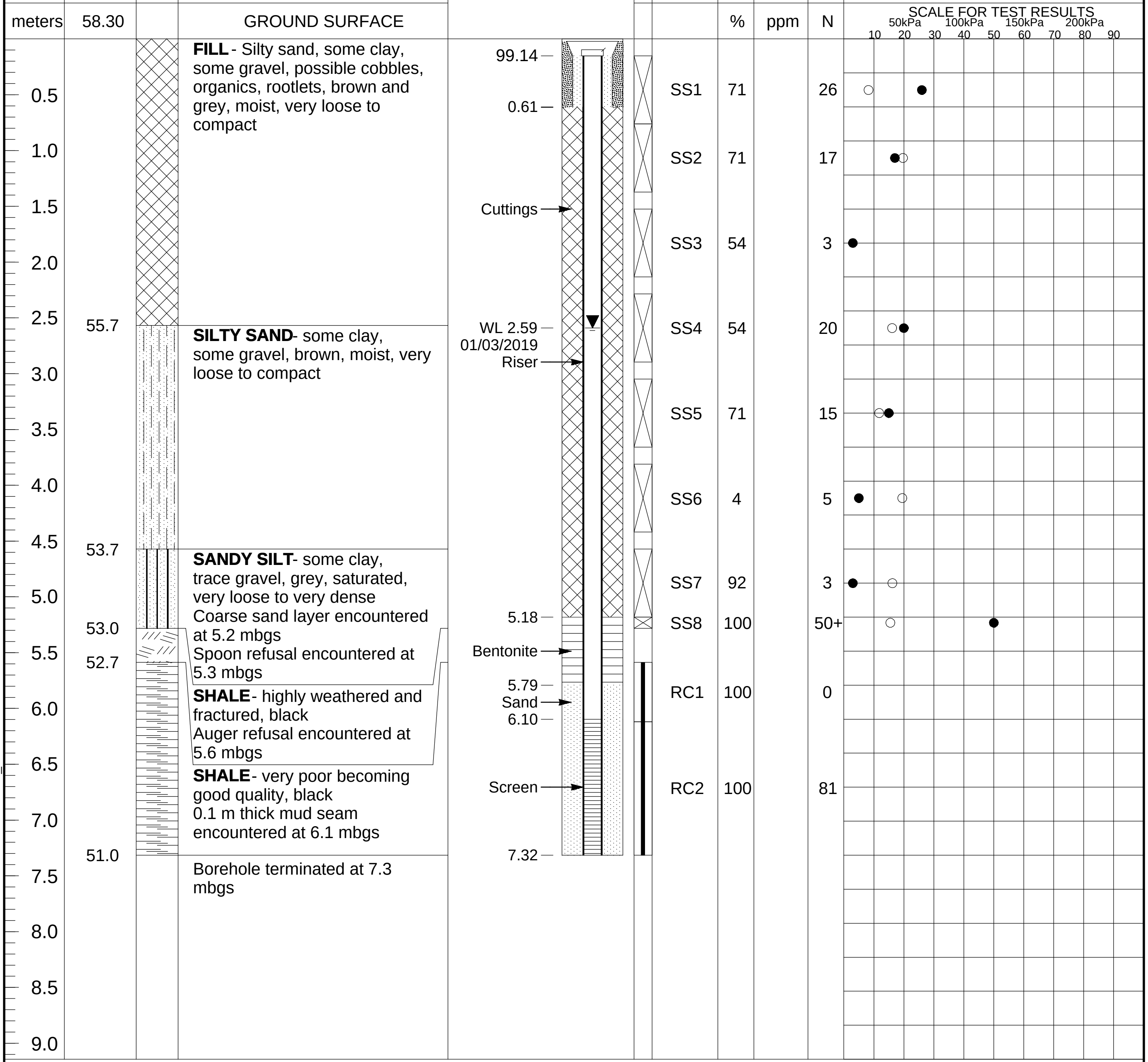
CHECKED BY: B. Vazhbakht

DATE (START): 17 December 2018

DATE (FINISH): 17 December 2018

LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer



Depth BGS (meters)	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK
	58.30	GROUND SURFACE
		FILL - Silty sand, some clay, some gravel, possible cobbles, organics, rootlets, brown and grey, moist, very loose to compact
	55.7	**SILTY SAND**- some clay, some gravel, brown, moist, very loose to compact
	53.7	**SANDY SILT**- some clay, trace gravel, grey, saturated, very loose to very dense Coarse sand layer encountered at 5.2 mbgs Spoon refusal encountered at 5.3 mbgs
	53.0	
	52.7	**SHALE**- highly weathered and fractured, black Auger refusal encountered at 5.6 mbgs
		SHALE- very poor becoming good quality, black 0.1 m thick mud seam encountered at 6.1 mbgs
	51.0	Borehole terminated at 7.3 mbgs

MONITOR WELL:

- 99.14
- 0.61
- Cuttings
- WL 2.59 01/03/2019
- Riser
- 5.18
- Bentonite
- 5.79
- Sand
- 6.10
- Screen
- 7.32

State	Type and Number	Recovery %	OVC ppm	Penetration Index / RQD N
☒	SS1	71		26
☒	SS2	71		17
☒	SS3	54		3
☒	SS4	54		20
☒	SS5	71		15
☒	SS6	4		5
☒	SS7	92		3
☒	SS8	100		50+
	RC1	100		0
	RC2	100		81

NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

BOREHOLE LOG 11186719-A1-BOREHOLE LOGS.GPJ INSPEC_SOL.GDT 16/12/19

REFERENCE No.: 11186719 ENCLOSURE No.: 6

BOREHOLE No.: BH6
ELEVATION: 59.40 m

BOREHOLE LOG

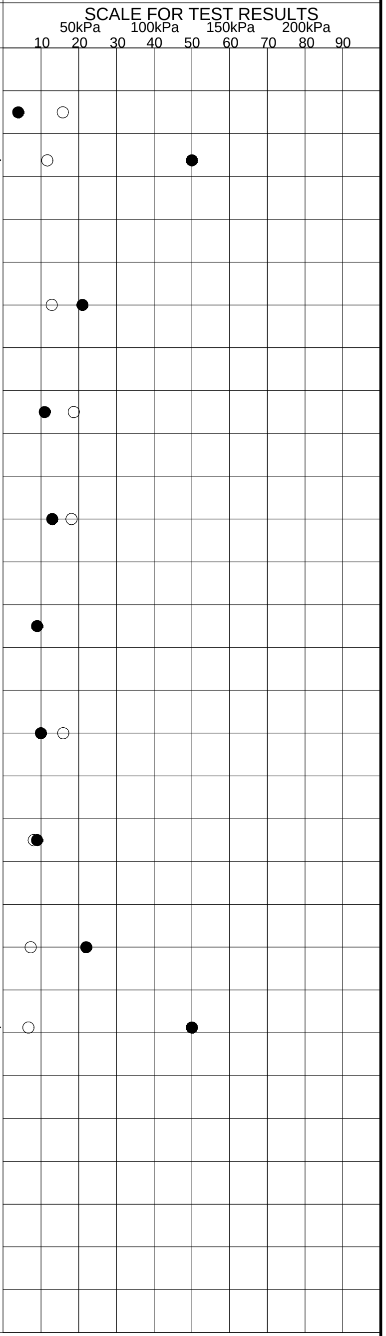
Page: 1 of 1

CLIENT: TC United Group
PROJECT: Geotechnical Investigation
LOCATION: 36 Robinson Avenue
DESCRIBED BY: S. Wheeler CHECKED BY: B. Vazhbakht
DATE (START): 17 December 2018 DATE (FINISH): 17 December 2018

LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY	SAMPLE DATA			
Depth BGS	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK	Type and Number	Recovery	OVC	Penetration Index / RQD
meters	59.40	GROUND SURFACE		%	ppm	N
		FILL - Silty sand, some clay, trace gravel, dark brown, moist, loose to very dense, possible cobbles				
0.5			SS1	58		4
			SS2	33		50+
1.0						
1.5						
			SS3	75		21
2.0						
	57.1	**SILTY SAND**- some clay becoming clayey, trace to some gravel, grey and brown, saturated, loose to compact				
2.5			SS4	83		11
3.0						
			SS5	88		13
3.5						
4.0						
			SS6	0		9
4.5						
			SS7	83		10
5.0						
5.5			SS8	100		9
6.0						
			SS9	100		22
6.5						
		Spoon refusal encounterd at 7.1 mbgs Auger refusal encountered at 7.2 mbgs	SS10	33		50+
7.0						
	52.3 52.2	**SHALE**- black, highly weathered and fractured				
		SHALE- black, excellent quality	RC1	100		92
7.5						
8.0						
			RC2	100		96
8.5						
	50.6					
9.0		Borehole terminated at 8.9 mbgs				



NOTES:
Monitoring well could not be installed due to the existence of a saturated sand layer.
Borehole backfilled with sand, bentonite and auger cuttings.
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

REFERENCE No.: 11186719

ENCLOSURE No.: 7

BOREHOLE No.: BH7

ELEVATION: 59.10 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

DESCRIBED BY: D.Cooper

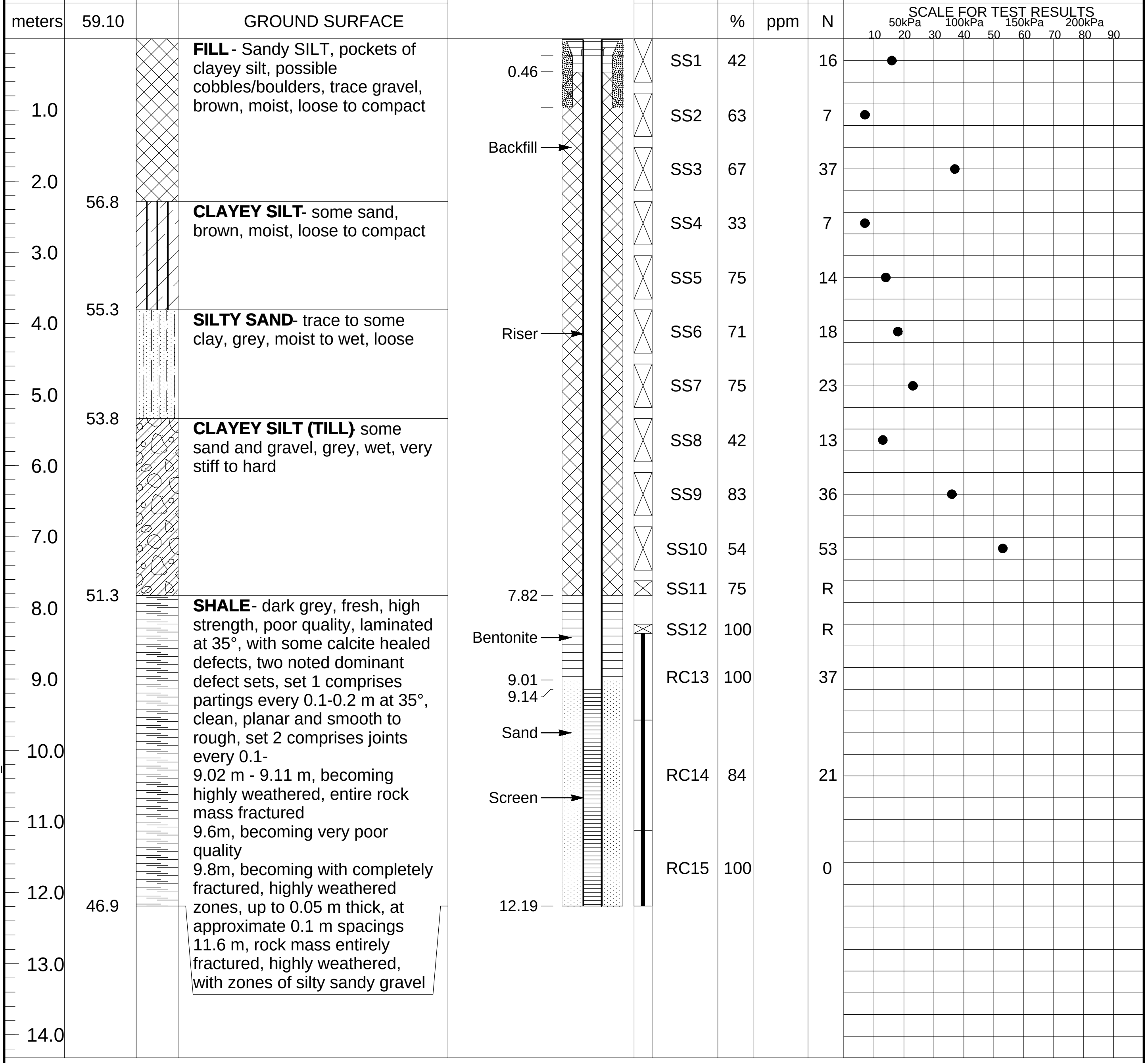
CHECKED BY: B. Vazhbakht

DATE (START): 18 November 2019

DATE (FINISH): 18 November 2019

LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
⊢⊣		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer



SCALE		STRATIGRAPHY		
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	
meters	59.10		GROUND SURFACE	
		☒	**FILL** - Sandy SILT, pockets of clayey silt, possible cobbles/boulders, trace gravel, brown, moist, loose to compact	
	56.8		**CLAYEY SILT**- some sand, brown, moist, loose to compact	
	55.3		**SILTY SAND**- trace to some clay, grey, moist to wet, loose	
	53.8		**CLAYEY SILT (TILL)**- some sand and gravel, grey, wet, very stiff to hard	
	51.3		**SHALE**- dark grey, fresh, high strength, poor quality, laminated at 35°, with some calcite healed defects, two noted dominant defect sets, set 1 comprises partings every 0.1-0.2 m at 35°, clean, planar and smooth to rough, set 2 comprises joints every 0.1- 9.02 m - 9.11 m, becoming highly weathered, entire rock mass fractured 9.6m, becoming very poor quality 9.8m, becoming with completely fractured, highly weathered zones, up to 0.05 m thick, at approximate 0.1 m spacings 11.6 m, rock mass entirely fractured, highly weathered, with zones of silty sandy gravel	
	46.9			

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0

MONITOR WELL

0.46

Backfill

Riser

7.82

Bentonite

9.01

9.14

Sand

Screen

12.19

SAMPLE DATA

State	Type and Number	Recovery %	OVC ppm	Penetration Index / RQD N
☒	SS1	42		16
☒	SS2	63		7
☒	SS3	67		37
☒	SS4	33		7
☒	SS5	75		14
☒	SS6	71		18
☒	SS7	75		23
☒	SS8	42		13
☒	SS9	83		36
☒	SS10	54		53
☒	SS11	75		R
☒	SS12	100		R
	RC13	100		37
	RC14	84		21
	RC15	100		0

NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

REFERENCE No.: 11186719

ENCLOSURE No.: 8

BOREHOLE No.: BH8

ELEVATION: 58.60 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

DESCRIBED BY: D.Cooper

CHECKED BY: B. Vazhbakht

DATE (START): 18 November 2019

DATE (FINISH): 18 November 2019

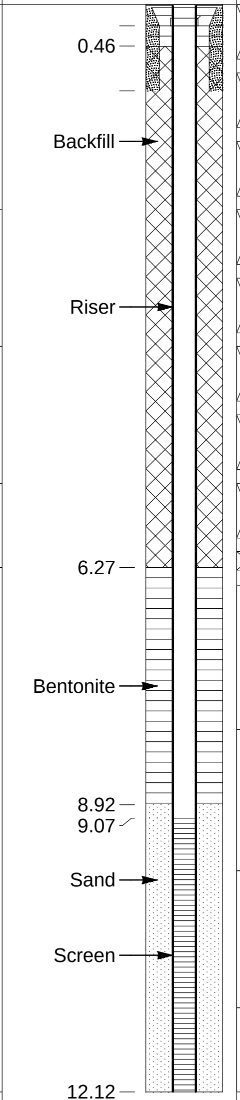
LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK
	58.60	GROUND SURFACE
		FILL - Silty sand, with some gravel, presence of brick fragments, organic pockets, brown, moist, loose to compact
	56.3	**CLAYEY SILT**- some gravel and sand, brown, saturated, compact to dense
	54.8	**SANDY SILT**- some gravel, trace of clay, grey, saturated, loose to compact
	53.3	**SANDY GRAVEL**- some silt, grey, saturated, compact to very dense becoming glacial till
	52.3	**SHALE**- dark grey, laminated at 0-5°, fresh, medium to high strength, fair quality, only defect set comprises partings at 0-5° 8.01 m, becoming excellent quality 9.65 m, becoming fair quality 11.2 m, becoming excellent quality
	46.5	

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0

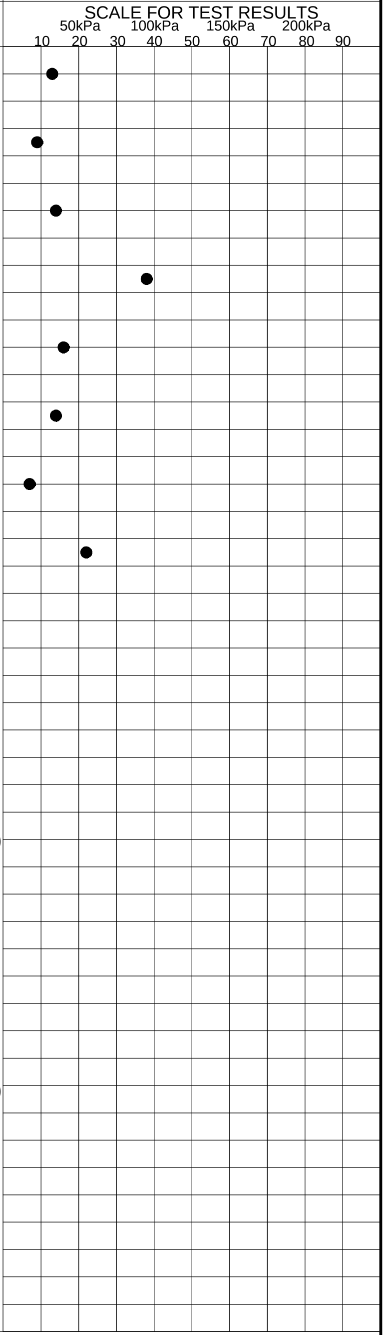
MONITOR WELL



SAMPLE DATA

Type and Number	Recovery (%)	OVC (ppm)	Penetration Index / RQD (N)
SS1	96		13
SS2	92		9
SS3	63		14
SS4	46		38
SS5	71		16
SS6	83		14
SS7	79		7
SS8	25		22
SS9	100		R
RC10	100		57
RC11	100		100
RC12	92		70
RC13	100		100

SCALE FOR TEST RESULTS



NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

REFERENCE No.: 11186719

ENCLOSURE No.: 9

BOREHOLE No.: BH9

ELEVATION: 59.10 m

BOREHOLE LOG

Page: 1 of 2

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

DESCRIBED BY: D.Cooper

CHECKED BY: B. Vazhbakht

DATE (START): 9 December 2019

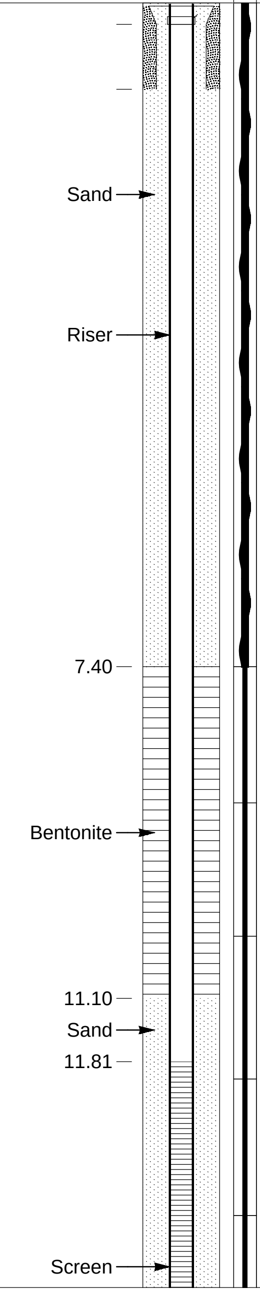
DATE (FINISH): 9 December 2019

LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
	S	Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY	MONITOR WELL	SAMPLE DATA					SCALE FOR TEST RESULTS
Depth BGS	Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK		State	Type and Number	Recovery	OVC	Penetration Index / RQD	50kPa 100kPa 150kPa 200kPa 10 20 30 40 50 60 70 80 90
meters	59.10	GROUND SURFACE				%	ppm	N	
		Augered to practical refusal with no sampling							
7.40	51.7	**SHALE**- Highly weathered and fractured, dark grey to black, fresh with completely weathered zones, thinly laminated at 20-30°. Dominant defects are partings at approximately 50 mm spacing Approximately 50mm seam of crushed rock			RC1	87		9	
					RC2	100		50	
		Abundantly fractured, fresh with completely weathered crushed seams			RC3	92		14	
		Fresh, laminated at approximately 30°. Dominant defects every 50 to 100 mm are partings, occasional calcite coatings, minor crushed seams (<5 mm) comprising of silty clayey sand every 300 mm. becoming more fractured			RC4	100		44	

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0



NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

REFERENCE No.: 11186719 ENCLOSURE No.: 9

BOREHOLE No.: BH9

ELEVATION: 59.10 m

BOREHOLE LOG

Page: 2 of 2

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

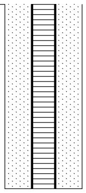
DESCRIBED BY: D.Cooper CHECKED BY: B. Vazhbakht

DATE (START): 9 December 2019 DATE (FINISH): 9 December 2019

LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY		MONITOR WELL	SAMPLE DATA				
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK		State	Type and Number	Recovery	OVC	Penetration Index / RQD
meters	59.10		GROUND SURFACE				%	ppm	N
15.0			Approximately 50 mm crushed seams encountered at 14.6 and 14.8 mbgs comprising of silty clayey gravel Crushed seams approximately every 100mm			RC5	84		30
16.0				16.38		RC6	88		29
	42.5		Borehole terminated at 16.6 mbgs						
17.0									
18.0									
19.0									
20.0									
21.0									
22.0									
23.0									
24.0									
25.0									
26.0									
27.0									
28.0									



SCALE FOR TEST RESULTS

50kPa 100kPa 150kPa 200kPa

10 20 30 40 50 60 70 80 90

NOTES:

mbgs: meters below ground surface

Elevations are approximate based on shoring drawing

REFERENCE No.: 11186719

ENCLOSURE No.: 10

BOREHOLE No.: BH10

ELEVATION: 59.20 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

DESCRIBED BY: D.Cooper

CHECKED BY: B. Vazhbakht

DATE (START): 10 December 2019

DATE (FINISH): 10 December 2019

LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY		SAMPLE DATA				
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	OVC	Penetration Index / RQD
meters	59.20		GROUND SURFACE			%	ppm	N
			Augered to practical refusal with no sampling	▮				
6.0	53.1		**BOULDER**					
	52.9		**SHALE**- dark grey, fresh, highly fractured, occasional seams of crushed rock comprising of silty clayey sand, laminated at 0 to 10°		RC1	50		8
			Vertical joint, planar, calcite coated encounterd from 8.2 to 8.9 mbgs		RC2	83		9
			Thinly laminated at 5 to 10°					
			5 mm crushed seam of sandy gravel encountered at 9.8 mbgs 13 mm crushed seam of sandy gravel seam encountered at 9.9 mbgs		RC3	100		67
			Subvertical joint, clean, planar, rough to smooth encountered at 10.1 mbgs Occasional joint at 45° every 300 mm, clean, rough to smooth		RC4	100		89
					RC5	79		94
	45.7		Borehole terminated at 13.5 mbgs					

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0

SCALE FOR TEST RESULTS

50kPa 100kPa 150kPa 200kPa

10 20 30 40 50 60 70 80 90

NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

REFERENCE No.: 11186719 ENCLOSURE No.: 11

BOREHOLE No.: **BH11**

ELEVATION: 59.20 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

DESCRIBED BY: D.Cooper CHECKED BY: B. Vazhbakht

DATE (START): 10 December 2019 DATE (FINISH): 10 December 2019

LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
⊢⊣		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer

SCALE		STRATIGRAPHY		SAMPLE DATA					
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State	Type and Number	Recovery	OVC	Penetration Index / RQD	SCALE FOR TEST RESULTS 50kPa 100kPa 150kPa 200kPa 10 20 30 40 50 60 70 80 90
meters	59.20		GROUND SURFACE			%	ppm	N	
1.0 2.0 3.0 4.0 5.0			Augered to practical refusal with no sampling						
	53.8		**LIMESTONE**		RC1	70		75	
6.0 7.0	53.1		**SHALE**- dark grey, fresh, thinly laminated at 0 to 5°. Defects comprisined of partings in line with bedding, clean, rough to smooth		RC2	67		44	
8.0					RC3	100		94	
9.0 10.0			Weathered/crusehd seam comprising of silty sand with some clay encountered at 8.8 mbgs Approximately 70° joint, smooth, clean to rough encountered at 8.9 mbgs Approximately 45° joint, smooth, clean to rough encountered at 9.3 mbgs		RC4	100		98	
11.0					RC5	93		82	
12.0 13.0 14.0	47.3		Borehole terminated at 11.9 mbgs						

NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

REFERENCE No.: 11186719

ENCLOSURE No.: 12

BOREHOLE No.: **BH12**

ELEVATION: 59.10 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: TC United Group

PROJECT: Geotechnical Investigation

LOCATION: 36 Robinson Avenue

DESCRIBED BY: D.Cooper

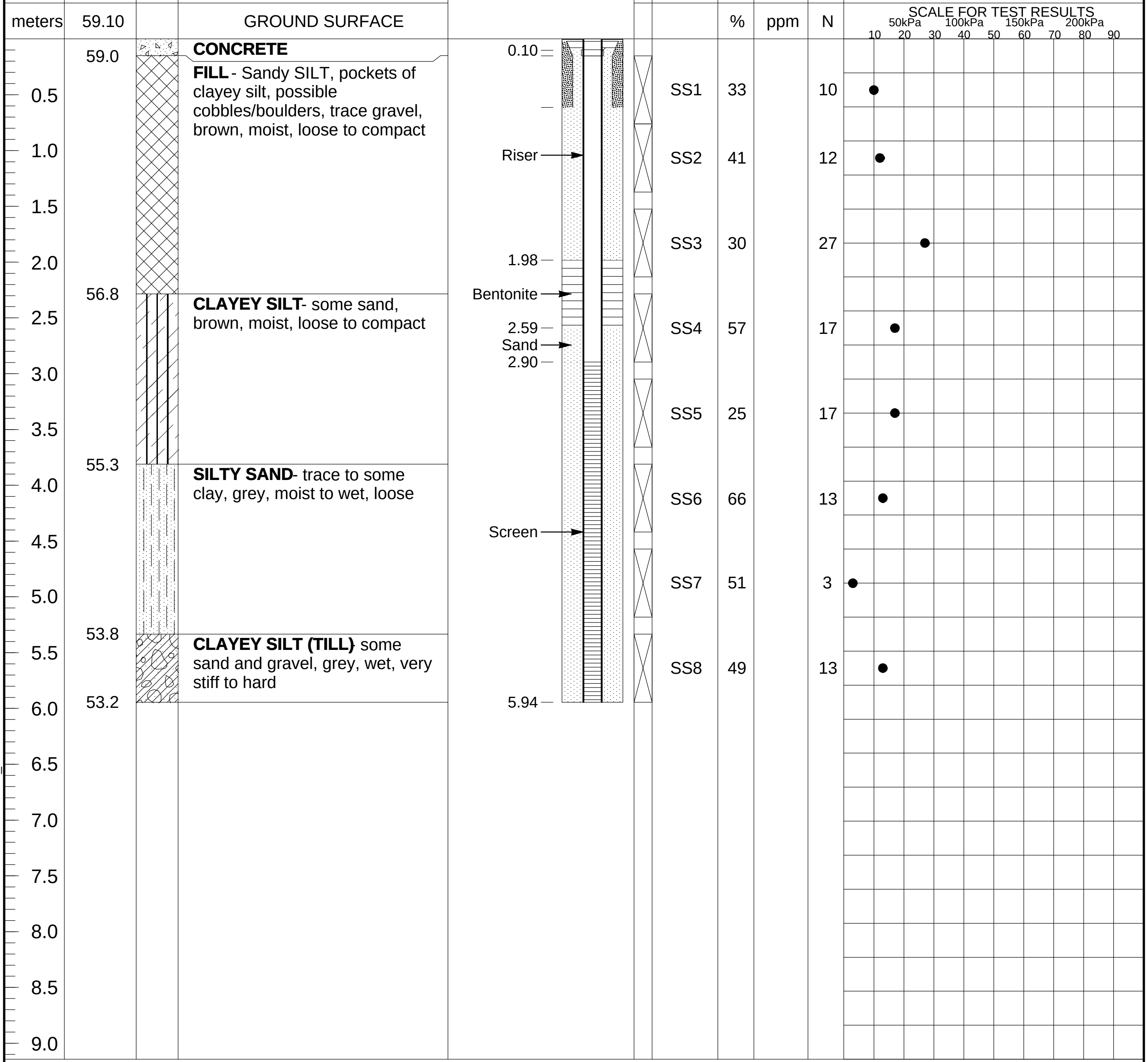
CHECKED BY: B. Vazhbakht

DATE (START): 9 December 2019

DATE (FINISH): 9 December 2019

LEGEND

☒	SS	Split Spoon
▮	GS	Auger Sample
▨	ST	Shelby Tube
▼		Water Level
○		Water content (%)
├─┤		Atterberg limits (%)
●	N	Penetration Index based on Split Spoon sample
•	N	Penetration Index based on Dynamic Cone sample
△	Cu	Shear Strength based on Field Vane
□	Cu	Shear Strength based on Lab Vane
S		Sensitivity Value of Soil
▲		Shear Strength based on Pocket Penetrometer



Elevation (m)	DESCRIPTION OF SOIL AND BEDROCK
59.10	GROUND SURFACE
59.0	**CONCRETE**
	FILL - Sandy SILT, pockets of clayey silt, possible cobbles/boulders, trace gravel, brown, moist, loose to compact
56.8	**CLAYEY SILT**- some sand, brown, moist, loose to compact
55.3	**SILTY SAND**- trace to some clay, grey, moist to wet, loose
53.8	**CLAYEY SILT (TILL)**- some sand and gravel, grey, wet, very stiff to hard
53.2	

State	Type and Number	Recovery %	OVC ppm	Penetration Index / RQD N
☒	SS1	33		10
☒	SS2	41		12
☒	SS3	30		27
☒	SS4	57		17
☒	SS5	25		17
☒	SS6	66		13
☒	SS7	51		3
☒	SS8	49		13

NOTES:
mbgs: meters below ground surface
Elevations are approximate based on shoring drawing

BOREHOLE LOG 11186719-A1-BOREHOLE LOGS.GPJ INSPEC_SOL.GDT 16/12/19

Notes on Borehole and Test Pit Reports

Soil description :

Each subsurface stratum is described using the following terminology. The relative density of granular soils is determined by the Standard Penetration Index ("N" value), while the consistency of clayey sols is measured by the value of undrained shear strength (Cu).

Classification (Unified system)

Clay	< 0.002 mm		
Silt	0.002 to 0.075 mm		
Sand	0.075 to 4.75 mm	fine	0.075 to 4.25 mm
		medium	0.425 to 2.0 mm
		coarse	2.0 to 4.75 mm
Gravel	4.75 to 75 mm	fine	4.75 to 19 mm
		coarse	19 to 75 mm
Cobbles	75 to 300 mm		
Boulders	>300 mm		

Terminology

"trace"	1-10%
"some"	10-20%
adjective (silty, sandy)	20-35%
"and"	35-50%

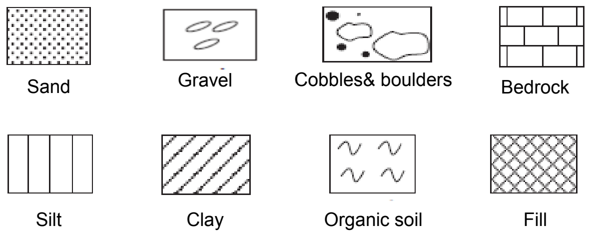
Relative density of granular soils	**Standard penetration index "N" value** (BLOWS/ft – 300 mm)
Very loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Consistency of cohesive soils	**Undrained shear strength (Cu)** (P.S.F)	(kPa)
Very soft	<250	<12
Soft	250-500	12-25
Firm	500-1000	25-50
Stiff	1000-2000	50-100
Very stiff	2000-4000	100-200
Hard	>4000	>200

Rock quality designation

"RQD" (%) Value	Quality
<25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
>90	Excellent

STRATIGRAPHIC LEGEND



Samples:

Type and Number

The type of sample recovered is shown on the log by the abbreviation listed hereafter. The numbering of samples is sequential for each type of sample.

SS: Split spoon
SSE, GSE, AGE: Environmental sampling
ST: Shelby tube
PS: Piston sample (Osterberg)
AG: Auger
RC: Rock core
GS: Grab sample

Recovery

The recovery, shown as a percentage, is the ratio of length of the sample obtained to the distance the sampler was driven/pushed into the soil

RQD

The "Rock Quality Designation" or "RQD" value, expressed as percentage, is the ratio of the total length of all core fragments of 4 inches (10 cm) or more to the total length of the run.

IN-SITU TESTS:

N: Standard penetration index
R: Refusal to penetration
N_c: Dynamic cone penetration index
Cu: Undrained shear strength
Pr: Pressure meter
k: Permeability
ABS: Absorption (Packer test)

LABORATORY TESTS:

I_p: Plasticity index
W_l: Liquid limit
Wp: Plastic limit
H: Hydrometer analysis
GSA: Grain size analysis
A: Atterberg limits
w: Water content
γ: Unit weight
C: Consolidation
CS: Swedish fall cone
CHEM: Chemical analysis
O.V.: Organic vapor

Table 1

Summary of Shear Wave Velocity Measurements
Seismic Site Class Determination
Proposed New Residential Condominium
TC United Group.
36 Robinson Avenue, Ottawa, ON

Table 1-A: Average Shear Wave Velocity (VS_{30})
(Assumed foundaiton at 6.0 m below existing ground surface)

Line 1

Layer No.	Depth (m bgs)		Thickness	V_s	d_i/V_{si}
	From	To	m	m/s	
1	6.0	8.6	2.6	414	0.0062
2	8.6	11.7	3.2	904	0.0035
3	11.7	15.7	4.0	1113	0.0036
4	15.7	20.7	5.0	1212	0.0041
5	20.7	36.0	15.3	1352	0.0113
Total			30.0		0.0287
Average Shear Wave Velocity Along the Line (m/s)					***1046***

Table 1-A: Average Shear Wave Velocity (VS_{30})
(Assumed foundaiton at 6.0 m below existing ground surface)

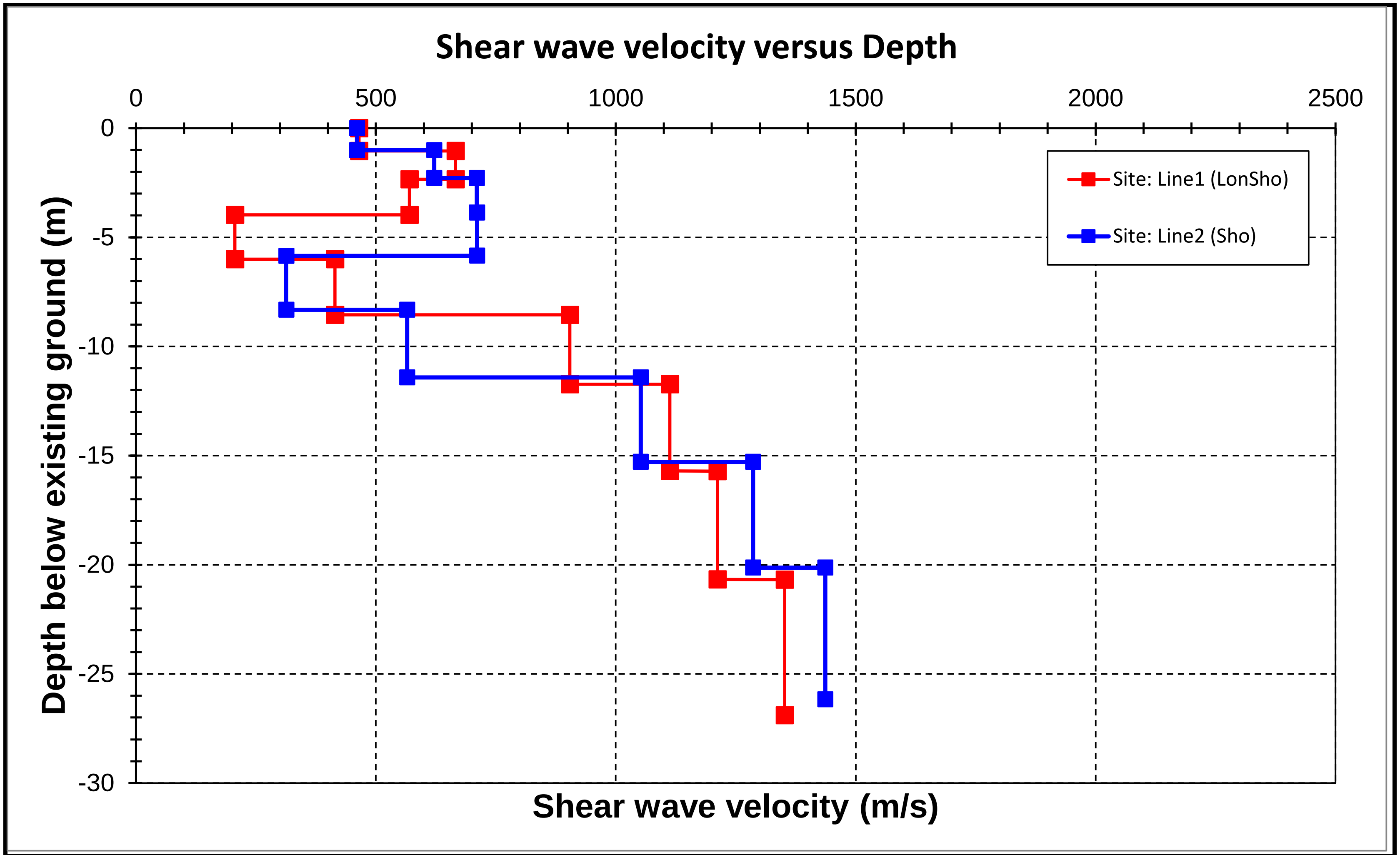
Line 2

Layer No.	Depth (m bgs)		Thickness	V_s	d_i/V_{si}
	From	To	m	m/s	
1	6.0	8.3	2.3	313	0.0074
2	8.3	11.4	3.1	565	0.0055
3	11.4	15.3	3.9	1052	0.0037
4	15.3	20.1	4.8	1286	0.0038
5	20.1	36.0	15.9	1437	0.0111
Total			30.0		0.0314
Average Shear Wave Velocity Along the Line (m/s)					***956***

Average VS_{30} = **1001** **m/s**

Recommended Site Class: **B** Subjected to Code requirements

Notes:
1 - The Seismic Site class is recommended in accordance to Table 4.1.8.4.A of the National Building code of Canada 2010 and based on the lowest measured average shear wave velocity measured along the investigated lines.
2 - VS30 is calculated based on the average shear wave velocity below the proposed founding elevation.
3 - Site Classes A and B are only applicable if footings are founded on bedrock or there is no more than 3.0 m of soil between founding elevation and bedrock.
4 - The recommended site class is only applicable if site conditions for Site Class F (liquefiable soil/soft soil layers more than 3.0 m thick) are not applicable.



Shear Wave Velocity versus Depth **Proposed New Residential Condominium** **TC United Group** **36 Robinson Avenue, Ottawa, ON**	GHD	ANALYSED BY:	SCALE	REFERENCE NO:
		Hassan Ali Ph.D	N.T.S	11186719
		REVIEWED BY:	DATE:	FIGURE NO:
		Ali Nasseri-Moghaddam Ph.D.	Dec-18	2

SIEVE ANALYSIS

CLIENT: TC United Group

LAB No.: G-18-10

PROJECT/SITE: 36 Robinson Avenue, Ottawa, On

PROJECT No.: 11186719-A1

SOURCE: BH5 SS7 (4.6-5.2m)

SAMPLE LOCATION: N/A

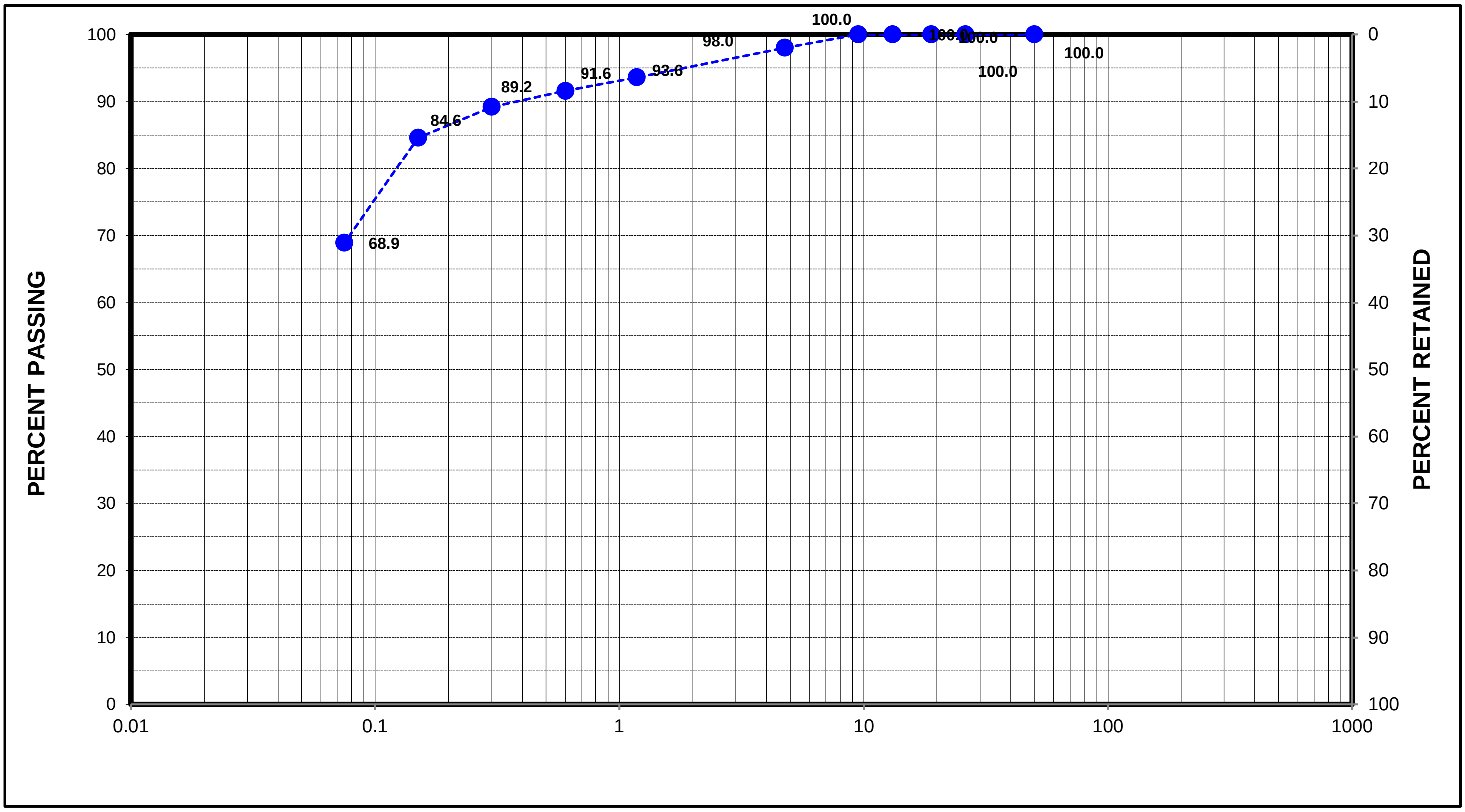
MATERIAL TYPE: N/A

PROPOSED USE: N/A

SAMPLED BY: SW

DATE SAMPLED: December 17, 2018

SIEVE SIZE (mm)	SAMPLE % PASSING	
50.0	100.0	
26.2	100.0	
19.0	100.0	
13.2	100.0	
9.5	100.0	
4.75	98.0	
1.18	93.6	
0.600	91.6	
0.300	89.2	
0.150	84.6	
0.075	68.9	



REMARKS:

PERFORMED BY: R. Vanden Tillaart

DATE: January 7, 2019

VERIFIED BY:

DATE: January 8, 2019

SIEVE ANALYSIS

CLIENT: TC United Group

LAB No.: G-18-10

PROJECT/SITE: 36 Robinson Avenue, Ottawa, On

PROJECT No.: 11186719-A1

SOURCE: BH6 SS9 (6.1-6.7m)

SAMPLE LOCATION: N/A

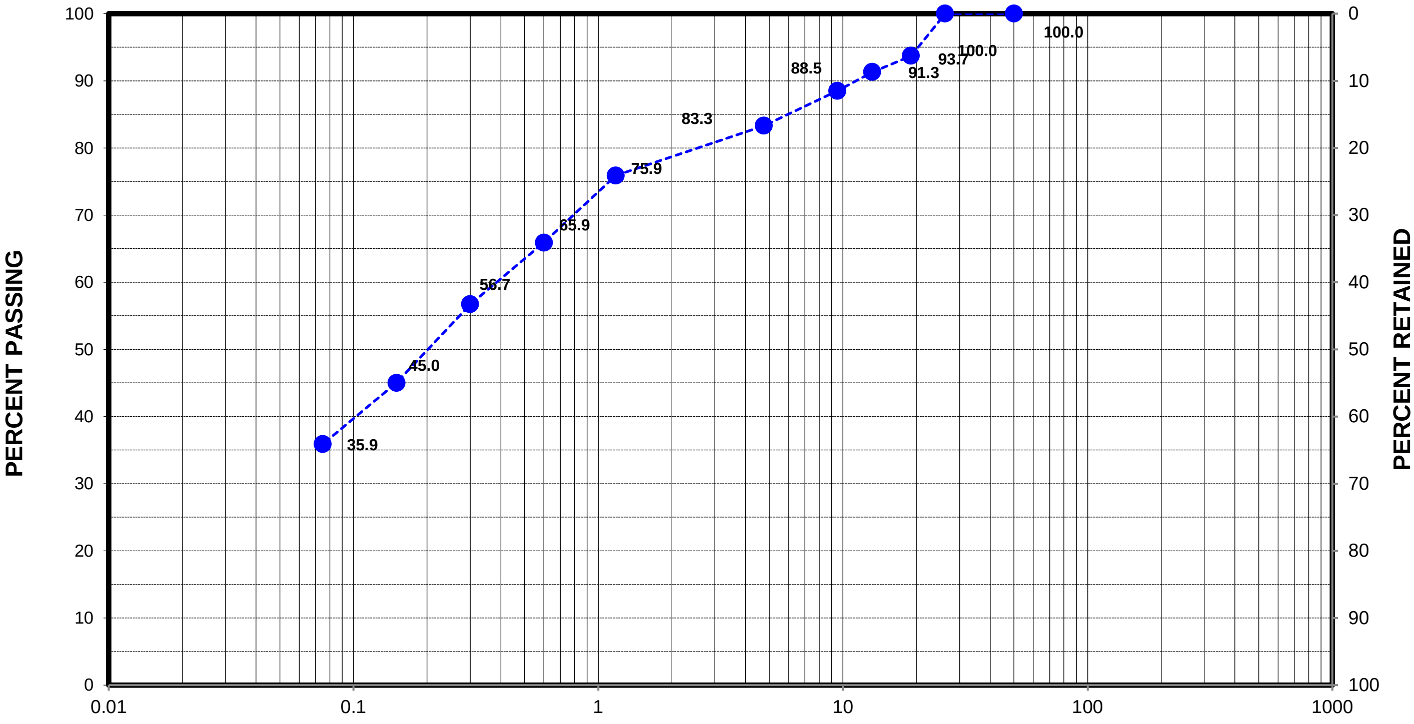
MATERIAL TYPE: N/A

PROPOSED USE: N/A

SAMPLED BY: SW

DATE SAMPLED: December 17, 2018

SIEVE SIZE (mm)	SAMPLE % PASSING	
50.0	100.0	
26.2	100.0	
19.0	93.7	
13.2	91.3	
9.5	88.5	
4.75	83.3	
1.18	75.9	
0.600	65.9	
0.300	56.7	
0.150	45.0	
0.075	35.9	



REMARKS:

PERFORMED BY: R. Vanden Tillaart

DATE: January 7, 2019

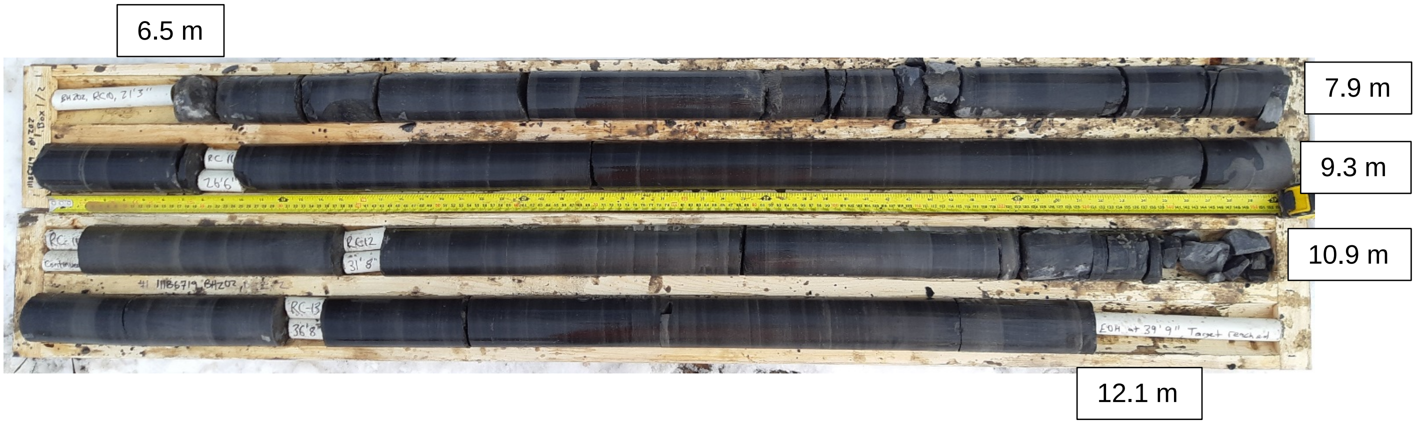
VERIFIED BY:

DATE: January 8, 2019

BH7-RC13/RC14/RC15

BH7-RC13, RC14, RC15 November 18, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH7 – RC13 – 8.4 to 9.6 mbgs	1.2	100	37% RQD
BH7 – RC14 – 9.6 to 11.1 mbgs	1.3	84	21% RQD
BH7 – RC15 – 11.1 to 12.2 mbgs	1.1	100	0% RQD

BH8-RC10/RC11/RC12/RC13



BH8-RC10, RC11, RC12, RC13 November 18, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	M	%	
BH8 – RC10 – 6.5 to 8.1 mbgs	1.6	100	57% RQD
BH8 – RC11 – 8.1 to 9.7 mbgs	1.6	100	100% RQD
BH8 – RC12 – 9.7 to 11.2 mbgs	1.4	92	70% RQD
BH8 – RC13 – 11.2 to 12.1 mbgs	0.9	100	100% RQD

BH9-RC1/RC2

7.4 m

8.9 m

BH9-RC3/RC4

10.4 m

12.0 m

13.5 m

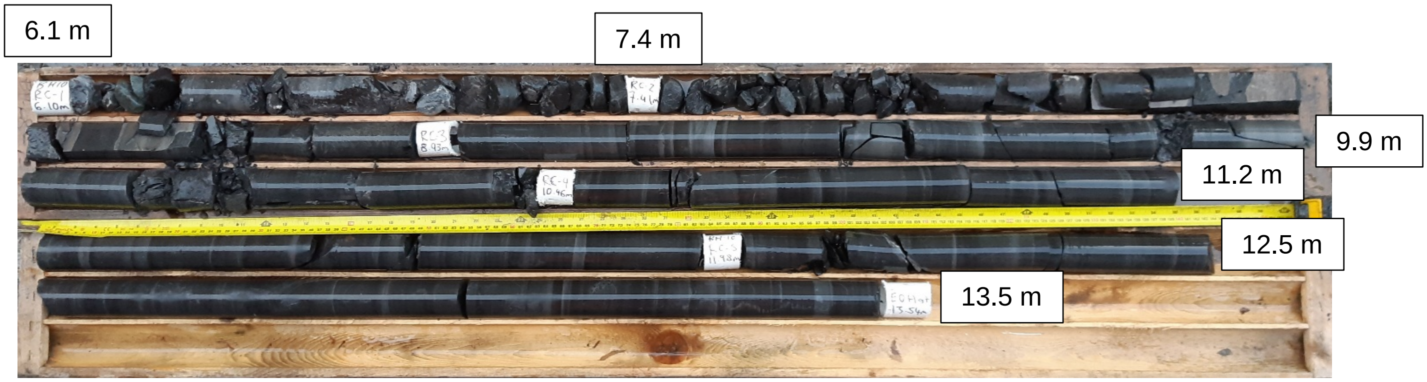
BH9-RC1, RC2, RC3, RC4
December 9, 2019

Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH9 – RC1 – 7.4 to 8.9 mbgs	1.3	87	9% RQD
BH9 – RC2 – 8.9 to 10.4 mbgs	1.5	100	50% RQD
BH9 – RC3 – 10.4 to 12.0 mbgs	1.5	92	14% RQD
BH9 – RC4 – 12.0 to 13.5 mbgs	1.5	100	44% RQD

BH9-RC5/RC6

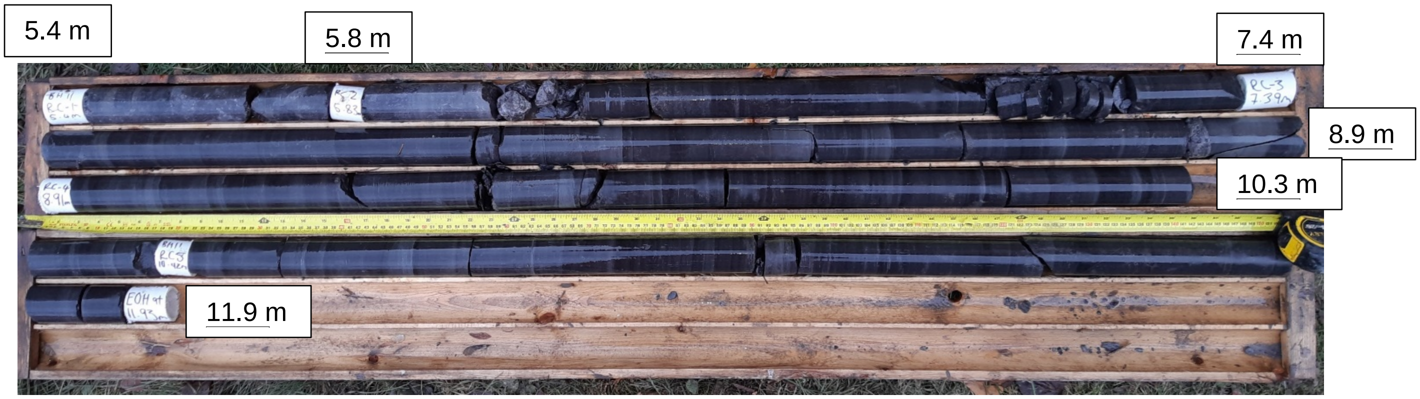
BH9-RC5, RC6 December 9, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	M	%	
BH9 – RC5 – 13.5 to 15.1 mbgs	1.3	84	30% RQD
BH9 – RC6 – 15.1 to 16.6 mbgs	1.3	88	29% RQD

BH10-RC1/RC2/RC3/RC4/RC5



BH10-RC1, RC2, RC3, RC4, RC5 December 10, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH10 – RC1 – 6.1 to 7.4 mbgs	0.7	50	8% RQD
BH10 – RC2 – 7.4 to 8.9 mbgs	1.2	83	9% RQD
BH10 – RC3 – 8.9 to 10.5 mbgs	1.6	100	67% RQD
BH10 – RC4 – 10.5 to 12.0 mbgs	1.5	100	89% RQD
BH10 – RC5 – 12.0 to 13.5 mbgs	1.2	79	94% RQD

BH11-RC1/RC2/RC3/RC4/RC5



BH11-RC1, RC2, RC3, RC4, RC5 December 10, 2019			
Core Run - Depth below ground surface (mbgs)	Recovery		Remarks
	m	%	
BH11 – RC1 – 5.4 to 5.8 mbgs	0.3	70	75% RQD
BH11 – RC2 – 5.8 to 7.4 mbgs	1.1	67	44% RQD
BH11 – RC3 – 7.4 to 8.9 mbgs	1.5	100	94% RQD
BH11 – RC4 – 8.9 to 10.4 mbgs	1.5	100	98% RQD
BH11 – RC5 – 10.4 to 11.9 mbgs	1.4	93	82% RQD

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

November 27, 2018

Site: 45.4183 N, 75.6658 W User File Reference:

Requested by: ,

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)**
0.450	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	0.188	0.302
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	0.045	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

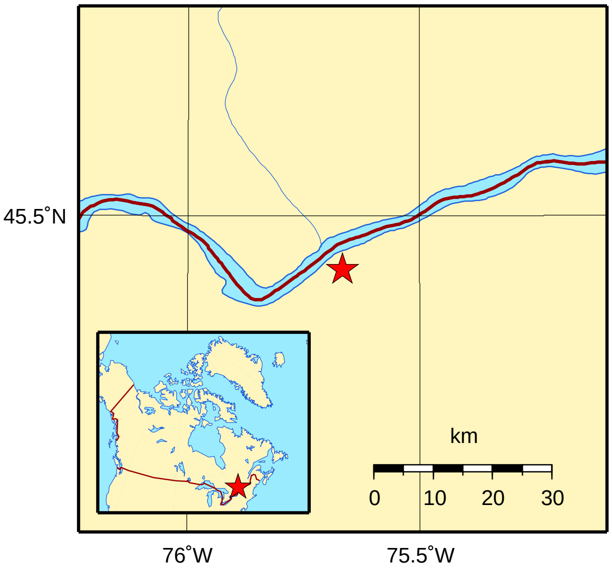
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

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



Natural Resources Canada Ressources naturelles Canada

Canada

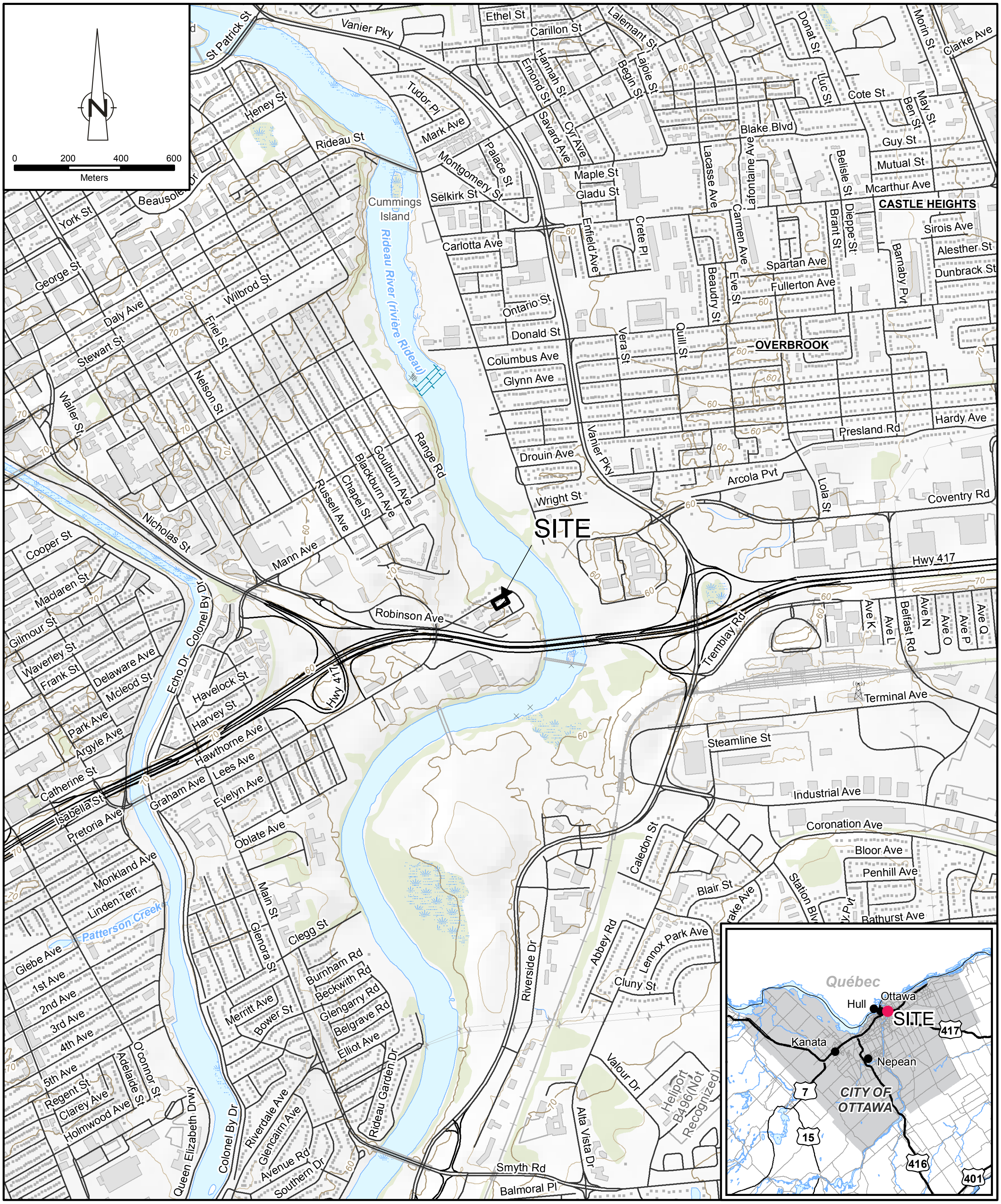
APPENDIX 2

FIGURE 1 - SITE LOCATION PLAN (BY OTHERS)

FIGURE 2 - BOREHOLE LOCATION PLAN (BY OTHERS)

FIGURE 3 - WATER SUPPRESSION SYSTEM

DRAWING PG5231-1 - TEST HOLE LOCATION PLAN



Source: MNRF NRVIS, 2018. Produced by GHD under licence from Ontario Ministry of Natural Resources and Forestry, © Queen's Printer 2019.
Coordinate System: NAD 1983 UTM Zone 18N

TC UNITED GROUP
36 ROBINSON AVENUE, OTTAWA, ONTARIO
GEOTECHNICAL INVESTIGATION

11186719-A1
Jan 2, 2019

SITE LOCATION MAP

FIGURE 1

GIS File: Q:\GIS\PROJECTS\11186000s\11186719\Layouts\001\11186719-A1(001)GIS-OT005.mxd

Source: Microsoft product screen shot(s) reprinted with permission from Microsoft Corporation, Date Unknown

0 5 10 15
Meters

Coordinate System:
NAD 1983 UTM Zone 18N

N

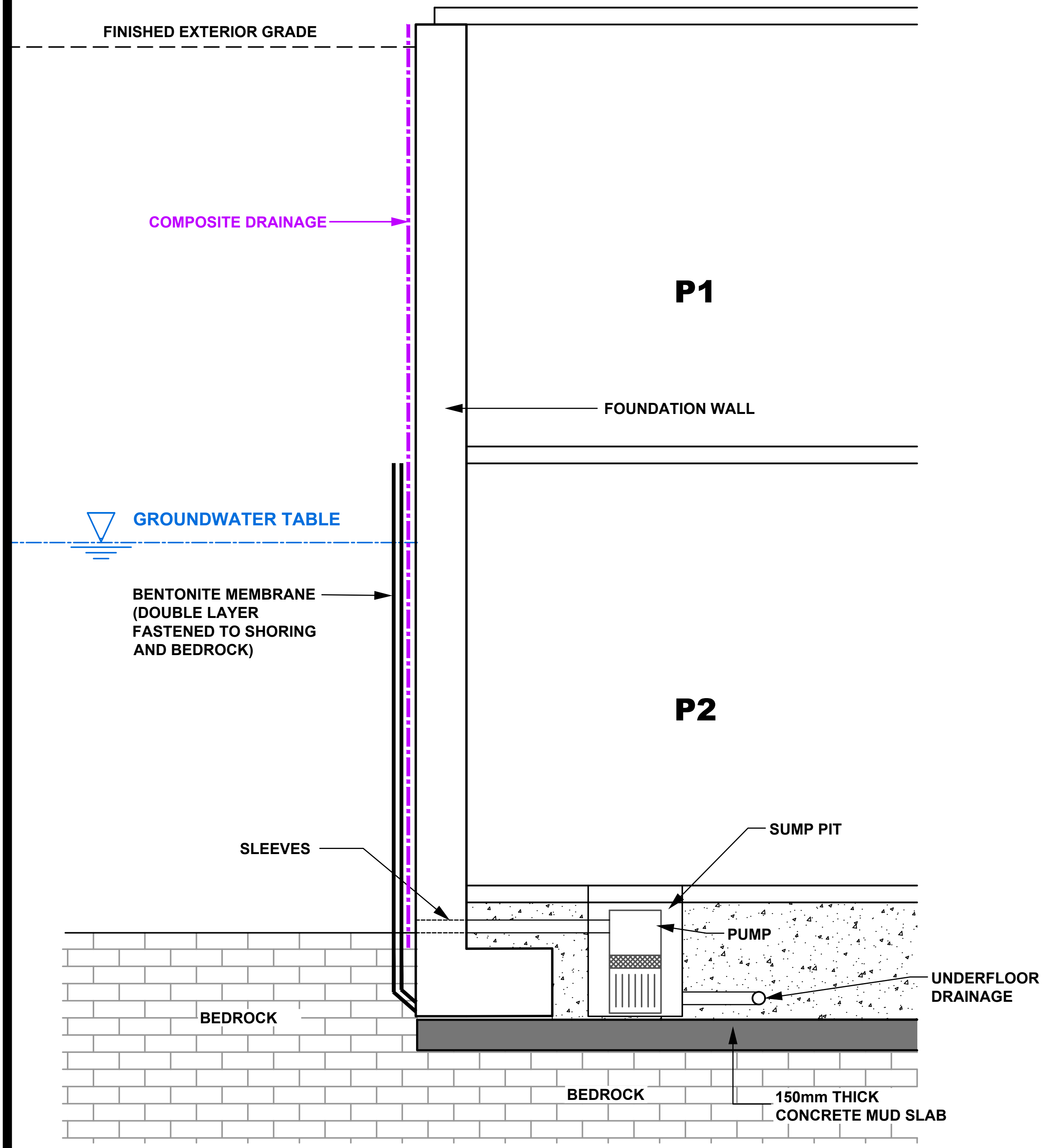
ROBINSON VILLAGE LPIV LIMITED PARTNERSHIP
36 ROBINSON AVENUE, OTTAWA, ONTARIO
GEOTECHNICAL INVESTIGATION

11186719-A1
Dec 11, 2019

BOREHOLE LOCATION PLAN

FIGURE 2

GIS File: Q:\GIS\PROJECTS\11186000s\11186719\Layouts\001\11186719-A1(001)GIS-OT006.mxd



patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344
www.patersongroup.ca

ROBINSON VILLAGE LIMITED PARTNERSHIP

PROPOSED MULTI-STOREY BUILDING
36 ROBINSON AVENUE
OTTAWA, ONTARIO

Title:

WATER SUPPRESSION SYSTEM

Date: 03/2020

Report No.: PG5231

Scale: N.T.S

Drawing No.: FIGURE 3

Drawn by: RCG

Checked by: CDS

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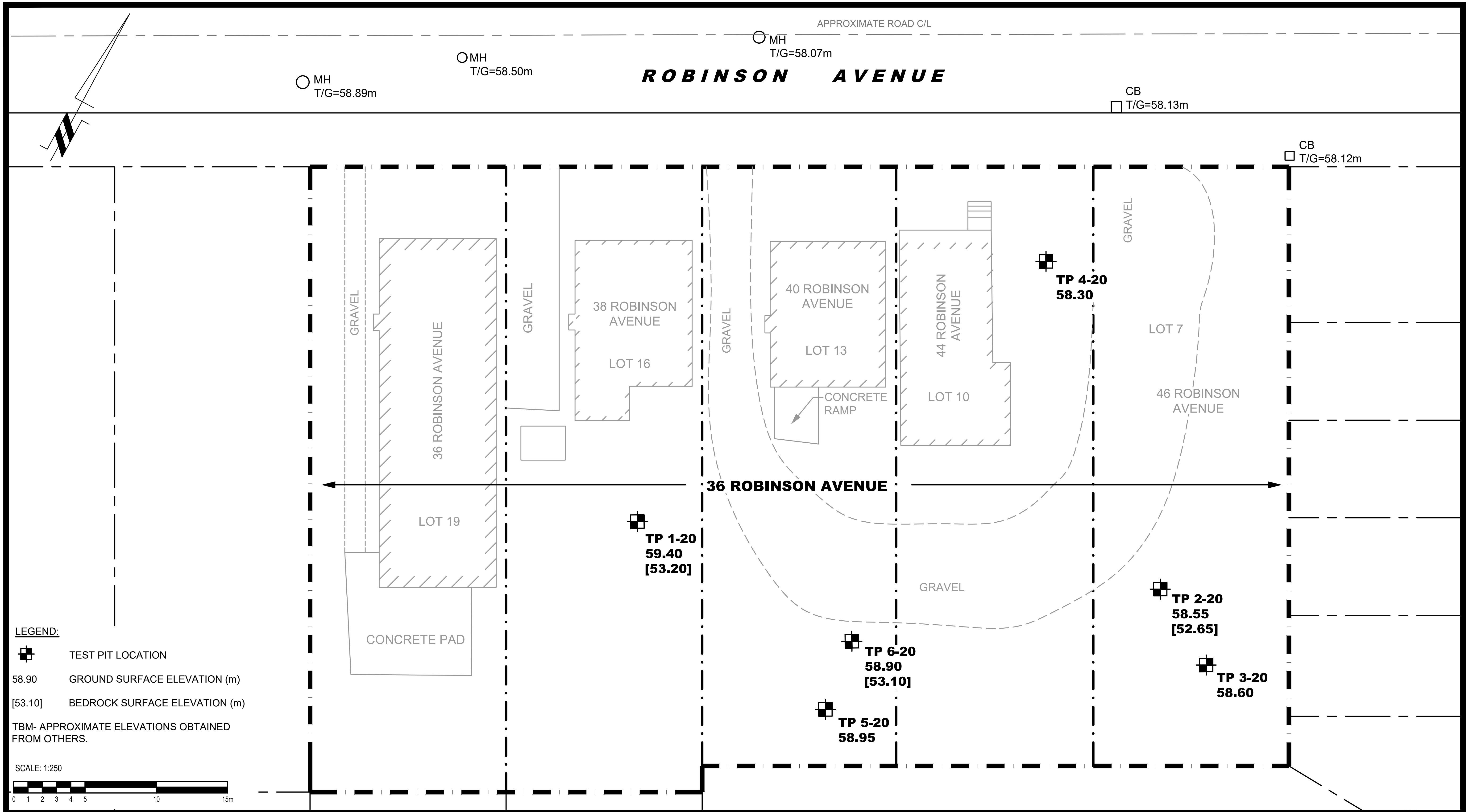

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

TCU DEVELOPMENTS
GEOTECHNICAL INVESTIGATION
36 ROBINSON AVENUE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 03/2020
Drawn by: YA
Report No.: PG5231-1
Checked by: DL
Dwg. No.: **PG5231-1**
Approved by: CDS
Revision No.:

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