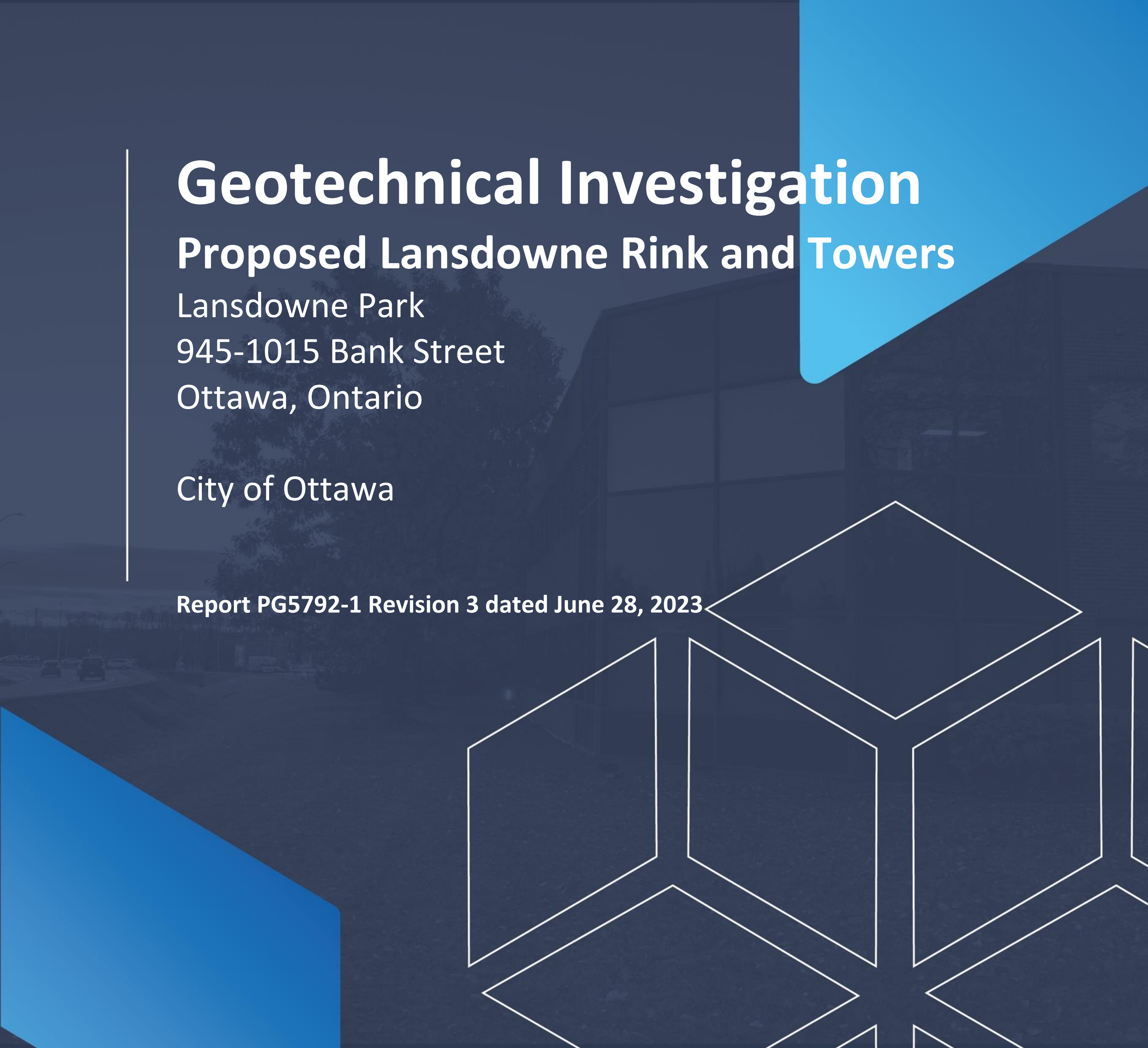

Geotechnical Investigation

Proposed Lansdowne Rink and Towers

Lansdowne Park
945-1015 Bank Street
Ottawa, Ontario

City of Ottawa

Report PG5792-1 Revision 3 dated June 28, 2023

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by City of Ottawa to conduct a geotechnical investigation for the proposed Lansdowne Park Redevelopment Project, to be located on 945-1015 Bank Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of boreholes.
- Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

The proposed Zoning By-law & Official Plan Amendments for Lansdowne 2.0 represent the next step in the evolution and progression of Lansdowne towards a redevelopment approach that will allow the site to succeed as an important residential, sports, culture, recreation, commercial, and entertainment destination, and a more vibrant day-to day hub for Ottawa.

The work described below requires both an Official Plan and Zoning By-law Amendment to align the proposed development plan with the City’s planning policy framework. In general, the proposed development includes two components – a public infrastructure component and a private infrastructure component.

These two principal components are advanced through a number of important elements, including:

- Increase residential density to foster daily vibrancy to the area through proposing three high-rise towers including a combination of condominium and rental including affordable housing units.
    - The private infrastructure component could include up to 1,200 new residential units on-site, which could be provided in three new towers atop the proposed retail podium.
    - The proposed heights of the towers as per the 2022 Council-endorsed concept are 29, 34, and 40 storeys, with the proposed maximum height limited to 40 storeys. Approximately 739 new parking spaces could be provided for the new residential units and located within the underground parking lot and within level 1 and level 1.5 (mezzanine) of the podium and north stadium stands.

- Add mixed-used retail space through replacing the current 3,809m² (41,000 ft²) retail space attached to the arena/stadium complex along Exhibition Way, with 9,290m² (100,000 ft²) of new mixed-use retail space in the podium of the new residential towers.
    - The new north stadium stands will be integrated with a new retail podium, which will provide additional retail options to the existing Lansdowne Park, enhance the existing public realm along Exhibition Way, and enhance the protected viewpoint of Aberdeen Pavilion from Bank Street.

- Replace the existing City facilities on-site through proposing a new 5,500-seat standalone multipurpose Event Centre and a re-designed and reconstructed 12,000-seat North Stadium Stands.
    - The public infrastructure component will include a new event centre which is intended to replace the existing 9,500 seat TD Place Arena which is located within the north stadium stands.
    - In addition to the new event centre, the north stadium stands will be replaced with 11,200 new seats, which will accommodate 12,000 spectators with additional standing-only areas.

- Consolidate service access & loading through including a common access point for service & loading is provided for the Event Centre, Stadium, Residential and Retail

- Facilitate City-led enhancements to the public realm and programming as per the direction of the Lansdowne Guiding Principles which will form an important part of Lansdowne 2.0.

Overall, the proposal intends to re-visit the form and function of Lansdowne, and specifically Exhibition Way, as a place of exhibition, open to the City as whole that fosters public gathering, vibrancy, and a centre of activity for the City. There will be a continued focus on placemaking, and the careful integration and enhancement of all new features with the objectives of the existing site – including a shared commitment to recognizing and celebrating Algonquin history, art and culture, respecting heritage building views, animating Exhibition Way, providing access to the Great Lawn, and preserving an incorporating existing public and private components of Lansdowne today.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out from October 25, 2021 to November 12, 2021 and consisted of advancing a total of six (6) boreholes to a maximum depth of 34 m below existing grade. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5792-1 - Test Hole Location Plan included in Appendix 2.

Boreholes were advanced using a low clearance drill rig operated by a two-person crew. The drilling procedure consisted of augering and coring to the required depths at the selected locations and sampling the overburden soils and bedrock. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department.

Sampling and In Situ Testing

Soil samples were recovered from the auger flights, using a 50 mm diameter split-spoon sampler, or core recovery barrels. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. Rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination. The depths at which the split-spoon, auger flights, and rock core samples were recovered from the boreholes are shown as SS, AU, and RC, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of each of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was completed at boreholes BH 3-21, BH 4-21, BH 5-21 and BH 6-21 to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage.

The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Boreholes BH 5-21, BH 6-21, BH 7-21 and BH 9-21 were fitted with 51 mm diameter PVC groundwater monitoring wells. The other boreholes were fitted with flexible polyethylene standpipes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- Slotted PVC screen at the base of each borehole.
- 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No.3 silica sand backfill within annular space around screen.
- Bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The borehole locations were selected by Paterson personnel in a manner to provide general coverage of the proposed development, taking into consideration existing site features. The borehole locations and ground surface elevations were referenced to a geodetic datum. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG5792-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

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

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures by Paterson. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by TD Place stadium, north and south side stands, Civic Center, a mixed-use commercial building and the Aberdeen Pavilion. One storey of underground parking currently exists under the northern portion of the property. Furthermore, a grass covered berm with an approximate maximum ground elevation ~73m and an art sculpture occupy the central portion of the site along with a flat landscaped area, having an average approximate geodetic elevation ~64m, occupying the east portion. Paved walkways are also present within the subject site.

The Lansdowne Park Development is bound by Bank Street to the west, commercial and residential developments to the north, and by Queen Elizabeth Driveway and the Rideau Canal to the south.

4.2 Subsurface Profile

Overburden

Generally, the soil profile at the borehole locations consists of topsoil and/or asphaltic concrete and fill overlying compact brown silty sand/sandy silt, trace gravel, followed by very dense to dense glacial till, underlain by bedrock. Based on the encountered fill thicknesses, the native subgrade is at approximate geodetic elevation of 62.7 to 64.7m. The glacial till was observed to consist of brown silty sand with gravel, cobbles, and boulders.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for details of the soil profile encountered at each borehole location.

Bedrock

BH 3-21, BH 4-21, BH 5-21, BH 6-21, BH 7-21, BH 8-21, and BH 9-21 were extended to bedrock surface. Bedrock was encountered at approximate elevations of 41 to 43m. The bedrock was cored in the locations of BH 3-21, BH 4-21, BH 5-21, BH 6-21, BH 7-21, BH 8-21, and BH 9-21 with RQD values ranging from 85 to 100%. This is indicative of excellent quality grey limestone bedrock.

Based on available geological mapping and coring records, the bedrock in the subject area consists of limestone and shale of the Billings formation, with an overburden drift thickness of 10 to 15 m.

4.3 Groundwater

A long-term groundwater level monitoring program took place from September 15, 2021, until November 9, 2022 (see PH4424-MEMO.01 Dated December 2, 2022, for specific details). Data loggers were installed in five existing monitoring wells (MW 15-06, MW 15-07, MW 15--09, MW 15-10, and MW 15-11) as well as in four monitoring wells installed by Paterson (BH 5-21, BH 6-21, BH 8-21 and BH 9-21). The groundwater measurements, as well as the minimum and maximum groundwater levels are presented in Table 1. It should be noted that groundwater levels are subject to seasonal fluctuations and the influence of the Rideau Canal, which is located southeast of the subject site. Therefore, groundwater levels may vary at the time of construction.

Table 1 – Groundwater Elevation Summary

		Groundwater Elevation (m asl)				Difference Between Maximum and Minimum Groundwater Elevation (m)
Monitoring Well ID	**Ground Surface Elevation (m asl)**	**Maximum**	**Date**	**Minimum**	**Date**	
MW15-06	64.90	60.74	16-08-22	*59.72	09-03-22 20-04-22 10-05-22	*1.02
MW15-07	64.51	60.42	18-08-22	59.18	20-04-22	1.24
MW15-09	65.25	60.60	16-08-22	*59.19	09-03-22	*1.41
MW15-10	64.91	60.57	17-08-22	*59.08	09-03-22	*1.49
MW15-11	64.57	60.67	09-22-22	59.12	10-11-21	1.54
BH5-21	65.14	60.58	16-08-22	59.20	08-02-22	1.39
BH6-21	66.62	60.55	26-09-22	59.13	09-03-22	1.42
BH8-21	65.45	60.60	26-09-22	59.30	09-03-22	1.31
BH9-21	67.07	60.78	26-09-22	59.41	09-03-22	1.37

Note "*": Dry well - the minimum groundwater elevation is noted to be at the elevation of the well invert.

Based on the results of the groundwater monitoring program, the groundwater table elevation was found to range from <59.08 to 60.78 m asl and is within the overburden materials. Depending on the depth of well installation, a low water elevation was not able to be recorded at all locations. Maximum and minimum groundwater elevations were observed at the end of summer/early fall and the end of winter/early spring, respectively, at each groundwater monitoring location, indicating that groundwater levels are seasonally influenced by water levels in the Rideau Canal.

Reference should be made to the individual monitoring locations for design considerations at specific locations. Based on monitoring completed to date, design specifications should be based on a water table elevation of **60.78 m asl**, the maximum groundwater elevation observed during the long-term groundwater monitoring period.

It should be noted that groundwater levels can fluctuate seasonally and with precipitation events. Therefore, groundwater levels could vary.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed redevelopment. In view of the anticipated building loads and founding depths, several foundation design options are considered to be suitable and are listed below:

- Conventional footings for podium areas outside the tower footprints (i.e., arena, podium, garage) to be placed on undisturbed compact to dense silty sand, and above the groundwater level.

- Raft foundation for finish floor levels below the groundwater level; (i.e., arena, music hall, and underground parking structure below T3).

- End bearing pile foundations or caisson foundations for the proposed towers.

Where the founding level extends below the groundwater level, a full watertight design will be required for the foundation walls and floor slabs (i.e., arena, entertainment venue and underground parking levels).

Existing foundation walls and other construction debris should be entirely removed from within the building perimeters. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeters. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000, connected to a perimeter drainage system.

5.3 Foundation Design

Several foundation design options are available for the proposed structures depending on the design loading and foundation depth. The following foundation options can be considered for support of the proposed structures:

Podium Areas Outside Tower Footprints: Conventional Shallow Foundation

Using continuously applied loads, footings for the proposed buildings placed over an undisturbed, compact to dense silty sand/glacial till bearing surface can be designed using a bearing resistance value at SLS of **250 kPa** and a factored bearing resistance value at ULS of **400 kPa**.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surface.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen, or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Settlement

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided herein will be subjected to potential post-construction total and differential settlements of 25 to 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to native soil when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil of the same or higher capacity as that of the bearing medium.

Tower Footprints: Deep Foundations

Caisson Foundation

End bearing cast-in-place caissons can be used where supplemental axial resistance is required for structural design for the proposed buildings. The caisson should be installed by driving a temporary steel casing and excavating the soil through the casing. A minimum of 35 MPa concrete should be used to in fill the caissons. The piles are to be reinforced structurally over their entire length.

Two alternate design options for drilled shafts are applicable for this site. The first alternative is a caisson installed on the sound rock. The compressive resistance for such piles is directly related to the compressive strength of the bedrock. It is recommended that the entire capacity be derived from the end bearing capacity.

The second alternative is a concrete caisson socketed into bedrock. The axial capacity is increased by the shear capacity of the concrete/rock interface. Furthermore, the tensile resistance of the caisson is increased by the rock capacity. It should be noted that the rock socket should be reinforced.

Table 2 below presents the estimated capacity (factored ULS) for different typical caisson sizes for a rock bearing caisson and rock socketed caisson extending 3 m into sound rock.

Table 2 - Caisson Pile Capacities

Caisson Diameter		Axial Capacity (kN)		Capacity Tension (kN)	
inch	**mm**	**End Bearing**	**Rock Socket**	**End Bearing**	**Rock Socket**
36	900	10,000	14,500	920	2,700
42	1,000	15,000	19,000	1,050	3,450
48	1,200	19,000	24,500	1,200	4,500

notes:
- 3 m rock socket in sound bedrock
- Reinforced caisson and rock socket, when applicable

-0.4 geotechnical factor applied to the shaft capacity

End Bearing Piles

A deep foundation method, such as end bearing piles, can be considered for the footprints of the towers. Concrete filled steel pipe piles driven to refusal on a bedrock surface are a typical deep foundation option in Ottawa.

Applicable pile resistance at SLS values and factored pile resistance at ULS values are provided in Table 3. Additional resistance values can be provided if available pile sizes vary from those detailed in Table 3. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated calculating the Hiley dynamic formula. The piles should be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of four piles is recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles will also be required after at least 48 hours have elapsed since initial driving.

Table 3 - End Bearing Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 25 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	10	975	1460	10	35
245	12	1100	1650	10	42
245	13	1175	1760	10	45

Raft Foundation – Arena Structure

Based on the finish floor level of the arena, the raft foundation is expected to be placed at an elevation of ~58 to 59 m. The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **250 kPa** will be considered acceptable. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal required for the proposed building. The factored bearing resistance (contact pressure) at ULS can be taken as **400 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **14 MPa/m** for a contact pressure of **250 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the following assumptions for the raft foundation, the proposed arena can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed structures as per the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave interpretation are presented in Appendix 2.

Field Program

The shear wave testing was located within the proposed building footprint, as presented in Drawing PG5792-1 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly a east-west orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the centre of the geophone array and 1.6, 3.1 and 9 m away from the first and last geophone.

The methods of testing completed by Paterson are guided by the standard testing procedures used by the expert seismologists at Carleton University and Geological Survey of Canada (GSC).

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The depth to bedrock is known to vary across the site, therefore a conservative estimate of 22 m below ground surface was used for calculation of the V_{s30}.

Overall, the average shear wave velocity through the overburden materials was interpreted to be 387 m/s. Under normal circumstances, the bedrock velocity is interpreted using the main refractor wave velocity, however, this particular test did not provide sufficiently accurate readings to determine a bedrock velocity. In its place, Paterson has assumed a conservative bedrock velocity of 1500 m/s.

The V_{s30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\sum\left(\frac{Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{22m}{387m/s}\right) + \left(\frac{8m}{1500m/s}\right)}$$

$$V_{s30} = 482m/s$$

Based on the results of the seismic testing, the average shear wave velocity of the upper 30 m profile below the proposed underside of foundation, Vs_{30}, was calculated to be **482 m/s**. Therefore, a **Site Class C** is applicable for design of the proposed structures as per OBC 2012.

5.5 Basement Slab Construction

With the removal of all topsoil and deleterious fill within the footprint of the proposed buildings, the native undisturbed silty sand and will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction. Provisions should be made to proof-rolling the soil subgrade using heavy vibratory compaction equipment under dry conditions prior to placing any fill. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Types I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone.

All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils 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. 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.

Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented below.

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 Ko·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 0.281 g 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 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lower level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 below.

Table 4 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
125	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE Fill or OPSS Granular B Type I or II material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

The flexible pavement structure presented in Table 5 should be used for at grade access lanes and heavy loading parking areas overlying the podium deck.

Table 5 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas

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
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type II overlying the Concrete Podium Deck.	

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 I or II material. The pavement granulars (base and subbase) should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Water Suppression System and Foundation Drainage

It is expected that a portion of the foundation walls for the proposed structures will be located below the seasonal high groundwater table. It is suggested that a full water suppression system be constructed for the portions of the foundations placed below the seasonal high groundwater level. The following system is recommended for the proposed structures:

- Where a temporary shoring system is present and a blind-sided pour for the foundation wall is anticipated, the shoring face should be prepared to receive a waterproofing membrane, such as lined bentonite sheets or an elastomeric membrane, followed by a composite drainage board. A waterproofing membrane is recommended for the exterior foundation walls from geodetic elevation 63.0m to the founding elevation. Alternatively, the waterproofing membrane should be placed over the composite drainage board for areas where a double-sided pour is completed and the exterior side of the foundation wall is exposed.

- A composite drainage layer will be placed between the waterproofing membrane and the foundation wall from finished grade to the top of the footing. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) be used. It is expected that 150 mm diameter sleeves placed at 3 m centers 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 basement area.

- The waterproofing membrane should also be extended horizontally along the subgrade surface across the entire footprint along with a suitably sized ballast to resist hydrostatic uplift. The ballast weight is dependent on the depth of foundation below the groundwater level and the full ballast system will be determined once the design details for the proposed structures are finalized. A waterproofing membrane, such as an elastomeric membrane, should be placed over the horizontal subgrade surface. A 75 mm thick lean concrete mud slab should be placed over the approved soil subgrade surface to provide a suitable substrate for placement of the waterproofing membrane.

- Underfloor drainage is required to control water infiltration below the underground level. For design purposes, it is recommended that a 150 mm diameter perforated pipe be placed at 6 m centres. 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.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials, such as site excavated soils, along with the use of a drainage geocomposite, such as Delta Drain 6000 or equivalent, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 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 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

It is expected that the footings along the entrance of the parking garage will not require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

Open Excavation

The side slopes of the anticipated excavation should either be cut back at acceptable slopes or be retained by shoring systems from the beginning of the excavation until the structure is backfilled. However, for most of the site, insufficient room will be available to permit the building excavation to be constructed by open-cut methods (i.e., unsupported excavations).

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 groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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 may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. It is the responsibility of the shoring contractor to ensure that the temporary shoring system is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. Inspections and approval of the temporary system will also be the responsibility of the designer.

Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary shoring system could consist of a soldier pile and lagging system or interlocking steel sheet piles. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced.

The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

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

Table 6 – Soils Parameter for Shoring System Design

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
Unit Weight (γ), kN/m^3	20
Submerged 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 calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level. The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

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

Underpinning of Adjacent Structures

Founding conditions of adjacent structures bordering the footprint of the proposed structures should be assessed and underpinning requirements should be evaluated.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. 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 or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's 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 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.

6.5 Groundwater Control

Groundwater Control for Building Construction

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. Infiltration levels are anticipated to be high through the excavation face for areas where footings are to be placed below the groundwater table (i.e., arena). The groundwater infiltration rate will depend on the depth below the water table. Dewatering methods, such as well points, may be required for areas where footings are to be placed well below the groundwater table (i.e., 1 to 2 m).

Expected Construction Dewatering and Water Taking Rates

Based on our slug testing results, hydraulic conductivity values ranged between 5.9 x 10^{-4} and 7.8 x 10^{-5} m/sec, therefore, a conservative hydraulic conductivity value of **6.0 x 10^{-4} m/sec** was chosen for assessing infiltration rates for excavations below the groundwater table.

Based on our groundwater monitoring program the maximum design groundwater level was measured to be at an elevation of 60.78 m and a maximum excavation elevation of 59.6 m, up to 1.2 m of saturated material is expected to be encountered. Further details regarding dewatering can be found in the Hydrogeological Report (PH4423-1R dated April 24, 2023) completed by Paterson Group, with preliminary construction dewatering volumes presented below in Table 7.

Table 7 – Preliminary Water Taking Rates

Section	Area (m^2)	Steady State Groundwater Dewatering Rate (L/d)	5 year - 1 hour Precipitation Volume (L)
Tower 1 and 2 Underground Parking	11,200	6,750,000 – 7,000,000	300,000
Tower 3- Underground Parking	2,600	6,300,000 – 6,550,000	70,000
Music Hall	1,800	5,000,000 - 5,250,000	50,000
Arena	6,400	7,500,000 – 7,750,000	170,000
Building OPS/Storage	1,200	4,500,000 – 4,750,000	32,500

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) will be required for this project as 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.

Impacts on Neighboring Properties

A local groundwater lowering is anticipated under short-term conditions to accommodate the construction of the proposed buildings. Based on the proximity of neighboring buildings and subsoil properties, the proposed development will not negatively impact the neighboring structures.

Due to the proposed water suppression system to be installed for each structure, no issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed building.

Surface Water and Groundwater

A number of MECP Brownfields Environmental Site Registry sites were located within 500 m of the subject site. However, all but one of them indicated that no groundwater remediation was required during cleanup. Of the sites that required a groundwater monitoring program, annual reports indicate that the reported parameter concentrations are below the 2011 Table 3 SCS for the property.

The groundwater removed from site excavations must be managed in an appropriate manner, and the contractor will be required to implement a water management program to dispose of the pumped water. It is expected that the groundwater will be discharged to the City of Ottawa sewer system in accordance with City Sewer Use By-Laws. The City of Ottawa will determine the appropriate discharge location (sanitary or storm sewer) dependent upon the results of the baseline testing performed for the discharge permit application.

With respect to nearby surface water bodies, the Rideau Canal is located approximately 150 m south and east of the property, however, the Rideau Canal is outside the theoretical radius of influence (approximately 80 - 90 m) and the anticipated water taking volumes are considered negligible compared to the expected daily flows from the Rideau Canal. As such, adverse effects to surface water features resulting from dewatering activities at the subject site are expected to be negligible.

Long-term Groundwater Control

Any groundwater encountered along the perimeter of the building or sub-slab drainage system will be directed to the cistern/sump pit of the proposed structures.

Provided the proposed groundwater suppression system is properly implemented and approved by the geotechnical consultant at the time of construction, the expected long-term groundwater flow should be low (i.e., less than 40,000 L/day/building) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction once groundwater infiltration levels are observed. The long-term groundwater flow is anticipated to be controllable using conventional open sumps.

No issues are expected with respect to groundwater lowering that would cause long term adverse effects to adjacent structures surrounding the proposed structures.

6.6 Winter Construction

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

The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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.

Precautions must be taken where excavations are carried out in proximity of existing structures which may 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 and 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.

6.7 Corrosion Potential and Sulphate

The results of analytical testing from an adjacent site 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 a moderate to severe corrosive environment.

7.0 Recommendations

It is recommended that the following be completed by Paterson once the final master plan and site development are determined:

- Review of geotechnical aspects of the excavating program, prior to construction.
- Review of the waterproofing details for the elevator shaft and building sump pits.
- Inspection of the installation of the waterproofing and perimeter and underground floor drainage system during construction.
- If applicable, inspection of end bearing piles/caisson installation.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- 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.
- 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 the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site, must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests 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 the 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 City of Ottawa 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
June 28, 2023
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B. Eng.

David J. Gilbert, P.Eng.

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

PHOTOGRAPHS OF ROCK CORE

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Redevelopment
Prop. Multi-Storey Buildings & Rink Structure, Ontario

DATUM Geodetic

FILE NO. **PG5792**

REMARKS

HOLE NO. **BH 1-21**

BORINGS BY CME-55 Low Clearance Drill

DATE October 25, 2021

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.93
Asphaltic concrete	0.10						
FILL: Crushed stone, trace sand	0.41	AU	1				
FILL: Brown silty sand with topsoil	0.53						
FILL: Brown silty sand to sandy silt, some clay, trace topsoil		SS	2	50	64	1	63.93
		SS	3	58	28		
	2.19					2	62.93
Compact, brown **SILTY SAND** - trace clay from 3.0 to 4.3m depth - trace gravel by 4.3m depth		SS	4	42	13		
						3	61.93
		SS	5	25	14		
		SS	6	50	15	4	60.93
		SS	7	33	20		
						5	59.93
	5.49						
GLACIAL TILL: Very dense to compact, brown silty sand with gravel, cobbles and boulders		SS	8	50	53		
						6	58.93
		SS	9	42	32		
		SS	10	33	31	7	57.93
		SS	11	25	26		
						8	56.93
		SS	12	42	21		
		SS	13	42	29	9	55.93
						10	54.93
		SS	14	33	39		
		SS	15		65		
	11.10					11	53.93
End of Borehole (GWL @ 5.09m - Nov. 12, 2021)							

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Redevelopment
Prop. Multi-Storey Buildings & Rink Structure, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 25, 2021

FILE NO. PG5792

HOLE NO. BH 2-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.04
Asphaltic concrete 0.10							
FILL: Brown silty sand with crushed stone and gravel 0.36		AU	1				
FILL: Brown silty sand, trace gravel		SS	2	33	32	1	65.04
2.21		SS	3	50	7	2	64.04
Compact, brown **SILTY SAND**		SS	4	50	14		
		SS	5	33	10	3	63.04
		SS	6	33	11	4	62.04
- trace gravel by 4.4m depth		SS	7	42	24	5	61.04
5.74		SS	8	25	59		
GLACIAL TILL: Very dense to dense, brown silty sand with gravel, cobbles and boulders		SS	9	63	50+	6	60.04
		SS	10	50	77	7	59.04
		SS	11	42	46	8	58.04
		SS	12	0	63		
		SS	13	8	61	9	57.04
- some shale fragments from 10.5 to 10.74m depth		SS	14		50+	10	56.04
10.74							
End of Borehole							

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Redevelopment
Prop. Multi-Storey Buildings & Rink Structure, Ontario

DATUM Geodetic

FILE NO. PG5792

REMARKS

HOLE NO. BH 3-21

BORINGS BY CME-55 Low Clearance Drill

DATE October 27, 2021

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL (to 0.36)				
FILL: Brown silty sand, some gravel, occasional cobble and boulders, trace clay and topsoil	AU	1		
	SS	2	33	16
	SS	3	22	50+
	SS	4	17	11
	SS	5	44	50+
- cored through boulder from 3.28 to 3.81m depth	RC	1	95	
	SS	6	33	6
	SS	7	33	47
- trace ash from 5.3 to 5.9m depth	SS	8	25	50+
	SS	9	25	59
- trace asphaltic concrete from 7.0 to 7.6m depth	SS	10	25	38
	SS	11	0	50+
(to 9.45)	SS	12	33	34
Compact, brown **SILTY SAND to SANDY SILT**	SS	13	50	14
	SS	14	58	22
(to 11.40)	SS	15	50	28
Compact, brown **SILTY SAND,** some gravel	SS	16	33	17

DEPTH (m)	ELEV. (m)
0	73.10
1	72.10
2	71.10
3	70.10
4	69.10
5	68.10
6	67.10
7	66.10
8	65.10
9	64.10
10	63.10
11	62.10
12	61.10

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

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SOIL PROFILE AND TEST DATA

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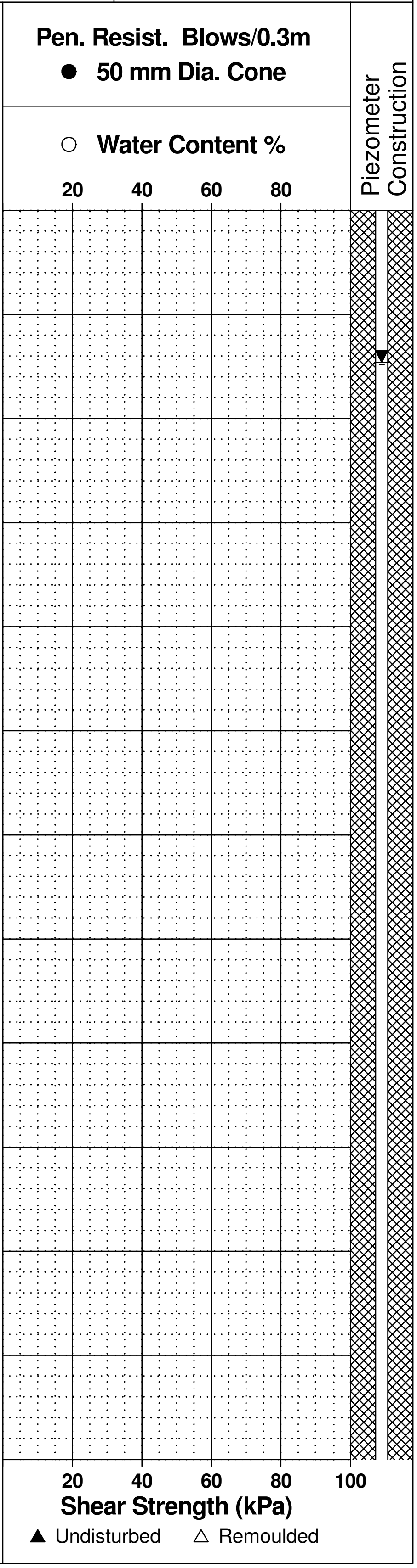
FILE NO. PG5792

HOLE NO. BH 3-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Compact, brown **SILTY SAND,** some gravel	SS	17	33	19
	SS	18	25	18
	SS	19	4	12
	SS	20	4	21
15.54				
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders	SS	21	50	36
	SS	22	67	60
	SS	23	33	50+
	RC	2	70	
	SS	24	4	50+
	RC	3	64	
- grey by 20.2m depth	RC	4	52	
- compact by 21.3m depth	RC	5	30	
	RC	6	13	

DEPTH (m)	ELEV. (m)
12	61.10
13	60.10
14	59.10
15	58.10
16	57.10
17	56.10
18	55.10
19	54.10
20	53.10
21	52.10
22	51.10
23	50.10
24	49.10



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DATE October 27, 2021

FILE NO. PG5792

HOLE NO. BH 3-21

GROUND SURFACE

GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders

- cobbles and boulders content decreasing with depth

31.57

BEDROCK: Good to excellent quality, grey limestone with occasional shale partings

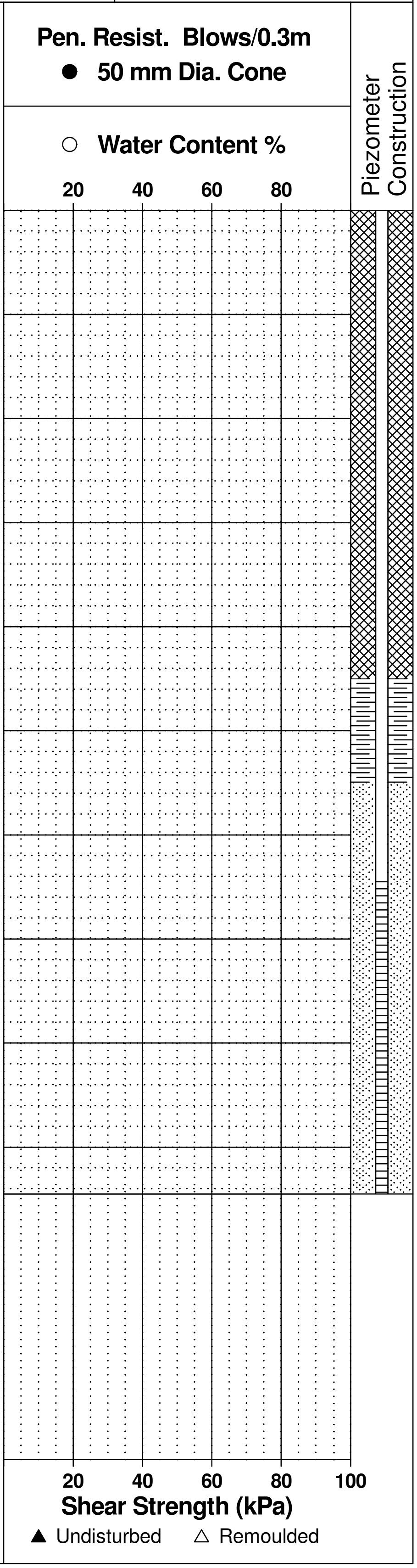
33.45

End of Borehole

(GWL @ 13.46m - Nov. 16, 2021)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
RC	7	8	
RC	8	0	
RC	9	0	
RC	10	0	
RC	11	100	71
RC	12	100	98

DEPTH (m)	ELEV. (m)
24	49.10
25	48.10
26	47.10
27	46.10
28	45.10
29	44.10
30	43.10
31	42.10
32	41.10
33	40.10



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Redevelopment
Prop. Multi-Storey Buildings & Rink Structure, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

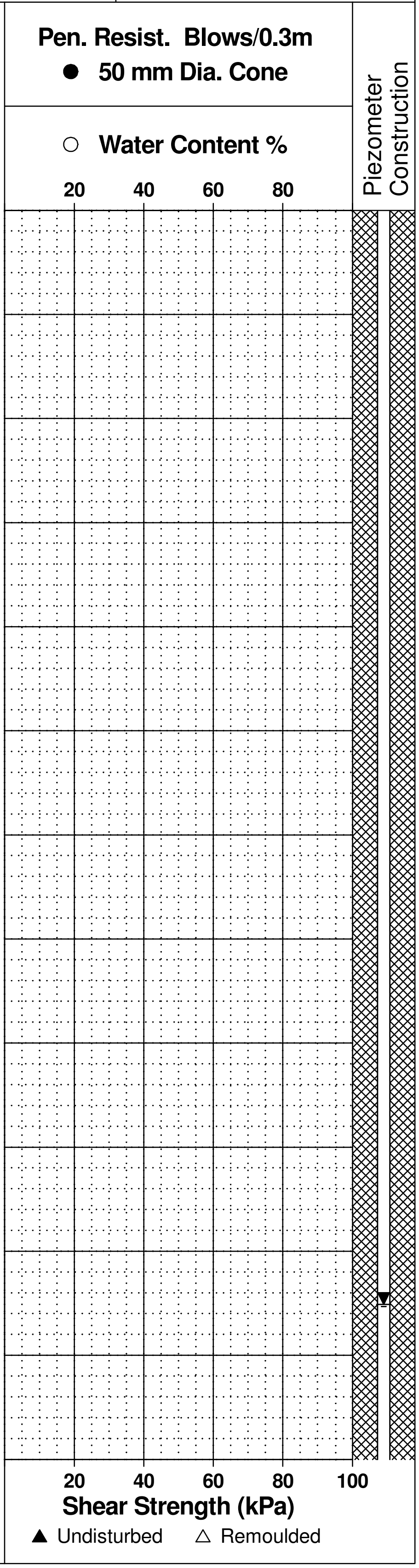
DATE November 5, 2021

FILE NO. PG5792

HOLE NO. BH 4-21

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30				
FILL: Brown silty sand iwth gravel and cobbles, occasional boulders, trace clay - some topsoil from 5.3 to 5.9m depth - some asphaltic concrete from 7.6 to 8.2m depth		AU	1		
		SS	2	33	5
		SS	3	58	49
		SS	4	50	10
		SS	5	50	8
		SS	6	50	8
		SS	7	42	46
		SS	8	33	28
		SS	9	50	19
		SS	10	18	9
	8.53	SS	11		50+
Compact, brown **SILTY SAND to SANDY SILT**		SS	12	58	13
		SS	13		14
		SS	14	42	19
	11.25	SS	15	50	18
GLACIAL TILL: Very dense to dense, silty sand with gravel, cobbles and boulders		SS	16	33	59

DEPTH (m)	ELEV. (m)
0	72.75
1	71.75
2	70.75
3	69.75
4	68.75
5	67.75
6	66.75
7	65.75
8	64.75
9	63.75
10	62.75
11	61.75
12	60.75



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Redevelopment
Prop. Multi-Storey Buildings & Rink Structure, Ontario

DATUM Geodetic

FILE NO. PG5792

REMARKS

HOLE NO. BH 4-21

BORINGS BY CME-55 Low Clearance Drill **DATE** November 5, 2021

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					12	60.75
					13	59.75
	SS	17	60	50+		
					14	58.75
	RC	1	33			
					15	57.75
	RC	2	41			
					16	56.75
GLACIAL TILL: Very dense to dense, silty sand with gravel, cobbles and boulders	SS	18	75	50+		
					17	55.75
	RC	3	34			
					18	54.75
	RC	4	24			
					19	53.75
	SS	19	0	50+		
					20	52.75
	RC	5	7			
- grey by 20.8m depth						
					21	51.75
	SS	20	42	15		
	RC	6	0			
					22	50.75
	SS	21	0	50+		
					23	49.75
	RC	7	20			
					24	48.75

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Redevelopment
Prop. Multi-Storey Buildings & Rink Structure, Ontario

DATUM Geodetic

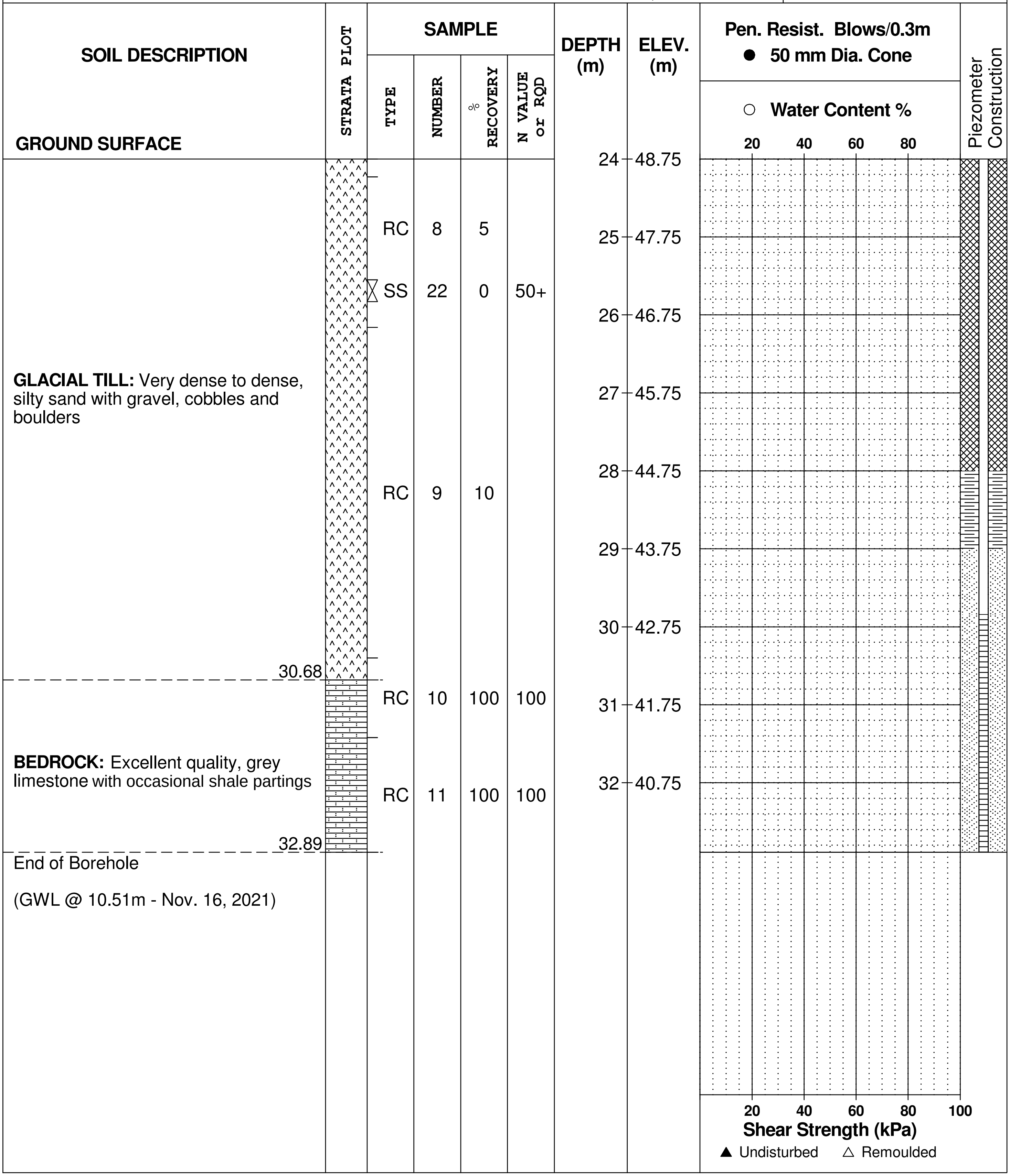
FILE NO. PG5792

REMARKS

HOLE NO. BH 4-21

BORINGS BY CME-55 Low Clearance Drill

DATE November 5, 2021



SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
					24	48.75
GLACIAL TILL: Very dense to dense, silty sand with gravel, cobbles and boulders	RC	8	5			
					25	47.75
	SS	22	0	50+		
					26	46.75
					27	45.75
					28	44.75
	RC	9	10			
					29	43.75
					30	42.75
30.68						
BEDROCK: Excellent quality, grey limestone with occasional shale partings	RC	10	100	100		
					31	41.75
	RC	11	100	100		
					32	40.75
32.89						
End of Borehole						
(GWL @ 10.51m - Nov. 16, 2021)						

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Redevelopment
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DATUM Geodetic

FILE NO. PG5792

REMARKS

HOLE NO. BH 5-21

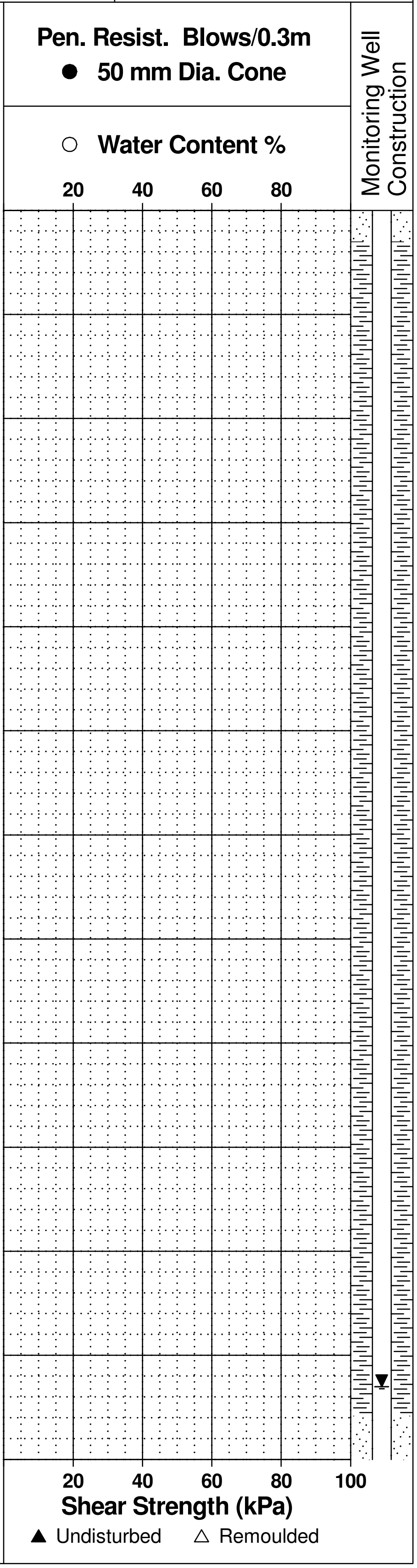
BORINGS BY CME 55 Power Auger

DATE November 9, 2021

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.36
FILL: Brown silty sand with gravel, occasional cobbles - trace topsoil and concrete from 2.3 to 2.9m depth - with asphaltic concrete by 6.1m depth	6.70
Compact to dense, brown **SILTY SAND** - some gravel by 8.5m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	63	50+
SS	3	50	19
SS	4	50	15
SS	5	0	14
SS	6	25	13
SS	7	0	50+
SS	8	58	43
SS	9	67	15
SS	10	50	14
SS	11	42	17
SS	12	50	34
SS	13	42	47
SS	14	50	48
SS	15	88	50+
SS	16	50	35

DEPTH (m)	ELEV. (m)
0	71.14
1	70.14
2	69.14
3	68.14
4	67.14
5	66.14
6	65.14
7	64.14
8	63.14
9	62.14
10	61.14
11	60.14
12	59.14



SOIL PROFILE AND TEST DATA

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REMARKS

HOLE NO. BH 5-21

BORINGS BY CME 55 Power Auger

DATE November 9, 2021

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Compact to dense, brown **SILTY SAND,** some gravel	SS	17	21	9
	SS	18	50	23
	SS	19	50	28
14.20				
GLACIAL TILL: Very dense to dense, brown silty sand with gravel, cobbles and boulders - grey by 18.2m depth	SS	20	55	50+
	RC	1	60	
	SS	21	42	71
	RC	2	22	
	SS	22	64	38
	RC	3	15	
	SS	23	100	50+
	RC	4	15	
	SS	24	0	50+
	RC	5	19	

DEPTH (m)	ELEV. (m)
12	59.14
13	58.14
14	57.14
15	56.14
16	55.14
17	54.14
18	53.14
19	52.14
20	51.14
21	50.14
22	49.14
23	48.14
24	47.14

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

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REMARKS

HOLE NO. BH 5-21

BORINGS BY CME 55 Power Auger

DATE November 9, 2021

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					24	47.14
GLACIAL TILL: Very dense to dense, brown silty sand with gravel, cobbles and boulders	SS	25	80	50+		
	RC	6	0		25	46.14
	SS	26	0	50+	26	45.14
	RC	7	0			
	SS	27	86	50+	27	44.14
	RC	8	37		28	43.14
	SS	28	0	10	29	42.14
29.95	RC	9	100	100	30	41.14
BEDROCK: Excellent quality, grey limestone with occasional shale partings	RC	10	100	93	31	40.14
31.55						

End of Borehole

(GWL @ 11.30m - Nov. 16, 2021)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

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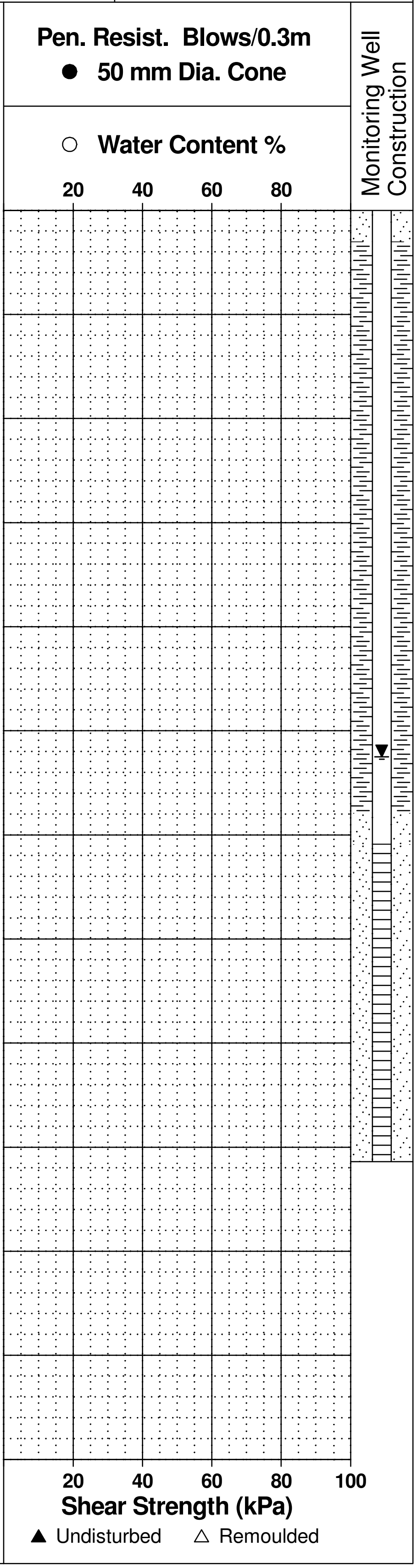
BORINGS BY CME-55 Low Clearance Drill

DATE November 11, 2021

FILE NO. PG5792

HOLE NO. BH 6-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	65.14
Asphaltic concrete 0.08						
FILL: Brown silty sand with crushed stone and gravel 0.91	SS	1	67	47		
					1	64.14
	SS	2	42	26		
	SS	3	50	17		
					2	63.14
	SS	4	58	13		
					3	62.14
Compact to dense, brown **SILTY SAND,** trace to some gravel	SS	5	50	43		
	SS	6	50	13	4	61.14
	SS	7	50	50+		
5.41					5	60.14
	SS	8	50	50+		
					6	59.14
	SS	9	42	34		
	SS	10	42	35	7	58.14
GLACIAL TILL: Dense brown silty sand with gravel, cobbles and boulders	SS	11	50	34		
					8	57.14
	SS	12	43	78		
- silty sand to sandy silt layer from 8.9 to 9.3m depth					9	56.14
	SS	13	50	43		
					10	55.14
	SS	14	42	38		
	SS	15	43	50+		
	RC	1	61		11	54.14
- grey by 12.2m depth	SS	16	40	50+		
	RC	2	75			
					12	53.14



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SOIL PROFILE AND TEST DATA

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Prop. Multi-Storey Buildings & Rink Structure, Ontario

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REMARKS

HOLE NO. BH 6-21

BORINGS BY CME-55 Low Clearance Drill

DATE November 11, 2021

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
GLACIAL TILL: Dense, grey silty sand with gravel, cobbles and boulders - some clay by 16.8m depth	SS	17		50+
	RC	3	34	
	SS	18	52	41
	RC	4	19	
	SS	19	86	50+
	RC	5	0	
	SS	20	50	28
	RC	6	11	
	SS	21	0	50+
	RC	7	14	
	SS	22	0	50+
	RC	8	35	
22.88				
BEDROCK: Good to excellent quality, grey limestone with occasional shale partings	RC	9	100	85

DEPTH (m)	ELEV. (m)
12	53.14
13	52.14
14	51.14
15	50.14
16	49.14
17	48.14
18	47.14
19	46.14
20	45.14
21	44.14
22	43.14
23	42.14
24	41.14

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

○ **Water Content %**

20 40 60 80

Monitoring Well Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

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FILE NO. PG5792

HOLE NO. BH 6-21

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	Monitoring Well Construction
GROUND SURFACE									
						24	41.14		
BEDROCK: Good to excellent quality, grey limestone with occasional shale partings		RC	10	100	98	25	40.14		
25.73									
End of Borehole (GWL @ 5.25m - Nov. 16, 2021)									

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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BORINGS BY CME-55 Low Clearance Drill

DATE November 15, 2021

FILE NO. PG5792

HOLE NO. BH 7-21

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.25
FILL: Brown silty sand, some gravel	1.93
Compact to dense, brown **SILTY SAND,** trace gravel	4.42
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	
- some shale fragments from 11.0 to 11.5m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	50	27
SS	2	0	48
SS	3	50	50+
SS	4	50	50+
RC	1	45	
SS	5	53	50+
SS	6	0	50+
RC	2	56	
RC	3	33	
SS	7	42	53

DEPTH (m)	ELEV. (m)
0	66.62
1	65.62
2	64.62
3	63.62
4	62.62
5	61.62
6	60.62
7	59.62
8	58.62
9	57.62
10	56.62
11	55.62
12	54.62

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

○ **Water Content %**

20 40 60 80

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

Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Redevelopment
Prop. Multi-Storey Buildings & Rink Structure, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 15, 2021

FILE NO. PG5792

HOLE NO. BH 7-21

SOIL DESCRIPTION

GROUND SURFACE

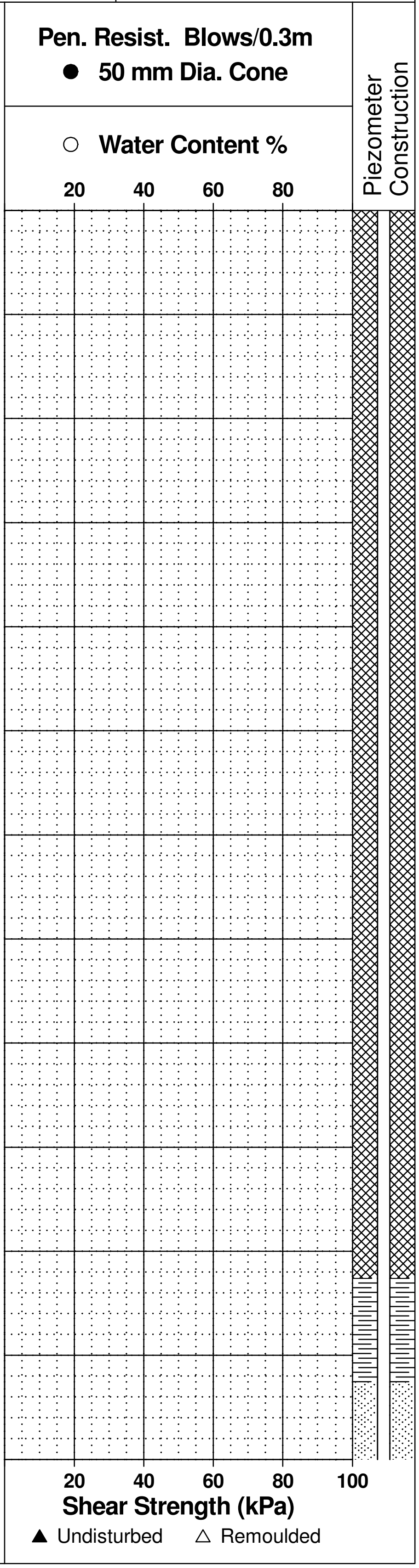
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders

- grey by 13.7m depth

23.80

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
RC	4	48	
SS	8	33	48
RC	5	47	
SS	9	33	50+
RC	6	0	
SS	10	0	50+
RC	7	30	
SS	11	73	50+
RC	8	12	
SS	12	77	50+
RC	9	18	
SS	13	0	50+
RC	10	100	100

DEPTH (m)	ELEV. (m)
12	54.62
13	53.62
14	52.62
15	51.62
16	50.62
17	49.62
18	48.62
19	47.62
20	46.62
21	45.62
22	44.62
23	43.62
24	42.62



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SOIL PROFILE AND TEST DATA

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REMARKS

HOLE NO. BH 7-21

BORINGS BY CME-55 Low Clearance Drill

DATE November 15, 2021

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					24	42.62
BEDROCK: Excellent quality, grey limestone with occasional shale partings	RC	11	100	100	25	41.62
	RC	12	100	94	26	40.62
					27	39.62
27.26 End of Borehole						
(BH dry - November 16, 2021)						

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

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SOIL PROFILE AND TEST DATA

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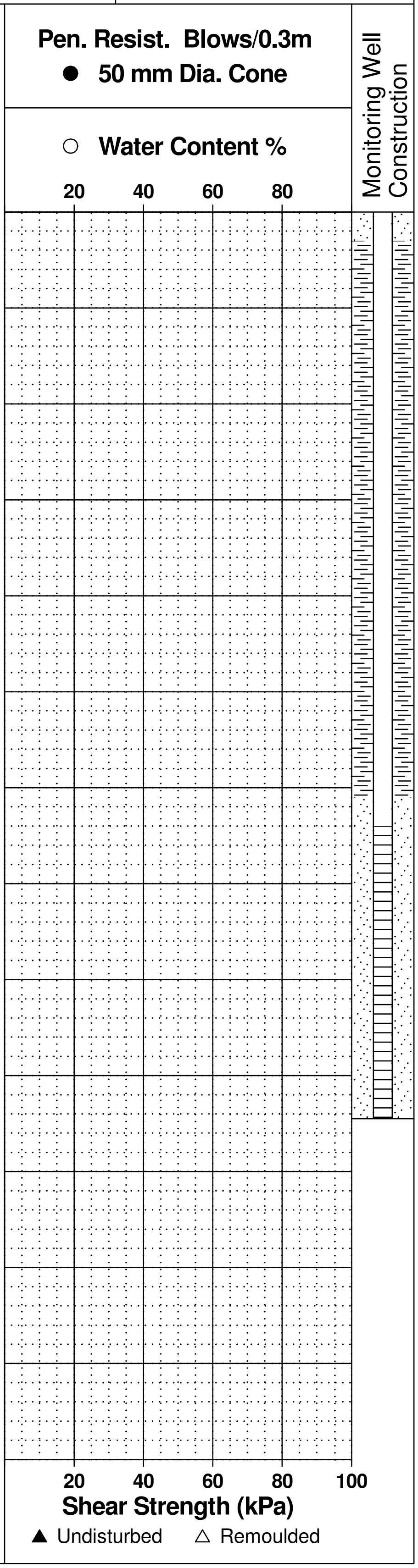
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE November 17, 2021
FILE NO. PG5792
HOLE NO. BH 8-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	65.45
Concrete patio stone 0.15		AU	1				
FILL: Crushed stone 0.46							
FILL: Brown silty sand with gravel, occasional cobbles		SS	2	42	20	1	64.45
		SS	3	0	15		
2.03						2	63.45
Compact to dense, brown silty sand, some gravel		SS	4	0	8		
		SS	5	17	37	3	62.45
		SS	6	42	41	4	61.45
		SS	7	50	57		
5.13						5	60.45
Dense, brown **SILTY SAND**		SS	8	42	36		
		SS	9	50	40	6	59.45
		SS	10	50	36	7	58.45
- some gravel, occasional cobbles and boudlers by 7.4m depth		SS	11	58	47		
						8	57.45
		SS	12	50	41		
8.89							
Dense, brown **SILTY SAND to SANDY SILT,** some gravel		SS	13	67	36	9	56.45
		SS	14		45	10	55.45
		SS	15	67	69		
11.18						11	54.45
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders		SS	16	67	43		
		SS	17	50	14	12	53.45
						13	52.45



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REMARKS

HOLE NO. BH 8-21

BORINGS BY CME-55 Low Clearance Drill **DATE** November 17, 2021

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	RC	1	55		13	52.45
					14	51.45
	RC	2	30			
					15	50.45
	SS	18	58	28		
					16	49.45
	RC	3	0			
	SS	19	0	50+		
					17	48.45
	RC	4	36			
					18	47.45
	SS	20	25	50+		
	RC	5	50			
					19	46.45
	SS	21	0			
					20	45.45
	RC	6	35			
					21	44.45
21.28	SS	22		50+		
BEDROCK: Excellent quality, grey limestone with occasional shale partings	RC	7	100	90		
					22	43.45
					23	42.45
	RC	8	100	95		
24.10					24	41.45
End of Borehole						

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

○ **Water Content %**

20 40 60 80

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

Monitoring Well Construction

SOIL PROFILE AND TEST DATA

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Lansdowne Park Redevelopment
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REMARKS

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DATE November 18, 2021

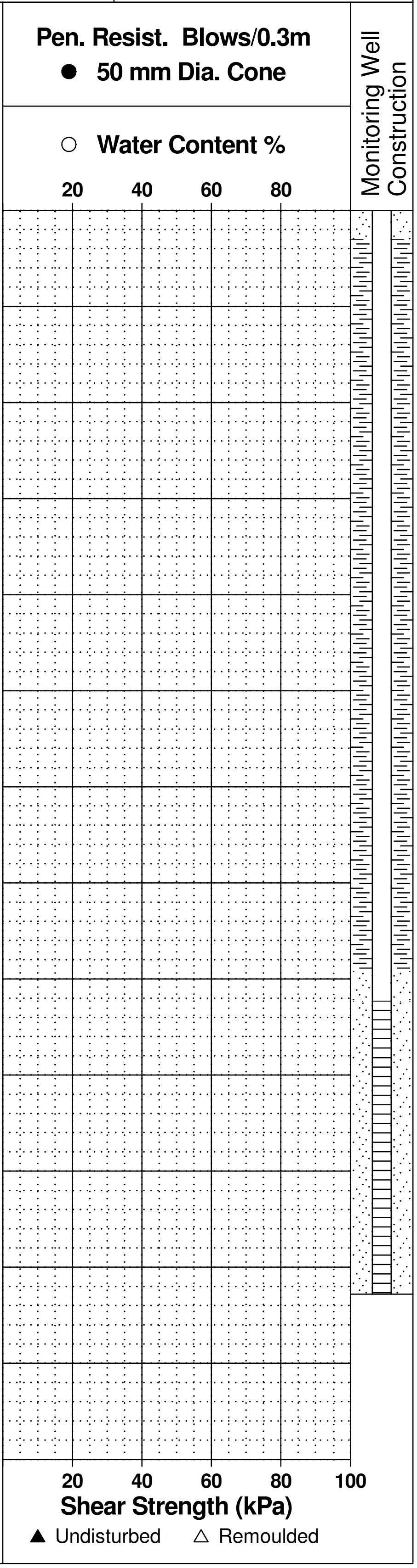
FILE NO. PG5792

HOLE NO. BH 9-21

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Concrete	0.15
FILL: Brown silty sand with crushed stone	0.46
FILL: Brown silty sand with gravel, occasional cobbles	4.34
Concrete (inferred footing)	4.75
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	17	18
SS	3	8	17
RC	1	63	
SS	4	42	6
SS	5		50+
RC	2	16	
SS	6	45	50+
RC	3	46	
SS	7	0	50+
SS	8	50	58
RC	4	42	
SS	9	25	43
SS	10	0	50+
SS	11	60	50+
RC	5	13	

DEPTH (m)	ELEV. (m)
0	67.07
1	66.07
2	65.07
3	64.07
4	63.07
5	62.07
6	61.07
7	60.07
8	59.07
9	58.07
10	57.07
11	56.07
12	55.07
13	54.07



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SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					13	54.07
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	SS	12	22	50+		
	RC	6	70		14	53.07
					15	52.07
	RC	7	37		16	51.07
	RC	8	25		17	50.07
	SS	13	0	50+	18	49.07
	RC	9	48		19	48.07
	RC	10	11		20	47.07
21.36					21	46.07
BEDROCK: Excellent quality, grey limestone with occasional shale partings	RC	11	100	90	22	45.07
					23	44.07
	RC	12	100	100		
24.08					24	43.07
End of Borehole						

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

○ Water Content %

20 40 60 80

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

Monitoring Well Construction

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 strength of cohesionless soils is the relative density, 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.

Relative Density	‘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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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 is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

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 NXL size core. However, it can be used on smaller core sizes, such as BX, 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
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture 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 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 = $(D30)^2 / (D10 \times D60)$
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)

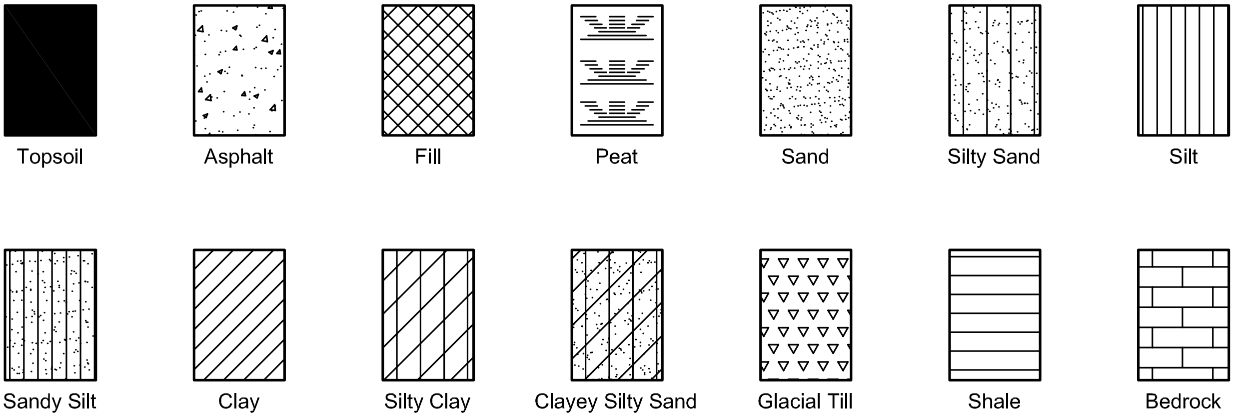
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

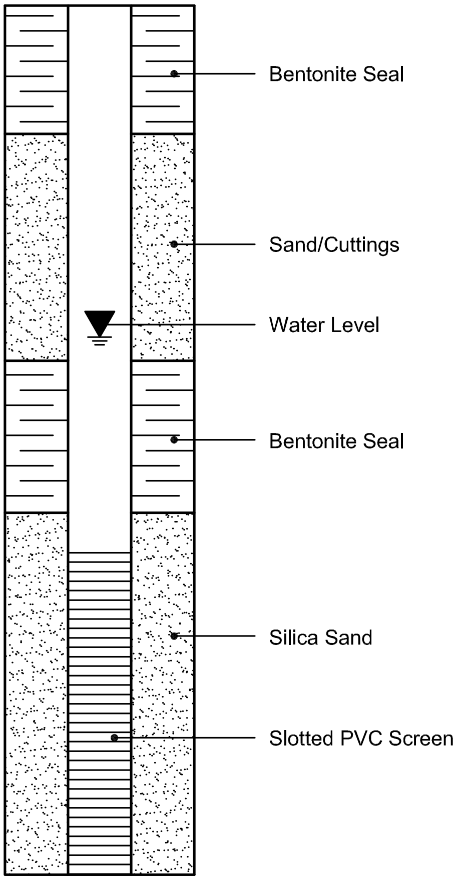
SYMBOLS AND TERMS (continued)

STRATA PLOT

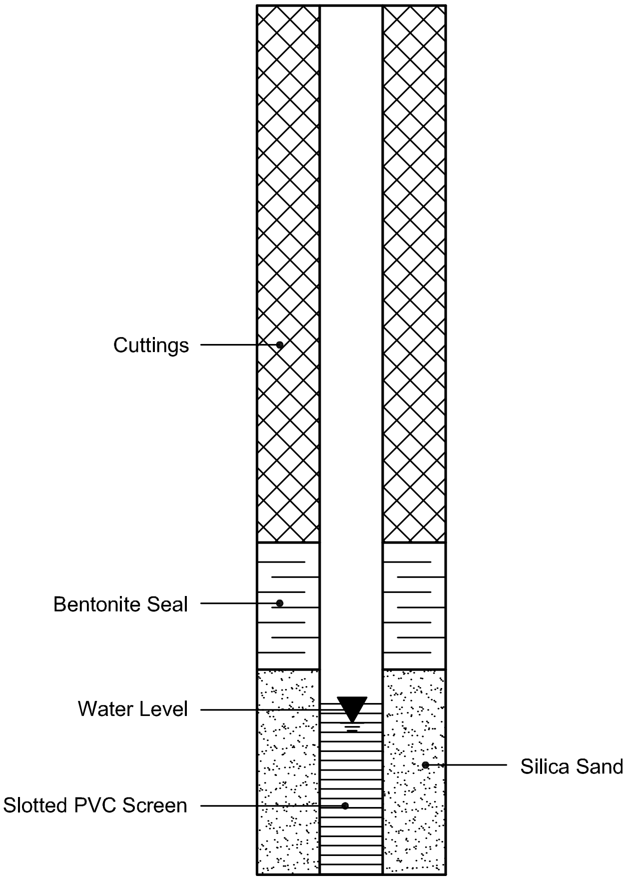


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Stratigraphic and Instrumentation Log: MW15-6 / GP15-10

Project No: TZ10100106
Location: 945 Bank Street, Ottawa
Logged By: JFT
Drill Date: October 21, 2015
Hole Size: 127 mm

Project Name: CPU Ground Water Monitoring Program
Client: City of Ottawa
Entered By: KYLT
Drill Method: Direct Push
Drilled By: Strata Drilling Group

Amec Foster Wheeler
300-210 Colonnade Road
Ottawa, Ontario K2E 7L5

SUBSURFACE PROFILE

Description	Elevation (m)
Ground Surface	64.9
TOPSOIL	0.0
	64.5
FILL Fine grained loamy sand, trace gravel, dark brown	0.4
Very fine grained sandy loam, dark brown, moist	
Brownish grey, wet	
Fine to medium grained sand, grey	
Trace gravel	
Fine to medium grained sandy loam and gravel	60.2
SAND Fine to coarse grained sand, trace gravel	4.7
	59.7
END OF BOREHOLE	5.2

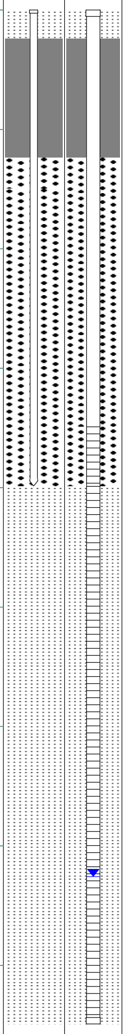
SAMPLE DATA

Type	Number	N or RQD	Recovery (%)
SS			
SS	1		45
SS	2		65
SS	3		43

Combustible Vapour (ppm) ○ 20 40 60 80 ○

Total Organic Vapour (ppm) ● 20 40 60 80 ●

WELLS



Remarks

Elevation: 64.924 masl
Easting: 368843.807
Northing: 5029183.520

Casing Elevation: 64.615 masl
Well Casing Size: MW 50.8 mm/GP 12.7 mm
Screen Slot Size: MW 0.25 mm/GP 6.4 mm

Filter Pack Size: MW 6.7 mm/GP 9.5 mm
Well Material: Schedule 40 PVC
Vapour Unit: N/A

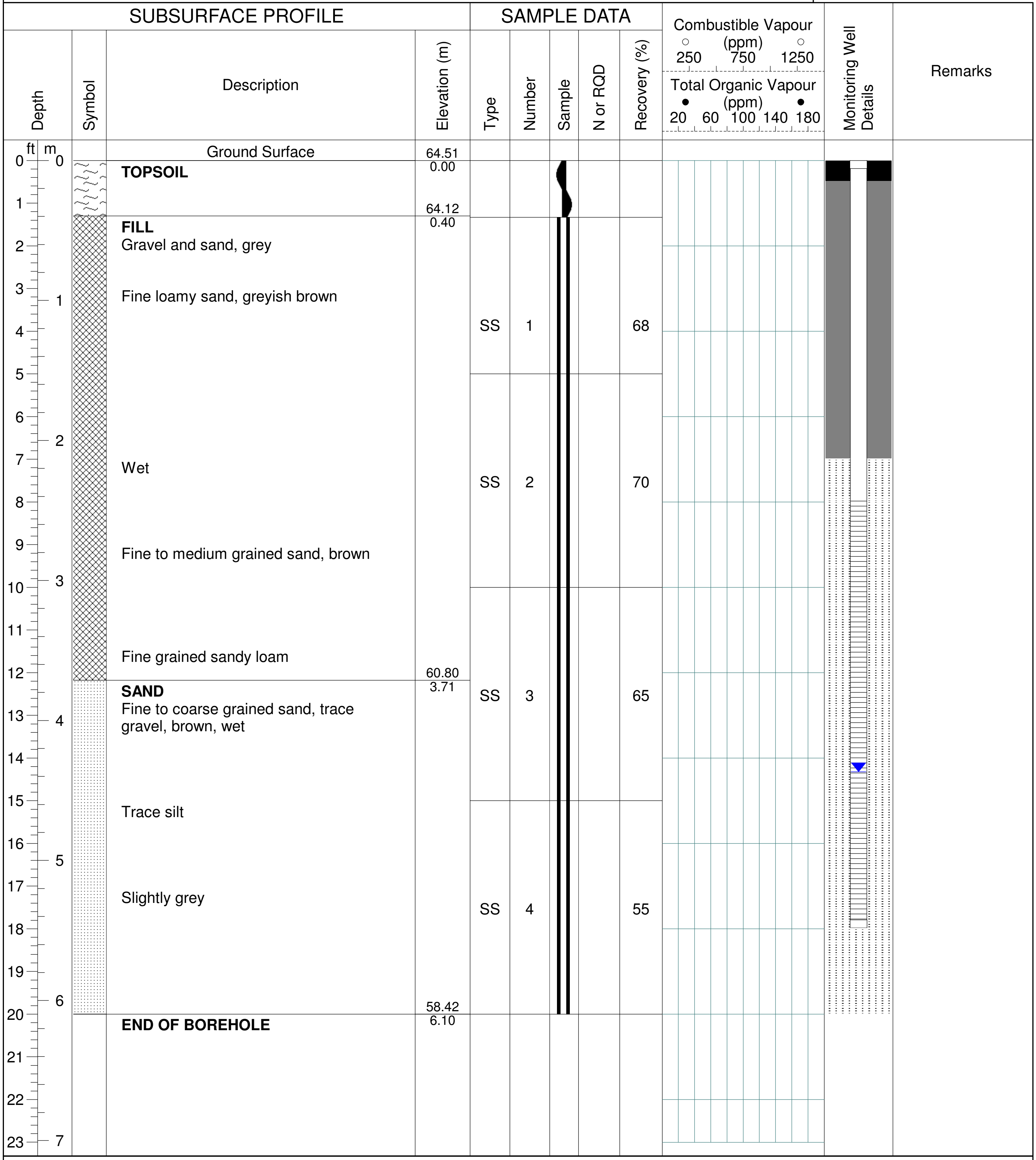
Datum: Geodetic
Checked by: KDH
Sheet: 1 of 1

Stratigraphic and Instrumentation Log: MW15-7

Project No: TZ10100106
Location: 945 Bank Street, Ottawa
Logged By: JFT
Drill Date: October 21, 2015
Hole Size: 127 mm

Project Name: CPU Ground Water Monitoring Program
Client: City of Ottawa
Entered By: KYLT
Drill Method: Direct Push
Drilled By: Strata Drilling Group

Amec Foster Wheeler
300-210 Colonnade Road
Ottawa, Ontario K2E 7L5



Description	Elevation (m)	Type	Number	Sample	N or RQD	Recovery (%)
Ground Surface	64.51					
TOPSOIL	0.00					
FILL Gravel and sand, grey	64.12 0.40					
Fine loamy sand, greyish brown		SS	1			68
Wet		SS	2			70
Fine to medium grained sand, brown						
Fine grained sandy loam						
SAND Fine to coarse grained sand, trace gravel, brown, wet	60.80 3.71	SS	3			65
Trace silt						
Slightly grey		SS	4			55
END OF BOREHOLE	58.42 6.10					

Elevation: 64.513 masl
Easting: 368911.901
Northing: 5029169.410

Casing Elevation: 64.431 masl
Well Casing Size: 50.8 mm
Screen Slot Size: 0.25 mm

Filter Pack Size: 6.7 mm
Well Material: Schedule 40 PVC
Vapour Unit: N/A

Datum: Geodetic
Checked by: KDH
Sheet: 1 of 1

Stratigraphic and Instrumentation Log: MW15-9

Amec Foster Wheeler
300-210 Colonnade Road
Ottawa, Ontario K2E 7L5

Project No: TZ10100106
Location: 945 Bank Street, Ottawa
Logged By: JFT
Drill Date: October 21, 2015
Hole Size: 127 mm

Project Name: CPU Ground Water Monitoring Program
Client: City of Ottawa
Entered By: KYLT
Drill Method: Direct Push
Drilled By: Strata Drilling Group

SUBSURFACE PROFILE

Depth (ft / m)	Description	Elevation (m)
0 / 0	Ground Surface	65.25
0 / 0	**ASPHALT**	0.00
1.3 / 0.40	**FILL** Fine to medium grained loamy sand, trace gravel, brown Fine to medium grained sand, trace coarse grained sand, brown Brownish grey Damp/moist Fine to medium grained sand Medium to coarse grained sand, moist/wet Very fine to fine grained sand, grey	64.86 0.40
15 / 4.57	**SAND** Fine to coarse grained sand, trace gravel, grey, wet	60.68 4.57
17 / 5.18	**LOAMY SAND** Fine to medium grained loamy sand and gravel, some pieces of rock	60.07 5.18
20 / 6.10	**END OF BOREHOLE**	59.16 6.10

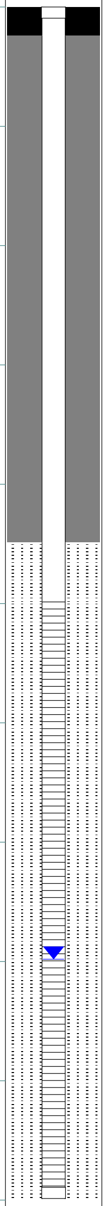
SAMPLE DATA

Type	Number	Sample	N or RQD	Recovery (%)
SS	1			68.1
SS	2			70
SS	3			65
SS	4			55

Combustible Vapour (ppm) ○ 250 750 1250

Total Organic Vapour (ppm) ● 20 60 100 140 180

Monitoring Well Details



Remarks

Elevation: 65.253 masl
Easting: 368798.392
Northing: 5029125.377

Casing Elevation: 65.148 masl
Well Casing Size: 50.8 mm
Screen Slot Size: 0.25 mm

Filter Pack Size: 6.7 mm
Well Material: Schedule 40 PVC
Vapour Unit: N/A

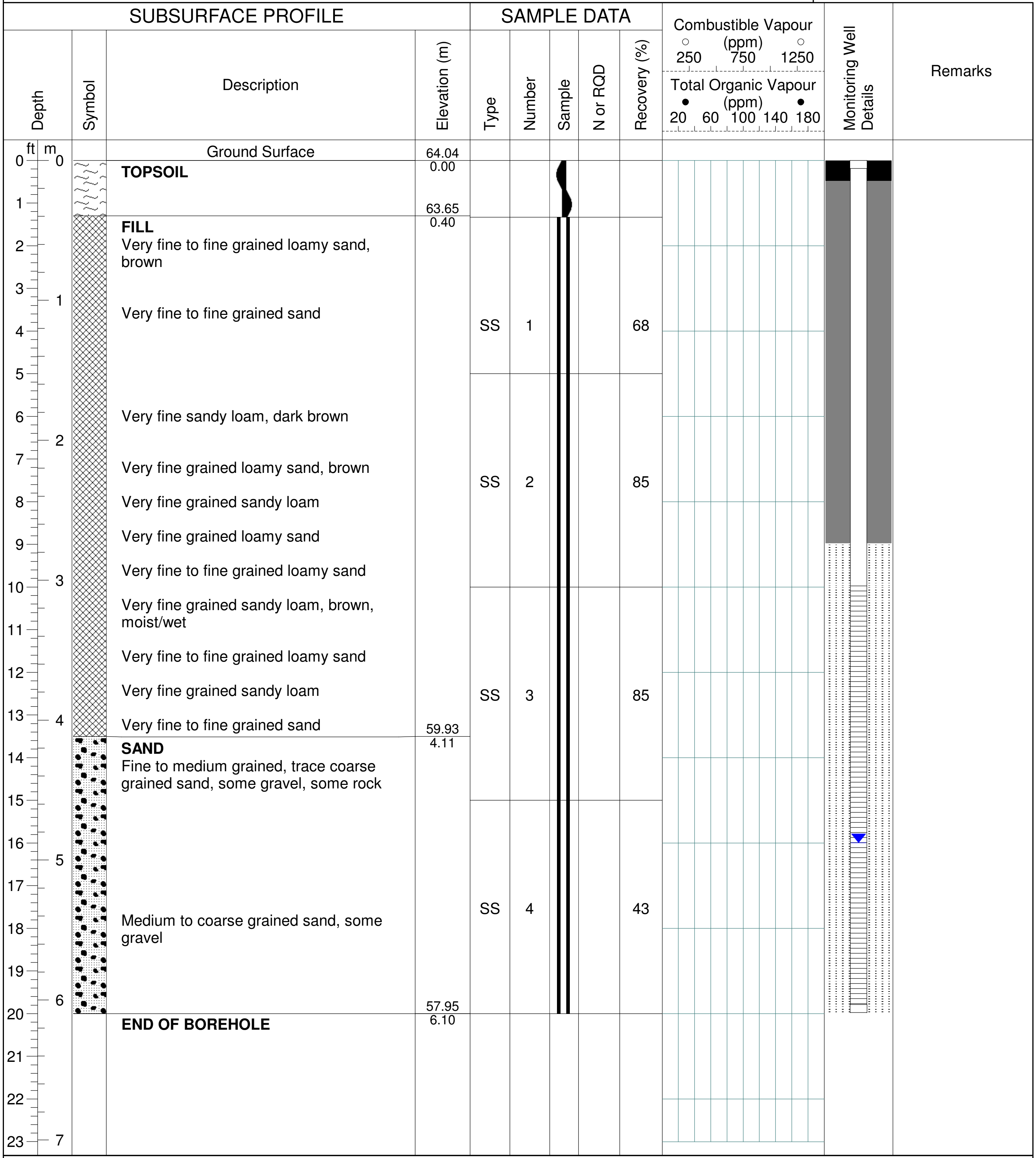
Datum: Geodetic
Checked by: KDH
Sheet: 1 of 1

Stratigraphic and Instrumentation Log: MW15-10

Project No: TZ10100106
Location: 945 Bank Street, Ottawa
Logged By: JFT
Drill Date: October 22, 2015
Hole Size: 127 mm

Project Name: CPU Ground Water Monitoring Program
Client: City of Ottawa
Entered By: KYLT
Drill Method: Direct Push
Drilled By: Strata Drilling Group

Amec Foster Wheeler
300-210 Colonnade Road
Ottawa, Ontario K2E 7L5



Description	Elevation (m)	Type	Number	Recovery (%)
Ground Surface	64.04			
TOPSOIL	0.00			
FILL Very fine to fine grained loamy sand, brown	63.65 0.40			
Very fine to fine grained sand		SS	1	68
Very fine sandy loam, dark brown				
Very fine grained loamy sand, brown				
Very fine grained sandy loam		SS	2	85
Very fine grained loamy sand				
Very fine to fine grained loamy sand				
Very fine grained sandy loam, brown, moist/wet				
Very fine to fine grained loamy sand				
Very fine grained sandy loam		SS	3	85
Very fine to fine grained sand				
SAND Fine to medium grained, trace coarse grained sand, some gravel, some rock	59.93 4.11			
Medium to coarse grained sand, some gravel		SS	4	43
END OF BOREHOLE	57.95 6.10			

Elevation: 64.043 masl
Easting: 368878.435
Northing: 5029083.949

Casing Elevation: 64.979 masl
Well Casing Size: 50.8 mm
Screen Slot Size: 0.25 mm

Filter Pack Size: 6.7 mm
Well Material: Schedule 40 PVC
Vapour Unit: N/A

Datum: Geodetic
Checked by: KDH
Sheet: 1 of 1

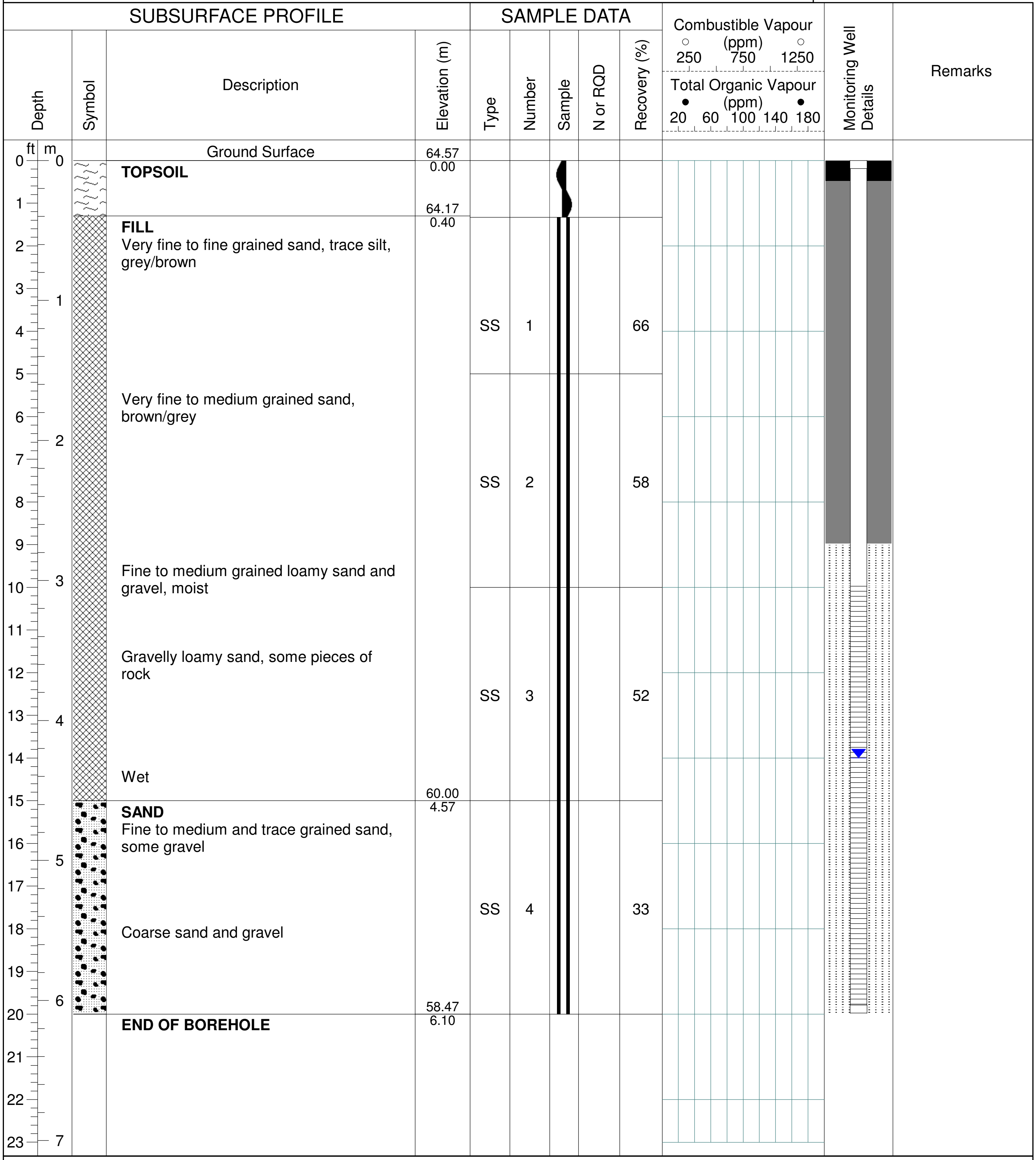
Stratigraphic and Instrumentation Log: MW15-11

amec foster wheeler

Amec Foster Wheeler
300-210 Colonnade Road
Ottawa, Ontario K2E 7L5

Project No: TZ10100106
Location: 945 Bank Street, Ottawa
Logged By: JFT
Drill Date: October 22, 2015
Hole Size: 127 mm

Project Name: CPU Ground Water Monitoring Program
Client: City of Ottawa
Entered By: KYLT
Drill Method: Direct Push
Drilled By: Strata Drilling Group



Depth (ft)	Depth (m)	Description	Elevation (m)	Type	Number	Recovery (%)
		Ground Surface	64.57			
0	0	**TOPSOIL**	0.00			
1			64.17 0.40			
2		**FILL** Very fine to fine grained sand, trace silt, grey/brown				
3	1					
4				SS	1	66
6	2	Very fine to medium grained sand, brown/grey				
8				SS	2	58
10	3	Fine to medium grained loamy sand and gravel, moist				
12		Gravelly loamy sand, some pieces of rock		SS	3	52
13	4					
14		Wet				
15		**SAND** Fine to medium and trace grained sand, some gravel	60.00 4.57			
16	5					
17				SS	4	33
18		Coarse sand and gravel				
20	6	**END OF BOREHOLE**	58.47 6.10			
23	7					

Elevation: 64.571 masl
Easting: 368858.743
Northing: 5028968.821

Casing Elevation: 64.447 masl
Well Casing Size: 50.8 mm
Screen Slot Size: 0.25 mm

Filter Pack Size: 6.7 mm
Well Material: Schedule 40 PVC
Vapour Unit: N/A

Datum: Geodetic
Checked by: KDH
Sheet: 1 of 1

Photographs of Bedrock Core – Lansdowne Park

Photo 1 – BH 3-21 RC11 and RC12

Photo 2 – BH 4-21 RC10

Photographs of Bedrock Core – Lansdowne Park

Photo 3 – BH 4-21 RC11

Photo 4 – BH 5-21 RC10

Photographs of Bedrock Core – Lansdowne Park

Photo 5 – BH 6-21 RC9

Photo 6 – BH 8-21 RC7

Photographs of Bedrock Core – Lansdowne Park

Photo 7 – BH 8-21 RC8

Photo 8 – BH 9-21 RC12

Photographs of Bedrock Core – Lansdowne Park

Photo 9 – BH 9-21 RC12

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5792-1 – TEST HOLE LOCATION PLAN

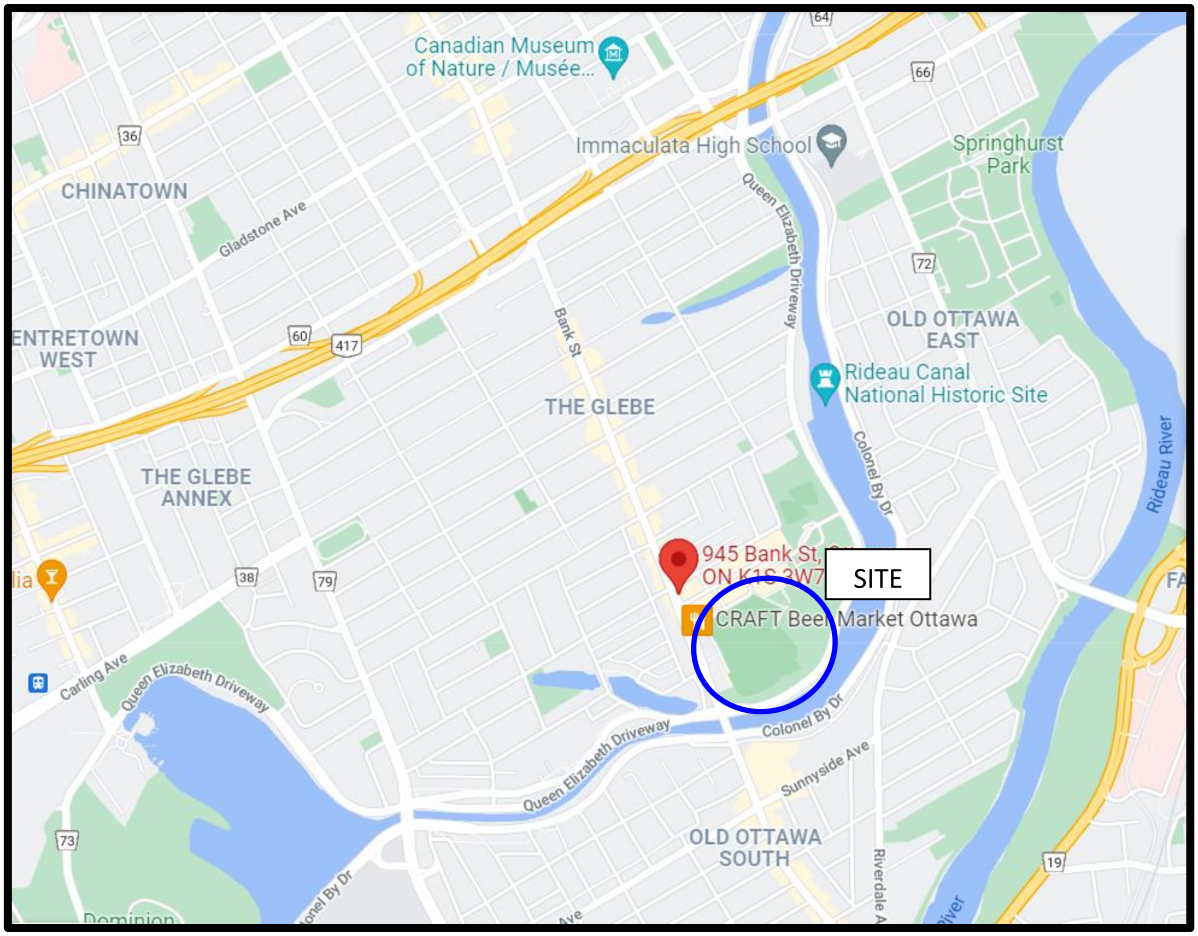


FIGURE 1

KEY PLAN

PATERSON GROUP

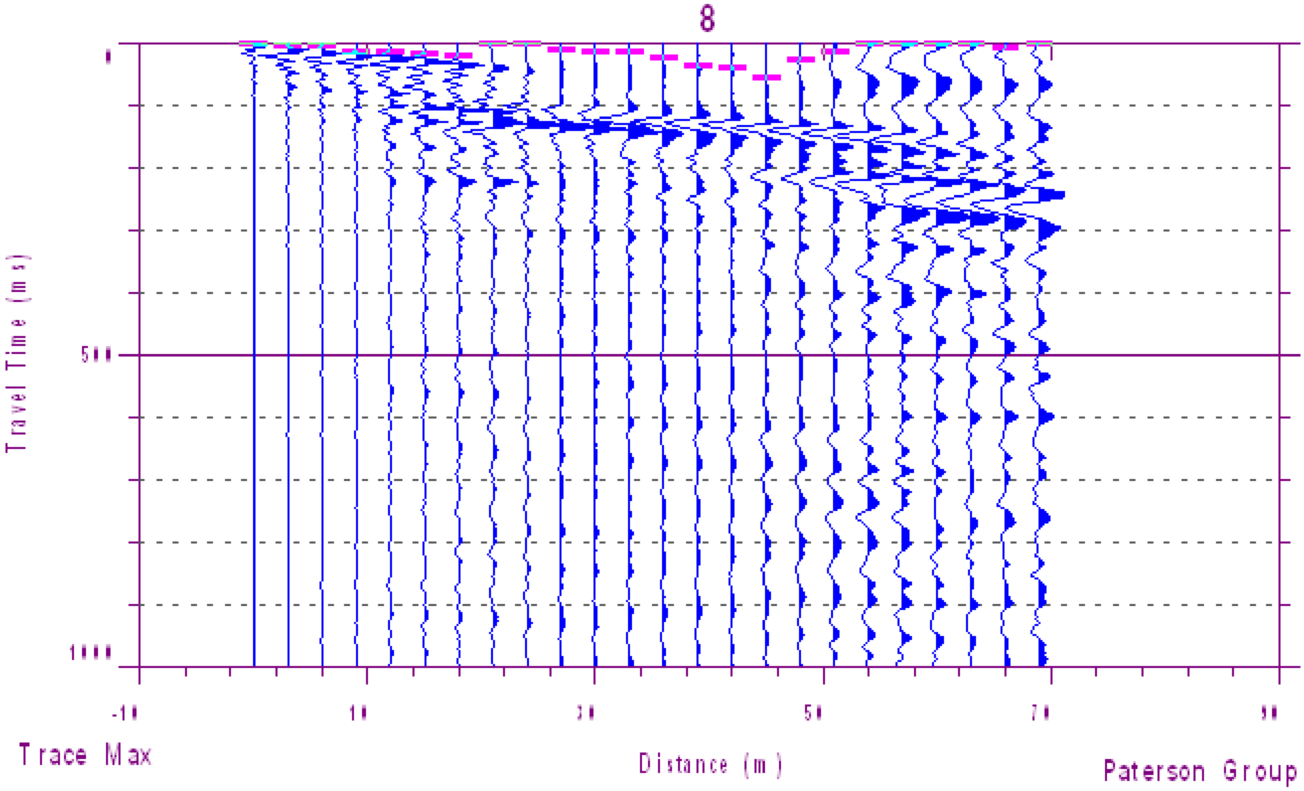


Figure 2 – Shear wave velocity profile at shot location -3.0m

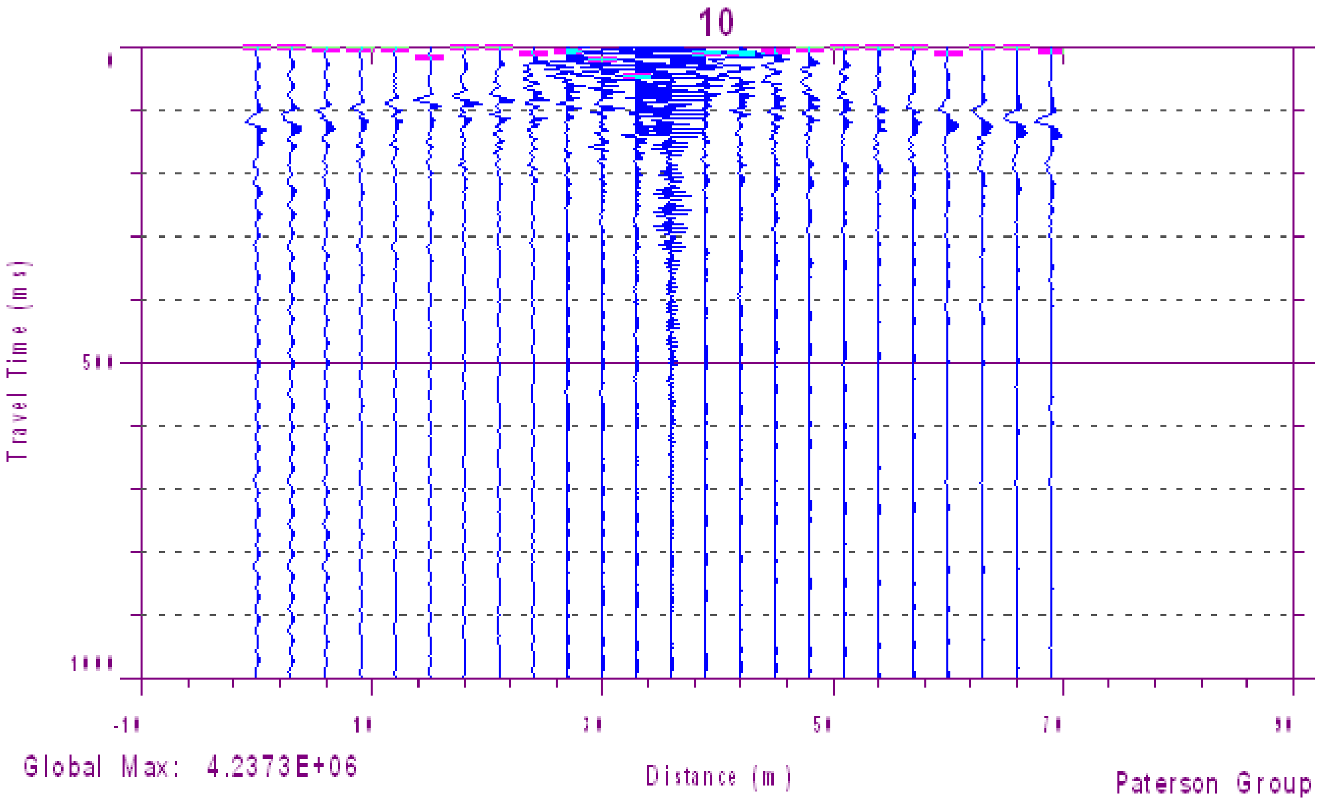
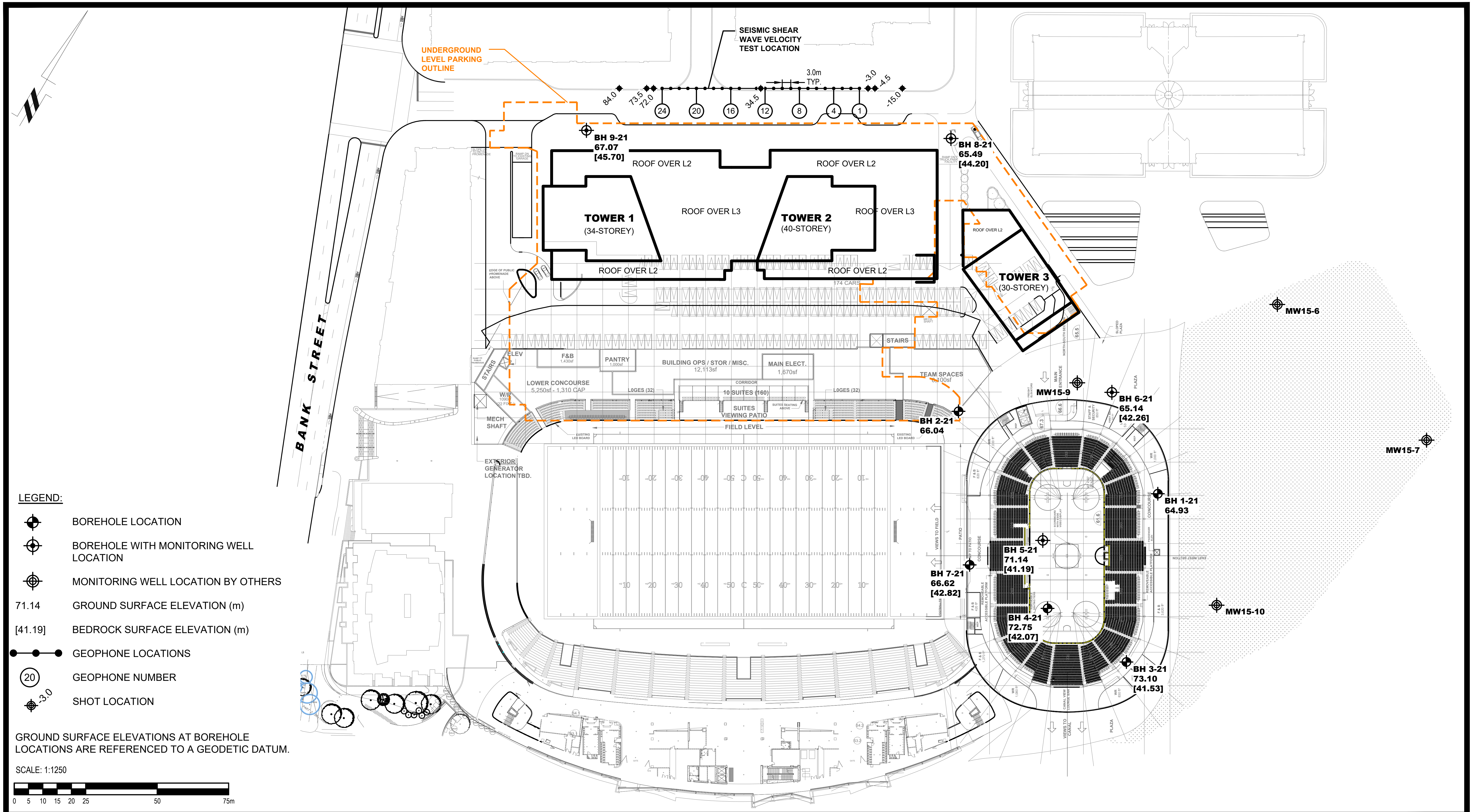


Figure 3 – Shear wave velocity profile at shot location 34.5m



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
⌖	MONITORING WELL LOCATION BY OTHERS
71.14	GROUND SURFACE ELEVATION (m)
[41.19]	BEDROCK SURFACE ELEVATION (m)
●—●—●	GEOPHONE LOCATIONS
(20)	GEOPHONE NUMBER
⌖ -3.0	SHOT LOCATION

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:1250

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2	NEW CONCEPTUAL PLAN UPDATED	17/05/2023	DP
1	NEW BASE PLAN UPDATED	20/03/2023	DP

CITY OF OTTAWA
GEOTECHNICAL INVESTIGATION
LANSDOWNE PARK REDEVELOPMENT
PROP. MULTI-STOREY BUILDINGS AND RINK STRUCTURE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1250
Drawn by: YA
Checked by: MS
Approved by: DJG
Date: 12/2021
Report No.: PG5792-1
Dwg. No.: PG5792-1
Revision No.: 2

p:\autocad drawings\geotechnical\pg57xx\pg5792\pg5792-gps\pg5792-1-test hole location plan (rev.01).dwg