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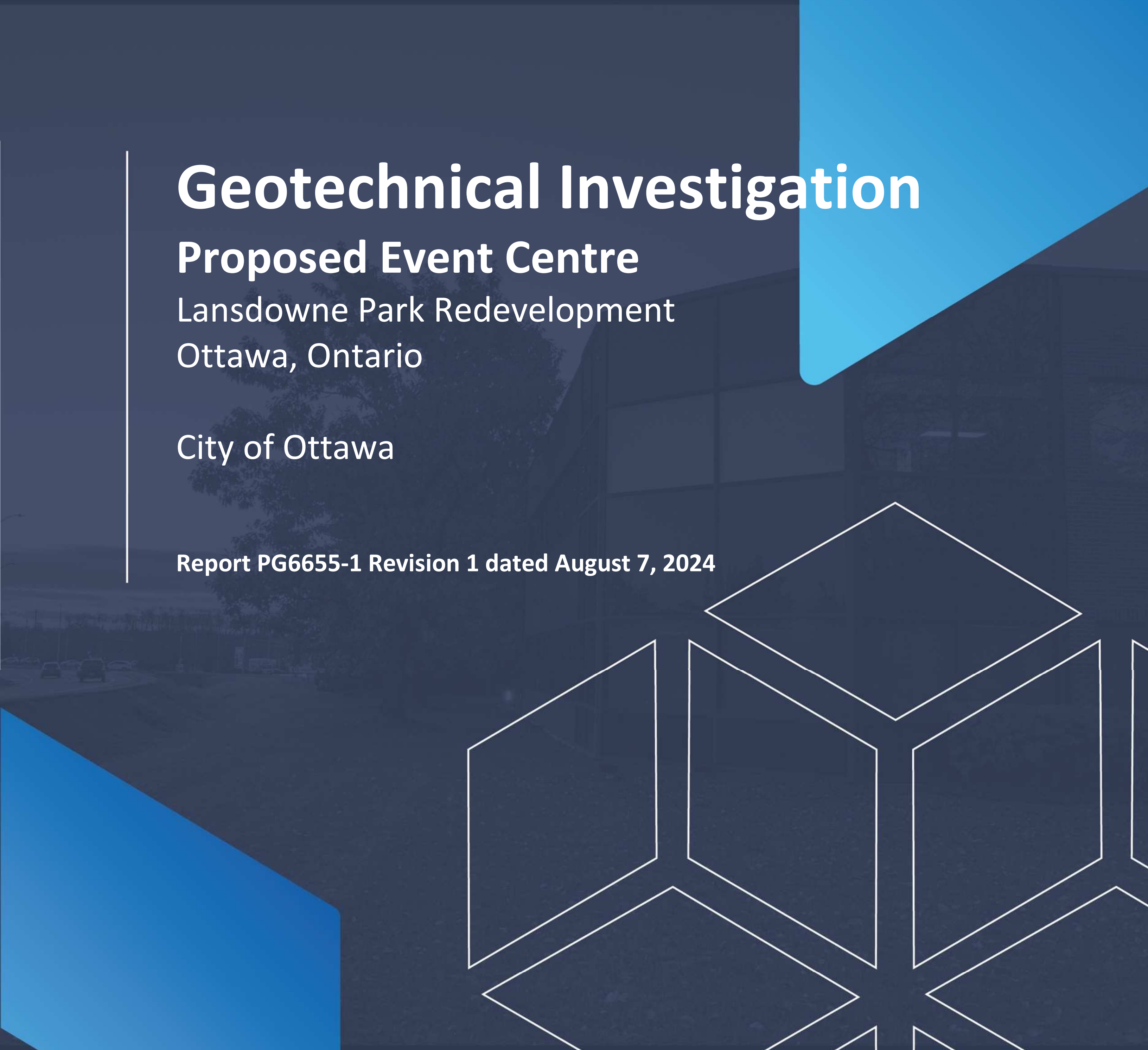

Geotechnical Investigation

Proposed Event Centre

Lansdowne Park Redevelopment
Ottawa, Ontario

City of Ottawa

Report PG6655-1 Revision 1 dated August 7, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by City of Ottawa to conduct a geotechnical investigation for the proposed Event Centre as part of 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

Based on the conceptual site plan, it is understood that the proposed event centre will consist of an underground closed-dome arena facility which will be provided with associated underground storage and team areas and above-ground concourses, suites, hallways and other associated event spaces.

The exterior of the arena facility will be surrounded by patios, terraced landscaped areas and hardscaping (i.e., paver and/or paved pathways), terraced seating and public art features. It is also understood that an open “Great Lawn” landscaped area will be located to the east of the arena and separated from the arena by an approximately 5.5 m high berm along within the landscaped area. It is expected that the proposed building will be municipally serviced.

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 17, 2021 and consisted of advancing a total of eight (8) boreholes to a maximum depth of 33.4 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.

A previous geotechnical investigation was completed by others on October 21 and October 22, 2015. At the time, the investigation consisted of advancing a total of five (5) boreholes to a maximum depth of 6.1 m below ground surface. The borehole locations of the current and previous investigations are shown on Drawing PG6655-1 - 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, BH 6-21, BH 7-21 and BH 8-21 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 and BH 8-21 were fitted with 51 mm diameter PVC groundwater monitoring wells. The remaining 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 for the current investigation 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 PG6655-1 - 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 located southeast of TD Place stadium and south of the Aberdeen Pavilion within Lansdowne Park Development. Currently, the location of the proposed event centre is landscaped, and grass covered. There is an approximately 9 m high grass covered berm throughout the southwestern portion of the subject site and throughout the footprint of the proposed arena footprint. The remainder of the subject site is relatively flat and either grass-covered or landscaped with pavers and associated hardscaped access lanes and walking paths.

The subject site within the Lansdowne Park Development is bound by TD Place stadium to the west, Aberdeen Pavilion to the north, and by Queen Elizabeth Driveway and the Rideau Canal to the south and east.

4.2 Subsurface Profile

Overburden

Generally, the soil profile encountered at the borehole locations consists of topsoil and/or asphaltic concrete and fill underlain by a deposit of silty sand which is further underlain by a glacial till deposit.

The fill material was observed to generally consist of brown silty sand to sandy silt with varying amounts of crushed stone, gravel, cobble, boulders, clay and topsoil. Trace amounts of asphaltic concrete were also observed at the location of BH 3-21, BH 4-21 and BH 5-21. Based on the encountered fill thicknesses, the native, in-situ, undisturbed soils were encountered at approximate geodetic elevations between 62.7 to 64.7m.

The fill was observed to be underlain by a compact, brown silty sand with trace amounts of clay and gravel. The silty sand layer was observed to extend to approximate geodetic elevation of 54.2 to 62.2 m and underlain by the glacial till deposit. The glacial till was observed to consist of very dense to compact, 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

The bedrock was cored in BH 3-21, BH 4-21, BH 5-21, BH 6-21, BH 7-21 and BH 8-21. Bedrock was encountered at approximate elevations of 41.1 to 44.1 m. The cored grey limestone bedrock had average RQD values ranging from 85 to 100%. The recovery values equaled 100% in all boreholes. This is indicative of excellent quality grey limestone bedrock. Photographs of the recovered bedrock cores are included in Appendix 1.

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 October 5, 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 three monitoring wells installed by Paterson (BH 5-21, BH 6-21 and BH 8-21). 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)				**Difference Between Maximum and Minimum Groundwater Elevation (m)**
Monitoring Well ID	**Ground Surface Elevation (m asl)**	**Maximum**	**Date**	**Minimum**	**Date**	
MW 15-06	64.90	60.74	16-08-22	*59.72	09-03-22 20-04-22 10-05-22	*1.02
MW 15-07	64.51	60.42	18-08-22	59.18	20-04-22	1.24
MW 15-09	65.25	60.60	16-08-22	*59.19	09-03-22	*1.41
MW 15-10	64.91	60.57	17-08-22	*59.08	09-03-22	*1.49
MW 15-11	64.57	60.67	09-22-22	59.12	10-11-21	1.54
BH 5-21	65.14	60.58	16-08-22	59.20	08-02-22	1.39
BH 6-21	66.62	60.55	26-09-22	59.13	09-03-22	1.42
BH 8-21	65.45	60.60	26-09-22	59.30	09-03-22	1.31

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 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**, 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 construction of the proposed Event Centre. In view of the anticipated building loads, the proposed structure may be founded on one of the following foundation design options:

- For foundations located above the current groundwater table specified in this report, conventional footings may be placed on an undisturbed compact to dense silty sand.

- For foundations located below the aforementioned groundwater table, a raft foundation is recommended to be considered as foundation support for the proposed building.

Where the founding level extends below the groundwater level, a full watertight design will be required for the foundation walls and floor slabs.

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.

Where fill is encountered at the subgrade depth for the proposed berm, it is recommended to proof-roll (i.e., recompact) the fill layer at the subgrade level with a suitably sized sheepsfoot roller making several passes under dry and above-freezing conditions and under the supervision of Paterson personnel.

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

Building Area

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 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. Compaction of these materials should be complete using a suitably sized sheepsfoot roller making a suitable number of passes and under the supervision of Paterson field personnel. 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 CCW MiraDRAIN 2000 or Delta-Teraxx, connected to a perimeter drainage system.

Landscape Berm

Fill placed for the proposed berm is anticipated to consist of site-generated fill material encountered throughout the existing berm footprint. The fill material is recommended to be placed in maximum 300 mm thick loose lifts and each lift compacted using a suitably sized vibratory sheepsfoot roller. Cobbles, boulders and other stones and debris larger than 200 mm in diameter are recommended to be segregated from the fill material to ensure suitable compaction of the soil fill.

The fill material is recommended to be placed in dry and above-freezing conditions. Frozen fill material that is placed during winter months will thaw and settle more than is expected to be considered throughout the finished surface. Therefore, all efforts should be made to plan to undertake these works during summer and fall seasons. Preparation and placement of the fill material is recommended to be verified and approved by Paterson field personnel at the time of construction.

Raft Slab Subgrade Preparation

It is recommended that a minimum 75 mm thick mud slab layer be placed over the prepared bearing medium once reviewed and approved by Paterson personnel. The purpose of the mud slab layer is to protect the approved founding subsoils from inclement weather, heavy truck traffic, worker traffic and other conditions that would adversely impact the bearing soils during the construction phase.

The mud slab concrete is recommended to consist of a minimum 15 MPa (28 compressive strength) concrete and should not be placed until the bearing medium has been reviewed and approved at the time of construction by Paterson personnel.

5.3 Foundation Design

The following foundation options can be considered for support of the proposed Event Centre structure.

Raft Foundation (Foundation Located Below Groundwater Table)

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 and upon an undisturbed, in-situ compact to dense silty sand and/or glacial till bearing surface. 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 Event Centre structure can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively. An undisturbed soil bearing surface consists of one from which all loose, frozen or disturbed materials, whether in situ or not, have been removed, in the dry, prior to placement of concrete for foundations.

Conventional Spread Footings (Foundations Located Above Groundwater Level)

Using continuously applied loads, footings for the proposed building and other structures placed over an undisturbed, compact to dense silty sand or glacial till bearing surface located above the groundwater table 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.

Proof Rolling and Subgrade Improvement for Loose Sand Below Footings

Where the sand bearing surface for foundations is considered loose by Paterson at the time of construction, it would be recommended to proof roll the bearing surface prior to forming for footings or sub-excavating in-situ material. Proof-rolling (i.e., re-compacting) is recommended to be undertaken in **dry conditions and above freezing temperatures** by an adequately sized vibratory roller making several passes to achieve optimal compaction levels.

The proof-rolling program should also be completed across paved areas to ensure that any poor performing soils are removed prior to pavement structure placement. The compaction program should be reviewed and approved by Paterson at the time of construction.

Depending on the looseness and degree of saturation of loose sandy soils at the time of construction, other measures (additional compaction, sub-excavation and reinstatement of crushed stone fill, mud slab) may be recommended to accommodate site conditions at the time of construction. However, these considerations would be evaluated at the time of design by Paterson on a footing-specific basis.

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.

5.4 Design for Earthquakes

Seismic Shear Wave Velocity Testing

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 Figure 3 and Figure 4 in Appendix 2 of the present report.

Field Program

The shear wave testing was located along Exhibit Way, as presented in Drawing PG6655-1A - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly an 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.

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 1,500 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

Conventional Spread Footings

With the removal of all topsoil and deleterious fill within the footprint of the proposed buildings, the native undisturbed silty sand 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 building (i.e., to build up the subgrade between footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

Raft Slab

Where a raft slab is considered, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements. The recommended pavement structures noted in Subsection 5.7 will be applicable where the basement level underlying foundation support consists of a raft foundation.

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.

A subfloor drainage system, consisting of lines of drainage pipe subdrains connected to a positive outlet (i.e., sump pits), should be provided in the backfill layer directly under the lowest basement floor slab structure. The spacing and associated design details of the underfloor drainage system should be provided by Paterson during the design phase of the project.

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 $Ko \cdot q$ 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 0.32 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

The flexible pavement structure presented in Table 2 and Table 3 should be used for at grade access lanes and heavy loading parking areas.

Table 2 - Recommended Light Duty Asphalt Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either approved fill, in-situ, or OPSS Granular B Type I or II material placed on in-situ soil or fill.	

Table 3 - 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 - Either approved fill, in-situ, or OPSS Granular B Type I or II material placed on in-situ soil or fill.	

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.

Provisions should be carried for remediating site conditions during the time of construction that would impact the construction of the above-noted design pavement structure (i.e., heavy truck traffic rutting and compromising subgrade soils, placement of subbase layers shortly following periods of spring thaw, snowmelt and rainfall events, over service trenches for utilities and poorly compacted backfill, etc.).

Recommendations would be site- and situation specific and only able to be confirmed at the time of construction. However, as a preliminary consideration, provisions should be carried for cow-pathing (over-building haul and access roads to minimize disturbing final subgrade subsoils) localized thickening of subbase layers, supplying and placing separation layers (woven geotextile layer) and reinforcing layers (bi-axial geogrid) advised upon by Paterson personnel at the time of construction.

5.8 Underground Stormwater Infiltration Tank System

It is understood that an underground stormwater infiltration tank system will be included as part of the proposed development. The tank is expected to be founded on a combination of in-situ, undisturbed silty sand/sandy silt and sandy fill. Based on the above, a bearing resistance value for the proposed structure may be considered to be **120 kPa** (SLS) and a factored bearing resistance value at ULS of **180 kPa** may be considered for the system and associated infrastructure/structures.

Based on the results of our geotechnical investigation and associated groundwater monitoring program, the current design groundwater table elevation may be considered at a geodetic elevation of 60.78 m. It is recommended that the bottom of the infiltration tank be founded a minimum of 1 m above the seasonal high post-development groundwater table level.

Consideration could be given to supplying a watertight stormwater management system and as advised by the project civil and environmental consultant to mitigate migration of contaminated groundwater within the proposed subsurface stormwater management system. Consideration could be given to installing a pre-fabricated watertight system, or supplementing a pervious system with an impermeable liner, however, preference should be given to the watertight system where possible.

It is recommended foot-print specific infiltration testing be undertaken by Paterson once the preliminary design details associated with the tank are known. This would consist of completing several test holes throughout the footprint of the proposed infiltration tank to the founding depth and 1 m below the founding depth and completing infiltration testing.

The infiltration testing results and associated design parameters would be provided to the civil designer to complete the associated system design.

A minimum of 2.1 m thick of soil cover, or insulation equivalent, should be present to provide adequate protection to the migration of frost to the storage tank and associated infrastructures bearing mediums.

If pavement structures will overlie the proposed infiltration system, it is recommended that Paterson review the associated tie-ins and details for constructing the pavement structure over the infiltration system from a geotechnical perspective.

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 structure 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 CCW MiraDRAIN 2000, Delta-Teraxx or equivalent other reviewed and approved by Paterson) 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.

Foundation Raft Slab Construction Joints

It is expected that the raft slab, where utilized, will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab.

Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab. The raft slab cold joints should also be overlapped in all directions and cast upon a waterproofing membrane across the length of the cold-joints.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas throughout the remainder of the subject site should be provided with a minimum 450 mm thick layer of OPSS Granular A or OPSS Granular B Type II. The subgrade material should be shaped to promote positive drainage towards the building perimeter drainage system.

This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of the materials SPMDD under dry and above-freezing conditions.

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 to acceptable slopes or be retained by shoring systems from the beginning of the excavation until the structure is backfilled. It is anticipated that insufficient room will be available to permit the building excavation to be constructed by open-cut methods (i.e., unsupported excavations) throughout the western half of the building footprint.

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 Type 2 and Type 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 system can consist of a combination of secant pile, sheet pile and soldier pile and lagging. A cut off wall will be required to control the groundwater influx into the excavation. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included in the earth pressures described below.

The use of a soldier pile and lagging system is not recommended for excavation extending below the groundwater table due to the presence of running sand that can slough into the open excavation during installation. A cut-off shoring system is recommended. Groundwater control will be influenced by the type of shoring system selected by the design engineer. It is important to note that the management of groundwater will be critical in implementing a temporary shoring system due to sandy and localized loose nature of the in-situ subsoils. Efforts will also need to be taken to mitigate the potential for sloughing of retained soils during the installation of lagging given the sandy nature of the subsoil located throughout the subject site.

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 4 – 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 and/or Shoring Support of Adjacent Structures

Based on historical information from Paterson's involvement in previous phases of the Lansdowne Redevelopment project, it had been anticipated the footings for the southern stand structure may have been founded at an approximate geodetic elevation of 63.9 m (assumed as being 2.1 m below the then understood finished floor elevation of 66.0 m).

However, it is recommended to confirm the founding depths and elevations of adjacent structures that will remain in use throughout the construction phase of the proposed development during the design phase of the proposed development through historical review of existing as-built drawings and reports available for all structures adjacent to the proposed building. These conditions should be provided to the pertinent project team members once they are known to ensure design details are developed to consider those structures.

Based on the close proximity between the proposed and existing structures, a supplemental study should be conducted to confirm the requirements of shoring and/or to maintain support for adjacent buildings.

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 5.9×10^{-4} and 7.8×10^{-5} m/sec, therefore, a conservative hydraulic conductivity value of **6.0×10^{-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. As such, up to 1.2 m of saturated material is expected to be encountered at the time of excavation.

Further details regarding dewatering can be found in the Hydrogeological Report (PH4423-1 Revision 2 dated October 6, 2023) completed by Paterson Group, with preliminary construction dewatering volumes presented below in Table 5.

Table 5 – Preliminary Water Taking Rates

Section	Area (m^2)	Steady State Groundwater Dewatering Rate (L/d)	5 year - 1 hour Precipitation Volume (L)
Arena	6,400	7,500,000 – 7,750,000	170,000

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 building. 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.

6.8 Slope Stability Assessment

Slope Conditions

Based on the available plans and drawings, it is understood that a berm and associated slope have been proposed as part of the landscaping at the subject site. The berm and slope are understood to be located east of the Event Centre and west of the Great Lawn also proposed throughout the subject site, and as indicated on Drawing PG6655-1 - Test Hole Location Plan, included in Appendix 2 of the present geotechnical report.

As part of the current investigation, Paterson completed a slope stability analysis of the proposed conditions to evaluate the stability of the slope taking into consideration existing and proposed features, and as described in the following sections. One (1) cross-section was studied as the worst-case scenario (i.e., steepest topographic relief and steepest slope inclination). The location of the cross-section is presented on Drawing PG6655-1 - Test Hole Location Plan, included in Appendix 2 of the present geotechnical report.

Slope Stability Analysis

The slope stability analysis was modeled in SLIDE, a computer program which permits a two-dimensional slope stability analysis calculating several methods including the Bishop's method, which is a widely accepted slope analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to forces favoring failure.

Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsurface soil and groundwater conditions, a factor of safety greater than 1.0 is generally required for the failure risk to be considered acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the slope failure would comprise permanent structures.

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16 g was considered for the cross-section for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

One (1) slope cross-section was analyzed based on proposed conditions under static and seismic loading. Subsoil conditions at the cross-section were inferred based on the findings of the geotechnical investigation and borehole information. The cross-section location is presented on Drawing PG6655-1 - Test Hole Location Plan in Appendix 2.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation. The effective strength soil parameters used for static analysis are presented in Table 6.

Table 6 – Effective Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Topsoil	16	33	5
Fill	18	31	0
Silty Sand	19	33	0

The total strength parameters for seismic analysis were chosen based on the subsurface conditions observed in the test holes, and our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 7.

Table 7 – Total Stress Soil and Material Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
Topsoil	16	33	-
Fill	18	31	-
Silty Sand	19	33	

Static Loading Analysis

The results for the static analysis under proposed conditions are presented in Figure 2A included in Appendix 2. The results indicate that the slope stability factor of safety was found to be greater than 1.5 for slope section A. Therefore, the proposed slope is considered stable under static loading.

Seismic Loading Analysis

The results of the analyses considering seismic loading are presented in Figure 2B in Appendix 2. The slope stability factor of safety was found to be greater than 1.1 for slope section A. Based on these results, the proposed slope is considered stable under seismic loading. Therefore, a stable slope allowance setback is not required from a geotechnical perspective.

Conclusion

Based on our review, the proposed berm re-location and currently proposed grading is considered acceptable and stable from a geotechnical perspective. The earthworks program to construct the berm will be key in ensuring the berm is constructed in a satisfactorily manner.

The subgrade, consisting of the existing materials should be proof rolled, where considered loose by Paterson at the time of construction. Proof-rolling is recommended to be undertaken in **dry conditions and above freezing temperatures** by an adequately sized vibratory roller making several passes to achieve optimal compaction levels. Any poor performing soils should be removed and replaced with suitable compacted material prior to construction of the slope.

The compaction program should be reviewed and approved by Paterson at the time of construction. Depending on the looseness and degree of saturation of existing subgrade material at the time of construction, other measures (additional compaction, sub-excavation and reinstatement with crushed stone fill, mud slab) may be recommended to accommodate site conditions at the time of construction.

It is recommended that a 100 to 150 mm thick layer of topsoil mixed with a hardy grass seed be placed across the slope face to contribute to the stability of the slope and reduce possible erosion from rainfall and snowmelt events.

It is recommended that Paterson be circulated changes in the planned grading and associated design of the proposed berm relocation.

6.9 Landscaping Considerations

Retaining Walls

It is understood that retaining walls are expected to be constructed throughout the subject site as part of the proposed development. It should be noted that all retaining walls should be designed by a Licensed Professional Engineer in the Province of Ontario and should be subject to a conforming global stability analysis.

All sections of the retaining walls should be designed so that their internal and external failure modes comply with CHBD requirements. Furthermore, any proposed retaining wall should be designed to maintain an adequate factor of safety greater than 1.5 under static loading conditions and greater than 1.1 under seismic loading conditions.

The applicable seismic design should incorporate Peak Ground Acceleration (PGA) for the Ottawa area as per the OBC 2012.

It is also required that the bearing medium of the proposed wall is reviewed by Paterson field personnel at the time of excavation and prior to placement of the granular bedding layer. Based on the results of the geotechnical investigation, it is anticipated that the walls will be founded over an engineered fill pad or undisturbed, in-situ soil bearing surfaces.

The soil parameters presented in Table 8 can be used in the design of the retaining walls.

Table 8 – Soil parameters for global stability analysis

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (°)	Effective Cohesion (kPa)	Total Cohesion (kPa)
Fill	18	31	0	0
Silty Sand	19	33	0	0

It is recommended that a 100 mm diameter perforated corrugated plastic pipe with geosock, surrounded by 150 mm of 19 mm clear crushed stone on all sides, be placed behind the heel of the wall. The pipe should have a positive outlet, either in front of, below, or to the side of the wall, towards a natural slope or drainage system.

Backfill Materials

Retaining walls should be backfilled with free-draining granular material, as Granular A or Granular B Type II materials. Longitudinal drains and outlets should also be incorporated to ensure proper drainage of the backfill material.

It is further recommended that backfill material be placed within a wedge-shaped area defined by a line drawn from below the rear edge of the wall's base block at a slope of 1H:1V, or a minimum of 1 m behind the rear of the blocks. All material must be compacted to a minimum of 98% of the materials SPMDD.

Geotechnical parameters of the proposed free-draining backfill material to be used at the subject site are provided in Table 9 for design purposes.

Table 9 – Geotechnical parameters for backfill material

Material Description	Unit Weight (kN/m^3)		Friction Angle (°) φ'	Friction Factor, tan δ	Lateral Earth Pressure Coefficients		
	Drained γ_{dry}	Effective γ'			Active K_a	At Rest K_o	Passive K_P
Granular A (Crushed Stone)	22	13.5	38	0.6	0.24	0.38	4.20
Granular B Type II (Crushed Stone)	22	13.5	40	0.6	0.22	0.36	4.60

Notes:

I. The properties of backfill materials are for a condition of 98% of the materials SPMDD.
II. Earth pressure coefficients provided are for the horizontal backfill profile.
III. For soil above the water table, the “drained” unit weight must be used and below the water table, the “effective” unit weight must be used.

Lateral Earth Pressure

It is recommended that a minimum of 1 m of backfill material consisting of clean, imported crushed stone as Granular A or Granular B Type II. The geotechnical soil parameters shown in Table 9 should be used for retaining wall design.

Tree Planting Considerations

Based on the results of the geotechnical investigation, it is expected that the proposed structures will be founded on non-cohesive soils. Therefore, the proposed development will not be subject to planting restrictions as based on the *City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines)* from a geotechnical perspective.

Any trees planted behind (on top) of retaining walls should be provided with a minimum setback of 2 m from the wall footprint. Furthermore, it is recommended that trees are planted with root control measures in pace, such as root barriers or bags. Additional geotechnical details and design information may be provided by Paterson during the design phase of the subject retaining walls.

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 excavation program, shoring design, and assumptions of the founding conditions for existing adjacent structures prior to construction.
- Review of the waterproofing details for the elevator shaft and building sump pits, as well as for the buildings foundation as recommended herein.
- Inspection of the installation of the waterproofing and perimeter and underground floor drainage system during construction.
- 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.
- Review of the earthworks program associated with the proposed berm.
- 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 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
Aug. 7, 2024
D. J. PETAHTEGOOSE
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PROVINCE OF ONTARIO

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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

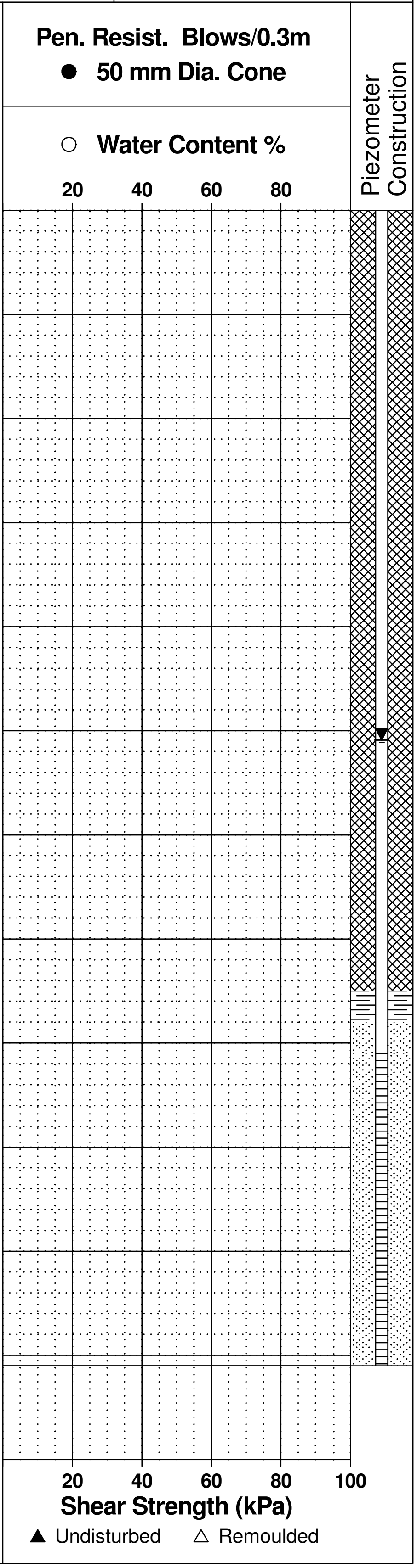
SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Crushed stone, trace sand	0.41
FILL: Brown silty sand with topsoil	0.53
FILL: Brown silty sand to sandy silt, some clay, trace topsoil	2.19
Compact, brown **SILTY SAND** - trace clay from 3.0 to 4.3m depth - trace gravel by 4.3m depth	5.49
GLACIAL TILL: Very dense to compact, brown silty sand with gravel, cobbles and boulders	11.10

End of Borehole

(GWL @ 5.09m - Nov. 12, 2021)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	64
SS	3	58	28
SS	4	42	13
SS	5	25	14
SS	6	50	15
SS	7	33	20
SS	8	50	53
SS	9	42	32
SS	10	33	31
SS	11	25	26
SS	12	42	21
SS	13	42	29
SS	14	33	39
SS	15		65

DEPTH (m)	ELEV. (m)
0	64.93
1	63.93
2	62.93
3	61.93
4	60.93
5	59.93
6	58.93
7	57.93
8	56.93
9	55.93
10	54.93
11	53.93



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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 October 25, 2021

FILE NO. PG5792

HOLE NO. BH 2-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE 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
		SS	3	50	7		
2.21						2	64.04
Compact, brown **SILTY SAND**		SS	4	50	14		
						3	63.04
		SS	5	33	10		
		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		
						9	57.04
		SS	13	8	61		
- 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

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 27, 2021

FILE NO. PG5792

HOLE NO. BH 3-21

SOIL DESCRIPTION	Depth of lower boundary (m)
GROUND SURFACE	
TOPSOIL	0.36
FILL: Brown silty sand, some gravel, occasional cobble and boulders, trace clay and topsoil - cored through boulder from 3.28 to 3.81m depth - trace ash from 5.3 to 5.9m depth - trace asphaltic concrete from 7.0 to 7.6m depth	9.45
Compact, brown **SILTY SAND to SANDY SILT**	11.40
Compact, brown **SILTY SAND,** some gravel	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	33	16
SS	3	22	50+
SS	4	17	11
SS	5	44	50+
RC	1	95	
SS	6	33	6
SS	7	33	47
SS	8	25	50+
SS	9	25	59
SS	10	25	38
SS	11	0	50+
SS	12	33	34
SS	13	50	14
SS	14	58	22
SS	15	50	28
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

Piezometer Construction

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

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 27, 2021

FILE NO. PG5792

HOLE NO. BH 3-21

SOIL DESCRIPTION

GROUND SURFACE

Compact, brown **SILTY SAND,** some gravel

15.54

GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders

- grey by 20.2m depth

- compact by 21.3m depth

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	17	33	19
SS	18	25	18
SS	19	4	12
SS	20	4	21
SS	21	50	36
SS	22	67	60
SS	23	33	50+
RC	2	70	
SS	24	4	50+
RC	3	64	
RC	4	52	
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

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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Lansdowne Park Redevelopment
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DATUM Geodetic

FILE NO. PG5792

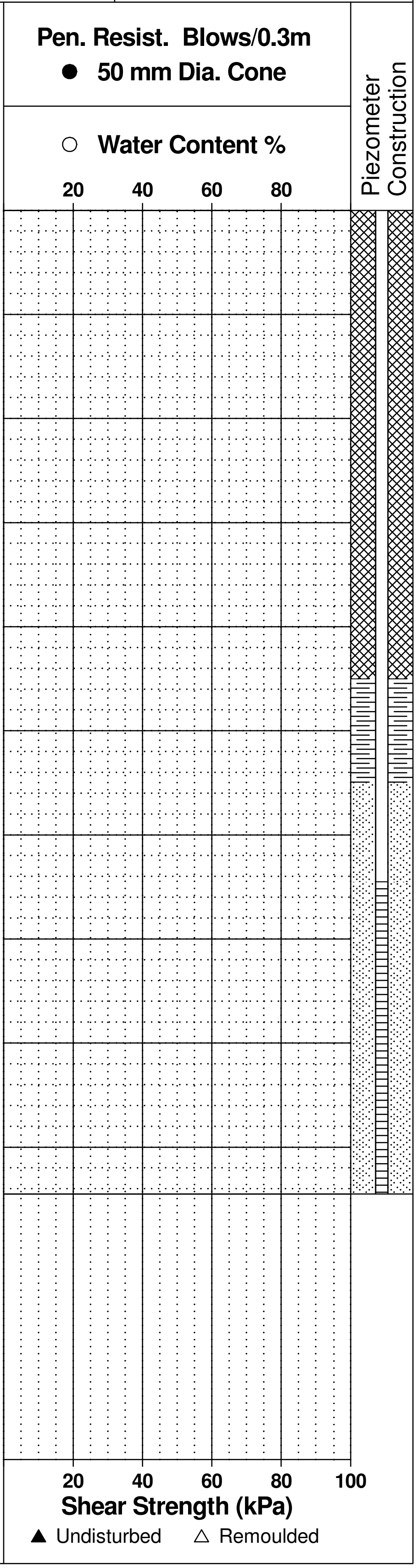
REMARKS

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	DEPTH (m)	ELEV. (m)
GROUND SURFACE					24	49.10
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders - cobbles and boulders content decreasing with depth	RC	7	8		25	48.10
	RC	8	0		26	47.10
	RC	9	0		27	46.10
	RC	10	0		28	45.10
	RC	11	100	71	29	44.10
31.57					30	43.10
BEDROCK: Good to excellent quality, grey limestone with occasional shale partings	RC	12	100	98	31	42.10
33.45					32	41.10
End of Borehole (GWL @ 13.46m - Nov. 16, 2021)					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

FILE NO. PG5792

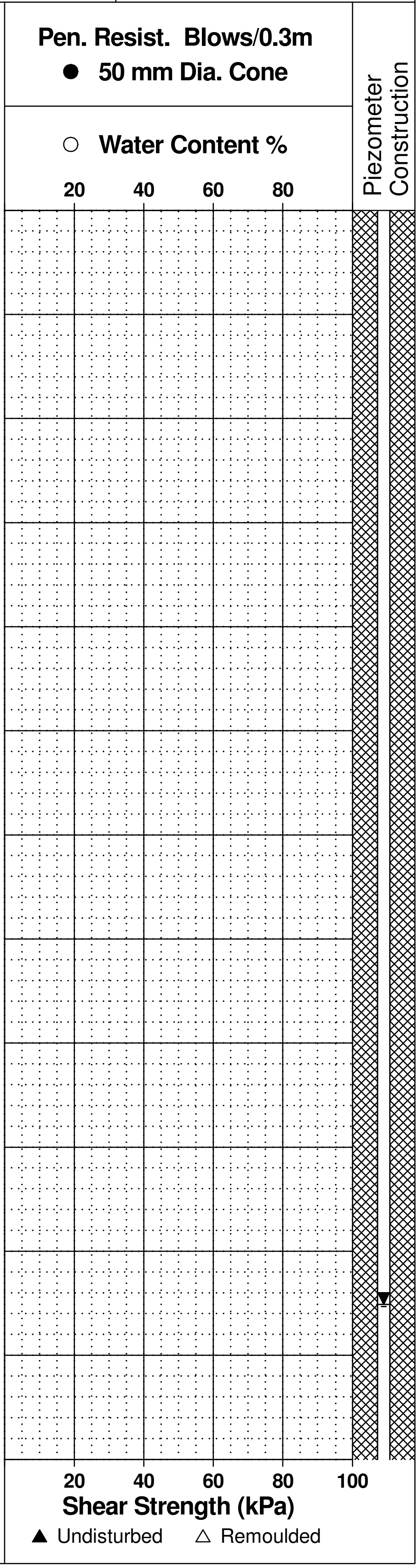
REMARKS

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					0	72.75
TOPSOIL 0.30						
	AU	1				
					1	71.75
	SS	2	33	5		
	SS	3	58	49		
					2	70.75
	SS	4	50	10		
					3	69.75
FILL: Brown silty sand iwth gravel and cobbles, occasional boulders, trace clay	SS	5	50	8		
					4	68.75
	SS	6	50	8		
	SS	7	42	46		
					5	67.75
- some topsoil from 5.3 to 5.9m depth	SS	8	33	28		
					6	66.75
	SS	9	50	19		
					7	65.75
- some asphaltic concrete from 7.6 to 8.2m depth	SS	10	18	9		
	SS	11		50+		
					8	64.75
8.53						
	SS	12	58	13		
					9	63.75
Compact, brown **SILTY SAND to SANDY SILT**	SS	13		14		
					10	62.75
	SS	14	42	19		
	SS	15	50	18		
					11	61.75
11.25						
GLACIAL TILL: Very dense to dense, silty sand with gravel, cobbles and boulders	SS	16	33	59		
					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

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 5, 2021

FILE NO. PG5792

HOLE NO. BH 4-21

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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Geotechnical Investigation
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DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

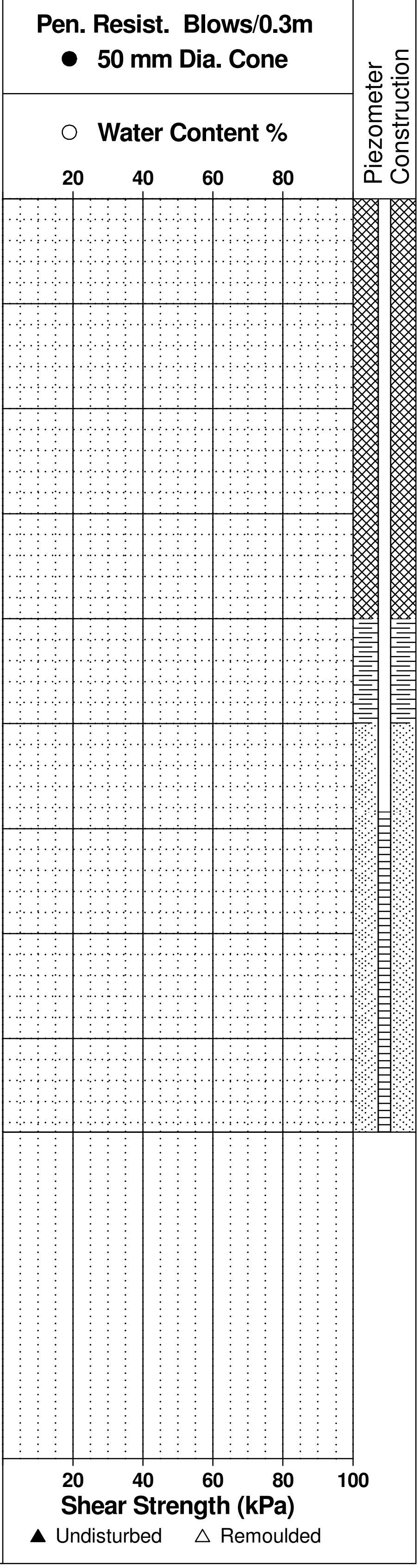
DATE November 5, 2021

FILE NO. PG5792

HOLE NO. BH 4-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
GLACIAL TILL: Very dense to dense, silty sand with gravel, cobbles and boulders	RC	8	5	
	SS	22	0	50+
	RC	9	10	
30.68				
BEDROCK: Excellent quality, grey limestone with occasional shale partings	RC	10	100	100
	RC	11	100	100
32.89				
End of Borehole (GWL @ 10.51m - Nov. 16, 2021)				

DEPTH (m)	ELEV. (m)
24	48.75
25	47.75
26	46.75
27	45.75
28	44.75
29	43.75
30	42.75
31	41.75
32	40.75



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SOIL PROFILE AND TEST DATA

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

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REMARKS

BORINGS BY CME 55 Power Auger

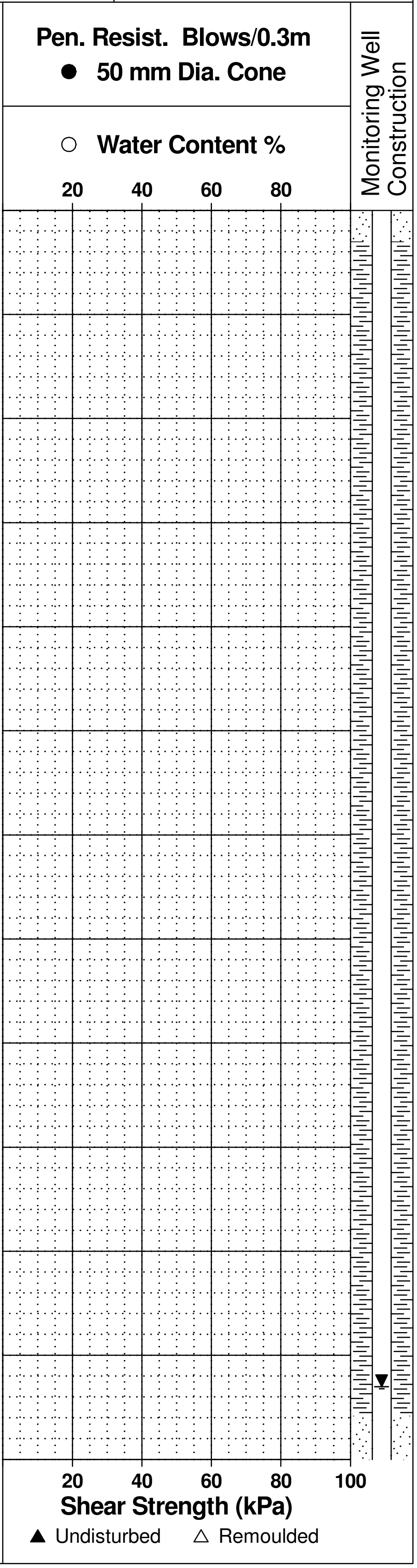
DATE November 9, 2021

FILE NO. PG5792

HOLE NO. BH 5-21

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.36	AU	1		
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	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
6.70	SS	9	67	15
Compact to dense, brown **SILTY SAND** - some gravel by 8.5m depth	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



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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

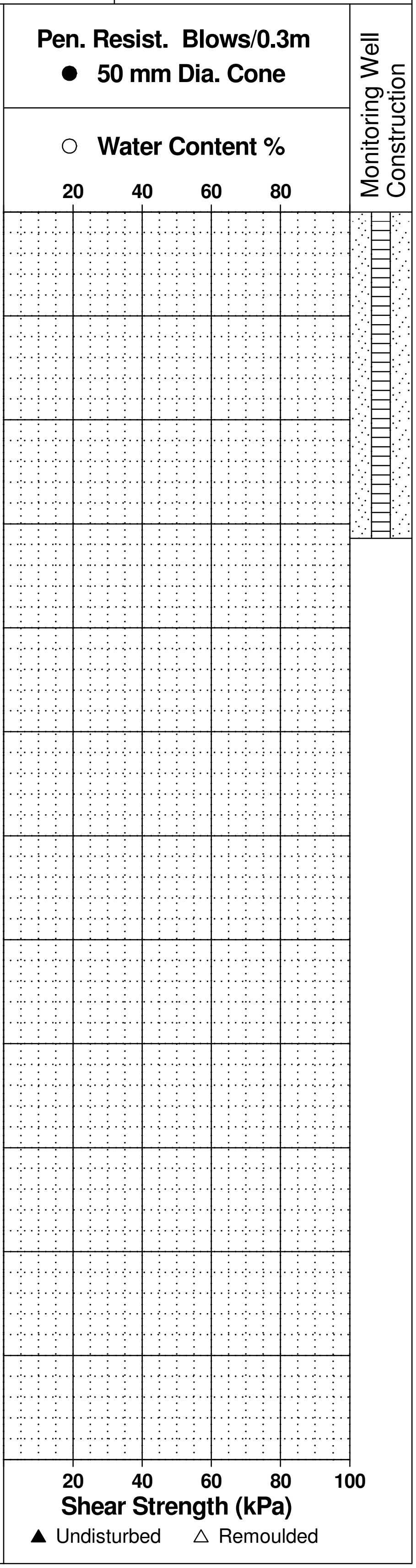
BORINGS BY CME 55 Power Auger

DATE November 9, 2021

FILE NO. PG5792

HOLE NO. BH 5-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					12	59.14
Compact to dense, brown **SILTY SAND,** some gravel	SS	17	21	9		
					13	58.14
	SS	18	50	23		
	SS	19	50	28		
					14	57.14
14.20						
GLACIAL TILL: Very dense to dense, brown silty sand with gravel, cobbles and boulders	SS	20	55	50+		
					15	56.14
	RC	1	60			
					16	55.14
	SS	21	42	71		
					17	54.14
	RC	2	22			
					18	53.14
	SS	22	64	38		
					19	52.14
- grey by 18.2m depth						
	RC	3	15			
					20	51.14
					21	50.14
	SS	23	100	50+		
	RC	4	15			
					22	49.14
	SS	24	0	50+		
					23	48.14
	RC	5	19			
					24	47.14



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SOIL PROFILE AND TEST DATA

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Prop. Multi-Storey Buildings & Rink Structure, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE November 9, 2021

FILE NO. PG5792

HOLE NO. BH 5-21

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

○ Water Content %

Monitoring Well Construction

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			
					27	44.14
	SS	27	86	50+		
	RC	8	37			
					28	43.14
	SS	28	0	10		
					29	42.14
	RC	9	100	100		
29.95						
BEDROCK: Excellent quality, grey limestone with occasional shale partings					30	41.14
	RC	10	100	93		
					31	40.14
31.55						
End of Borehole (GWL @ 11.30m - Nov. 16, 2021)						

Water Content %: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ 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

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 11, 2021

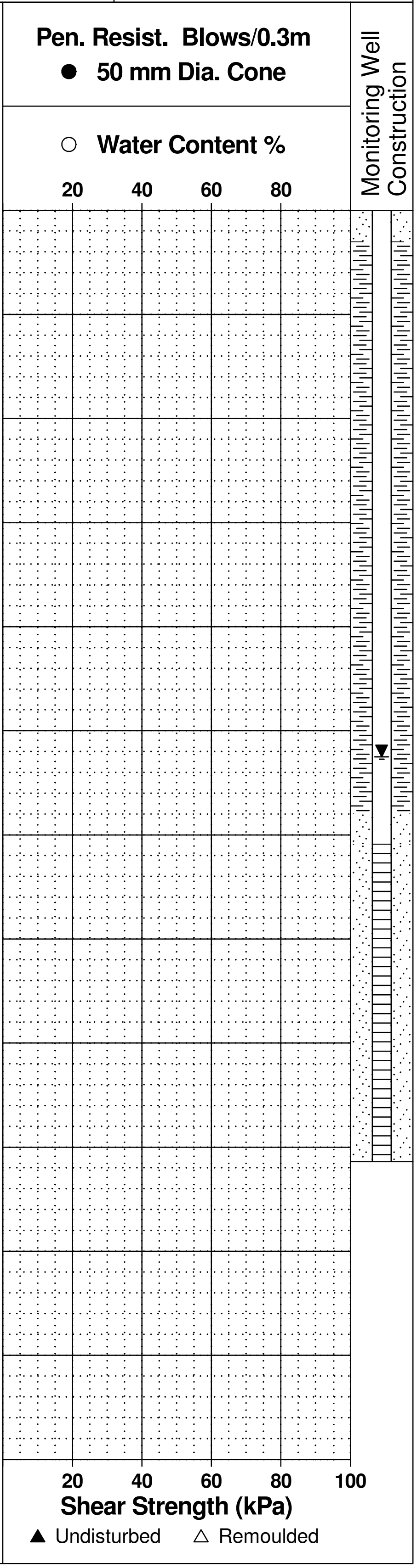
FILE NO. PG5792

HOLE NO. BH 6-21

SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone and gravel	0.91
Compact to dense, brown **SILTY SAND,** trace to some gravel	5.41
GLACIAL TILL: Dense brown silty sand with gravel, cobbles and boulders - silty sand to sandy silt layer from 8.9 to 9.3m depth - grey by 12.2m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	67	47
SS	2	42	26
SS	3	50	17
SS	4	58	13
SS	5	50	43
SS	6	50	13
SS	7	50	50+
SS	8	50	50+
SS	9	42	34
SS	10	42	35
SS	11	50	34
SS	12	43	78
SS	13	50	43
SS	14	42	38
SS	15	43	50+
RC	1	61	
SS	16	40	50+
RC	2	75	

DEPTH (m)	ELEV. (m)
0	65.14
1	64.14
2	63.14
3	62.14
4	61.14
5	60.14
6	59.14
7	58.14
8	57.14
9	56.14
10	55.14
11	54.14
12	53.14



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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 6-21

BORINGS BY CME-55 Low Clearance Drill

DATE November 11, 2021

SOIL DESCRIPTION

GROUND SURFACE

GLACIAL TILL: Dense, grey silty sand with gravel, cobbles and boulders

- some clay by 16.8m depth

22.88

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

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
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	
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

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Monitoring Well Construction

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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 6-21

BORINGS BY CME-55 Low Clearance Drill

DATE November 11, 2021

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
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	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	66.62
TOPSOIL 0.25						
FILL: Brown silty sand, some gravel 1.93					1	65.62
					2	64.62
Compact to dense, brown **SILTY SAND,** trace gravel					3	63.62
4.42	SS	1	50	27	4	62.62
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	SS	2	0	48	5	61.62
	SS	3	50	50+		
	SS	4	50	50+	6	60.62
	RC	1	45		7	59.62
	SS	5	53	50+	8	58.62
	SS	6	0	50+		
	RC	2	56		9	57.62
	RC	3	33		10	56.62
- some shale fragments from 11.0 to 11.5m depth	SS	7	42	53	11	55.62
					12	54.62

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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DATUM Geodetic
REMARKS
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DATE November 15, 2021
FILE NO. PG5792
HOLE NO. BH 7-21

SOIL DESCRIPTION

GROUND SURFACE

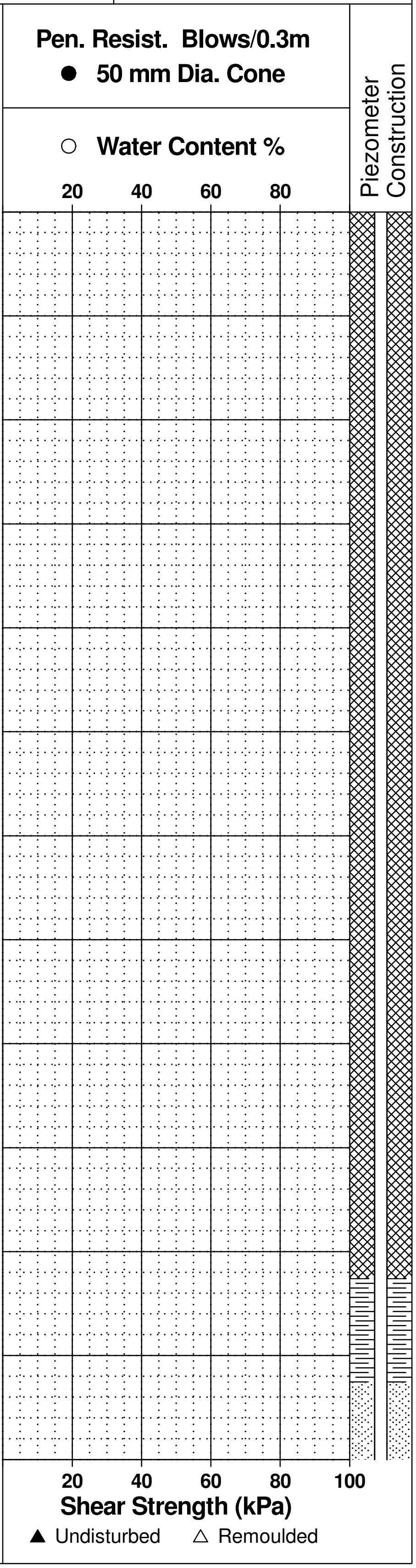
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders

- grey by 13.7m depth

23.80

SAMPLE 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

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

DATUM Geodetic

FILE NO. **PG5792**

REMARKS

HOLE NO. **BH 7-21**

BORINGS BY CME-55 Low Clearance Drill

DATE November 15, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						24	42.62
		RC	11	100	100		
						25	41.62
BEDROCK: Excellent quality, grey limestone with occasional shale partings							
						26	40.62
		RC	12	100	94		
						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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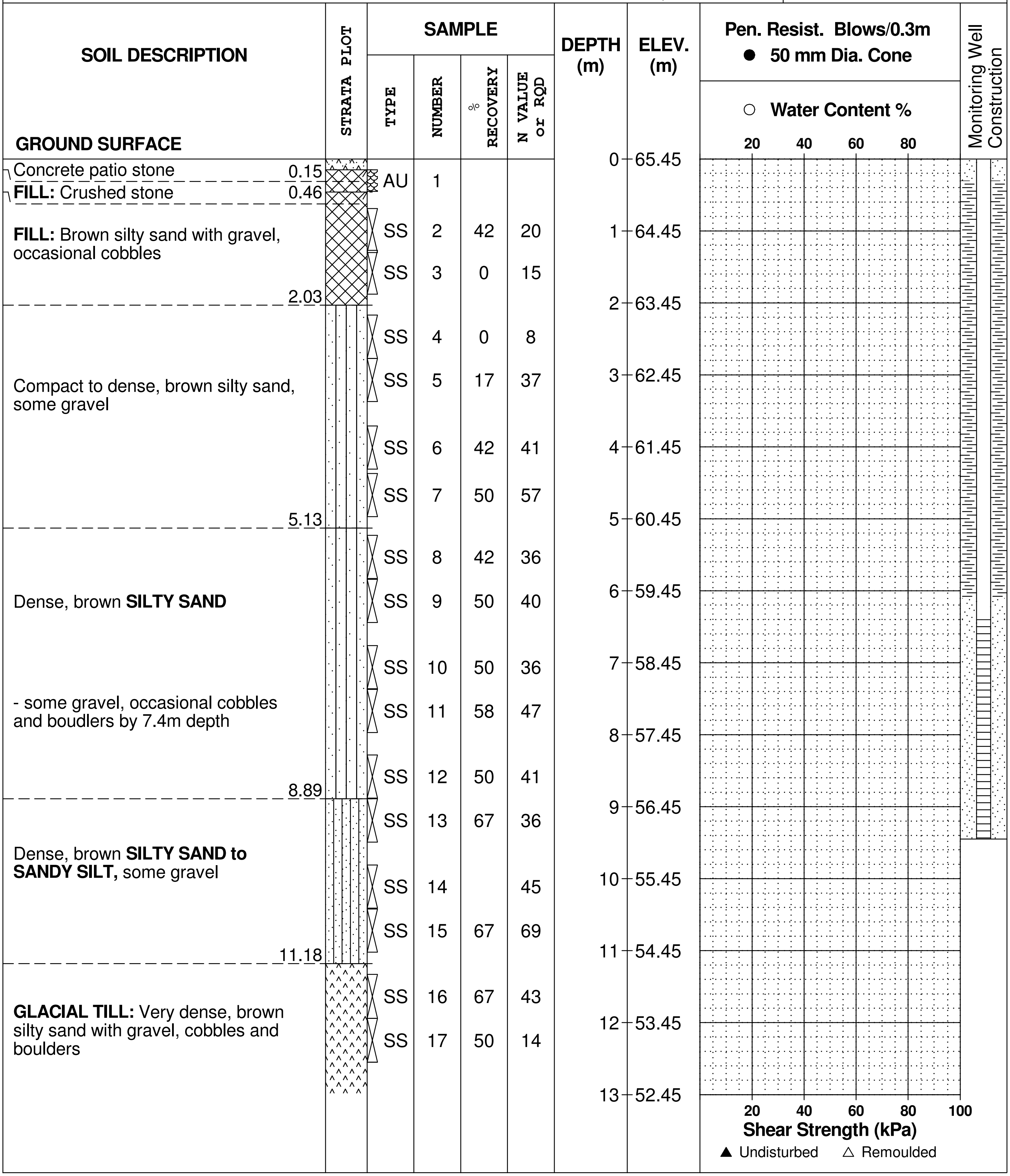
SOIL PROFILE AND TEST DATA

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

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



SOIL DESCRIPTION	
GROUND SURFACE	
Concrete patio stone	0.15
FILL: Crushed stone	0.46
FILL: Brown silty sand with gravel, occasional cobbles	2.03
Compact to dense, brown silty sand, some gravel	5.13
Dense, brown **SILTY SAND** - some gravel, occasional cobbles and boudlers by 7.4m depth	8.89
Dense, brown **SILTY SAND to SANDY SILT,** some gravel	11.18
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	20
SS	3	0	15
SS	4	0	8
SS	5	17	37
SS	6	42	41
SS	7	50	57
SS	8	42	36
SS	9	50	40
SS	10	50	36
SS	11	58	47
SS	12	50	41
SS	13	67	36
SS	14		45
SS	15	67	69
SS	16	67	43
SS	17	50	14

DEPTH (m)	ELEV. (m)
0	65.45
1	64.45
2	63.45
3	62.45
4	61.45
5	60.45
6	59.45
7	58.45
8	57.45
9	56.45
10	55.45
11	54.45
12	53.45
13	52.45

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 8-21

BORINGS BY CME-55 Low Clearance Drill

DATE November 17, 2021

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
	RC	1	55		13	52.45
					14	51.45
	RC	2	30			
					15	50.45
	SS	18	58	28		
					16	49.45
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	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+		
	RC	7	100	90		
					22	43.45
BEDROCK: Excellent quality, grey limestone with occasional shale partings						
					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

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 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)

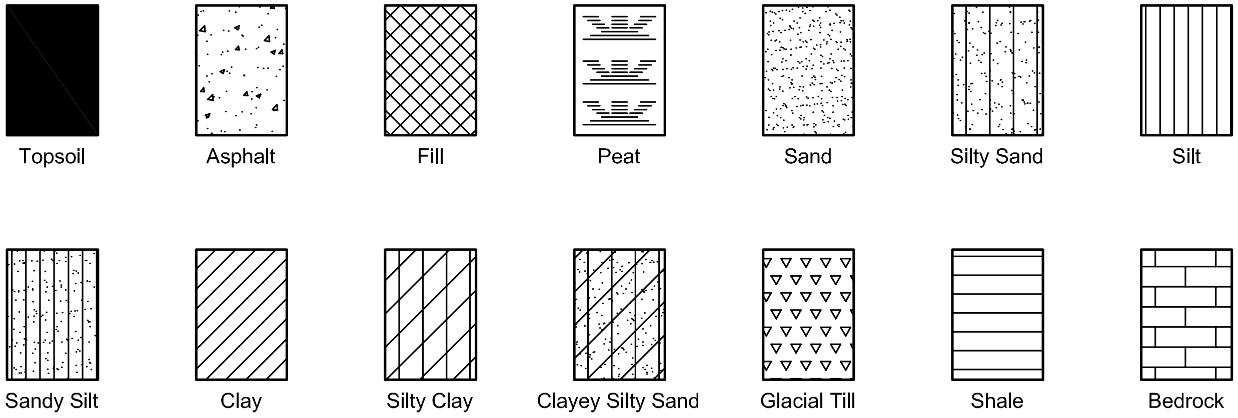
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

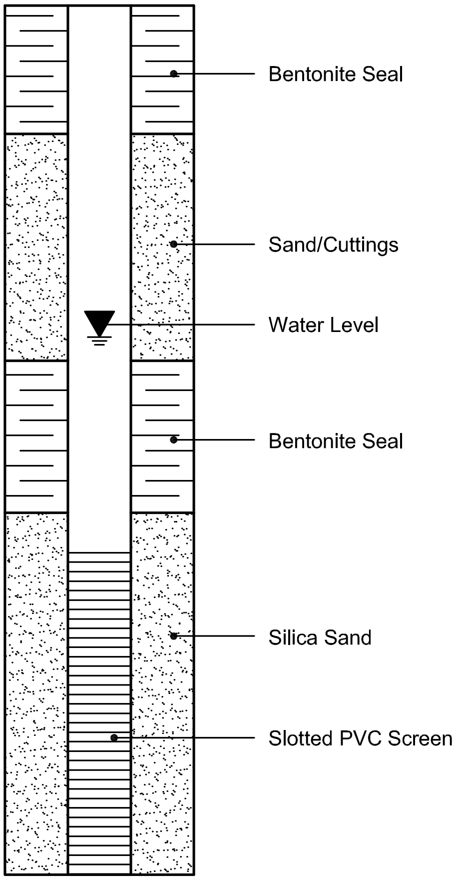
SYMBOLS AND TERMS (continued)

STRATA PLOT

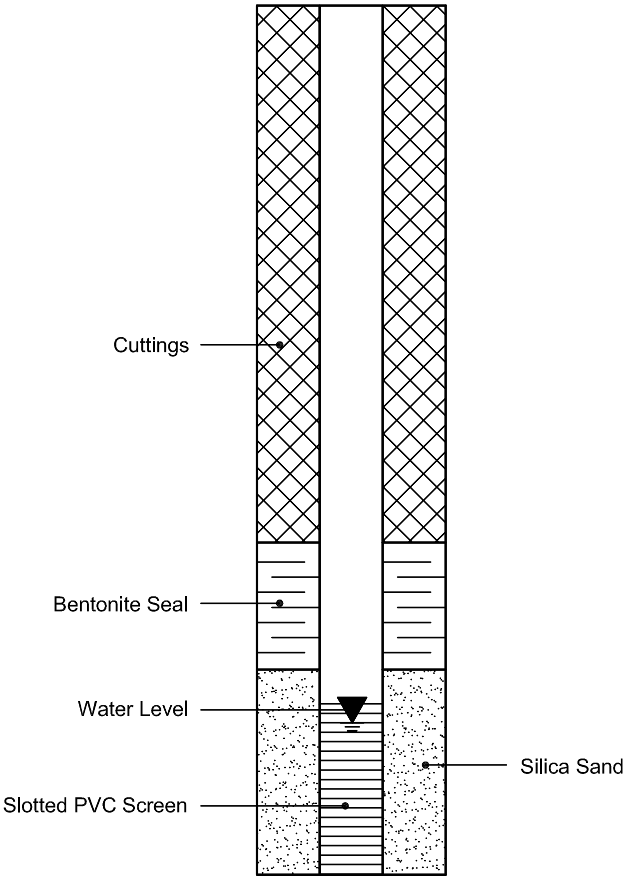


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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

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)	Depth (m)	Description	Elevation (m) / Depth (m)
0	0	Ground Surface	64.9
0	0	**TOPSOIL**	0.0
1.3	0.4	**FILL** Fine grained loamy sand, trace gravel, dark brown	64.5 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	
	4.7	**SAND** Fine to coarse grained sand, trace gravel	60.2 4.7
	5.2	**END OF BOREHOLE**	59.7 5.2

SAMPLE DATA

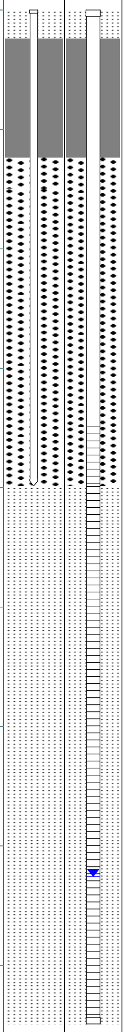
Type	Number	Sample	N or RQD	Recovery (%)
SS				
SS	1			45
SS	2			65
SS	3			43

Combustible Vapour ○ (ppm) 20 40 60 80

Total Organic Vapour ● (ppm) 20 40 60 80

WELLS

GP MW



Remarks

Elevation: 64.924 masl	Casing Elevation: 64.615 masl	Filter Pack Size: MW 6.7 mm/GP 9.5 mm	Datum: Geodetic
Easting: 368843.807	Well Casing Size: MW 50.8 mm/GP 12.7 mm	Well Material: Schedule 40 PVC	Checked by: KDH
Northing: 5029183.520	Screen Slot Size: MW 0.25 mm/GP 6.4 mm	Vapour Unit: N/A	Sheet: 1 of 1

Stratigraphic and Instrumentation Log: MW15-7

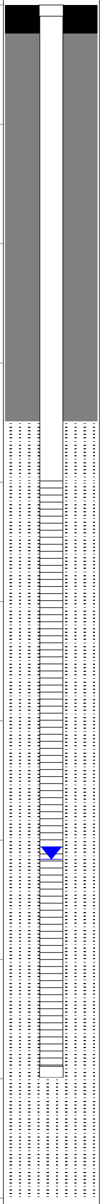
Project No: TZ10100106
Location: 945 Bank Street, Ottawa
Logged By: JFT
Drill Date: October 21, 2015
Hole Size: 127 mm

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

SUBSURFACE PROFILE		SAMPLE DATA					Combustible Vapour (ppm) 250 750 1250 / Total Organic Vapour (ppm) 20 60 100 140 180	Monitoring Well Details	Remarks
Description	Elevation (m)	Type	Number	Sample	N or RQD	Recovery (%)			
Ground Surface	64.51								
TOPSOIL	0.00								
FILL Gravel and sand, grey Fine loamy sand, greyish brown	64.12 0.40	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								

Depth: ft 0–23; m 0–7



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

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

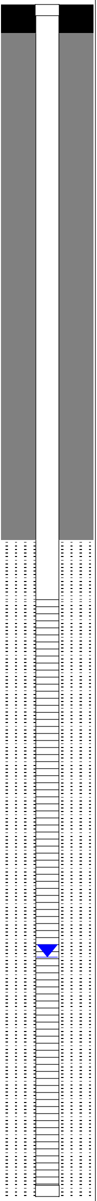
Amec Foster Wheeler
300-210 Colonnade Road
Ottawa, Ontario K2E 7L5

SUBSURFACE PROFILE / SAMPLE DATA

Description	Elevation (m)	Type	Number	N or RQD	Recovery (%)
Ground Surface	65.25				
ASPHALT	0.00				
FILL Fine to medium grained loamy sand, trace gravel, brown	64.86 0.40	SS	1		68.1
Fine to medium grained sand, trace coarse grained sand, brown		SS	2		70
Brownish grey					
Damp/moist Fine to medium grained sand		SS	3		65
Medium to coarse grained sand, moist/wet					
Very fine to fine grained sand, grey					
SAND Fine to coarse grained sand, trace gravel, grey, wet	60.68 4.57				
LOAMY SAND Fine to medium grained loamy sand and gravel, some pieces of rock	60.07 5.18	SS	4		55
END OF BOREHOLE	59.16 6.10				

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

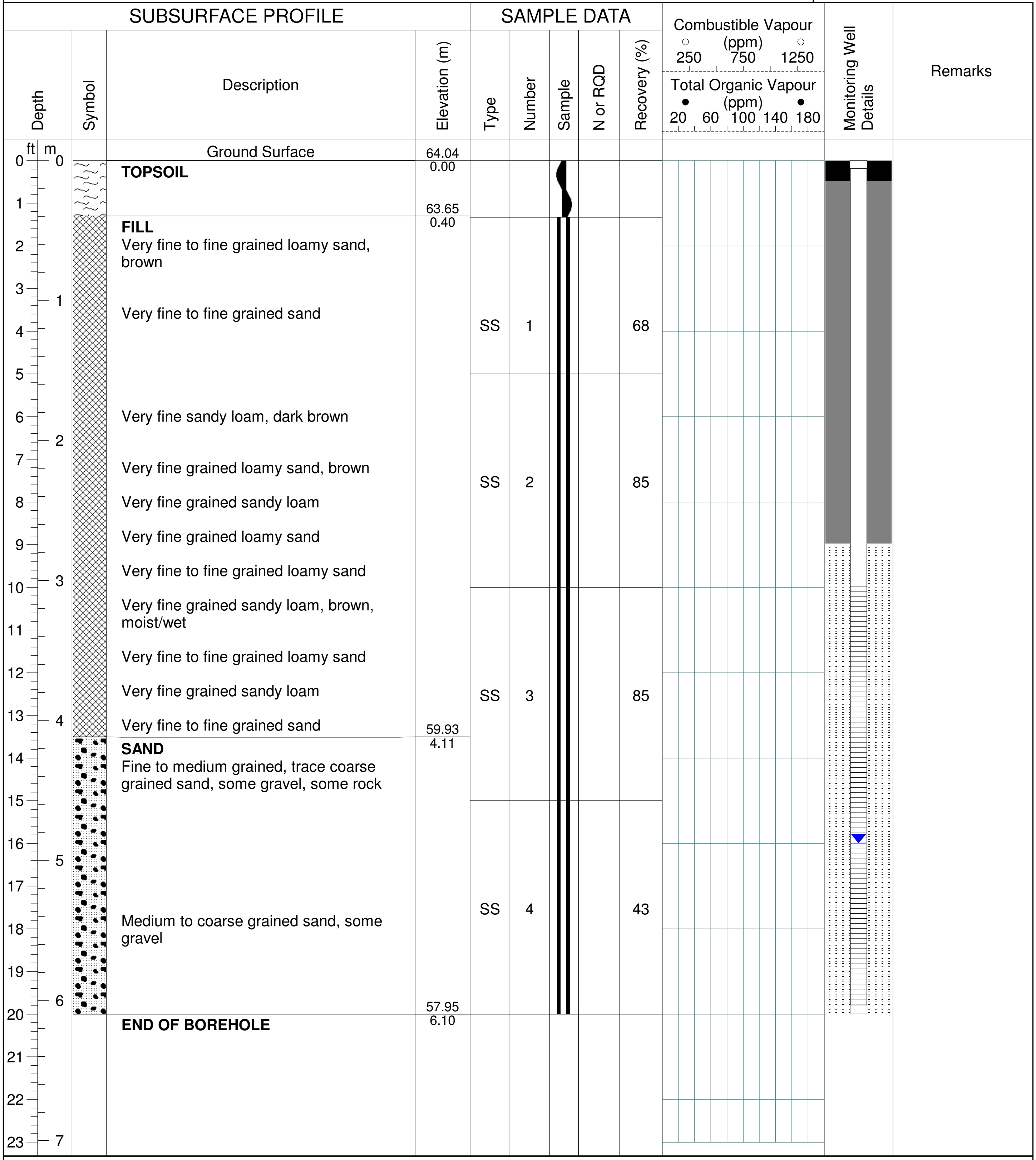
Datum: Geodetic
Checked by: KDH
Sheet: 1 of 1

Stratigraphic and Instrumentation Log: MW15-10

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



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

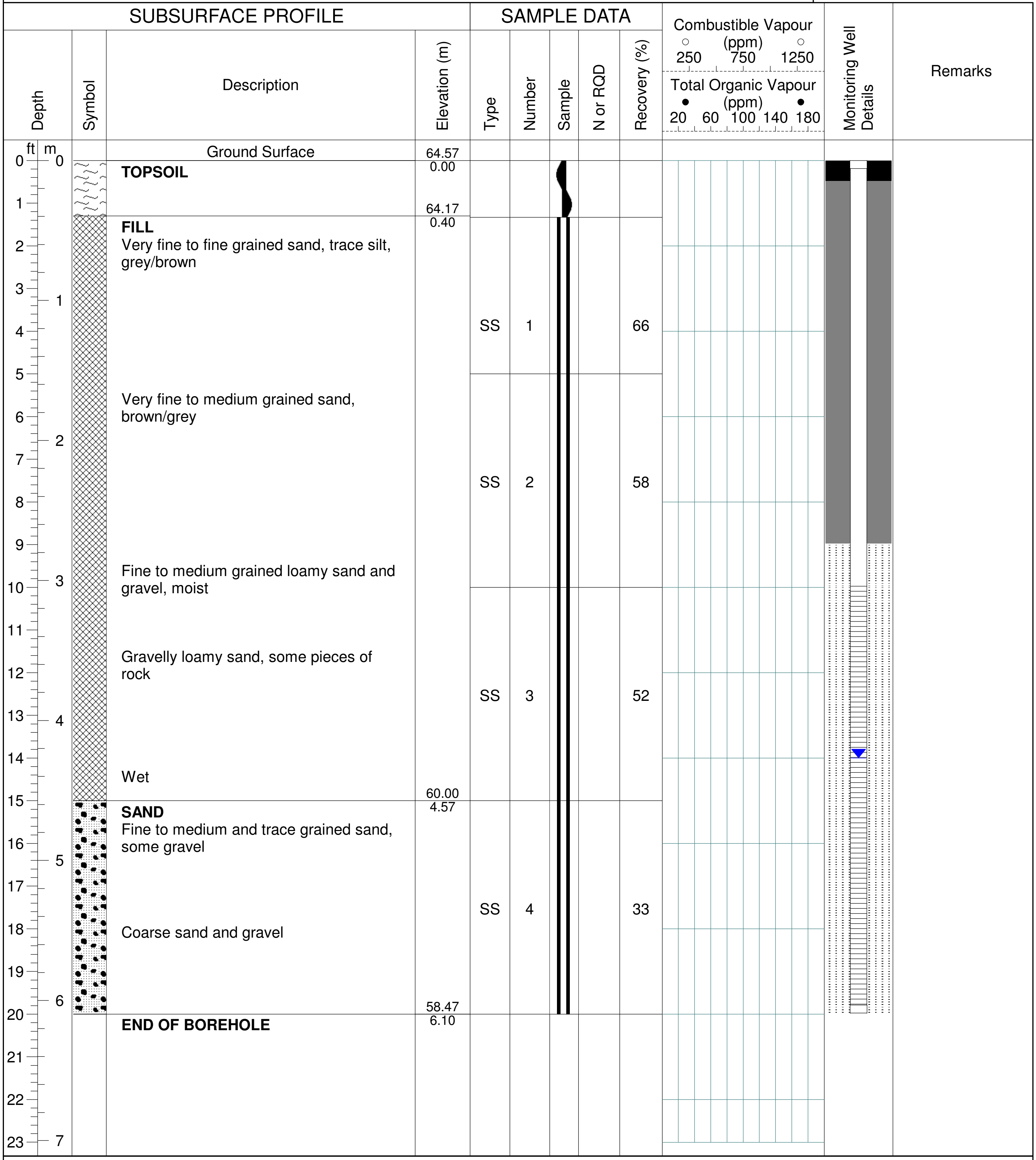
Datum: Geodetic
Checked by: KDH
Sheet: 1 of 1

Stratigraphic and Instrumentation Log: MW15-11

Amec Foster Wheeler
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



Description	Elevation (m)	Type	Number	Recovery (%)
Ground Surface	64.57 0.00			
TOPSOIL				
FILL Very fine to fine grained sand, trace silt, grey/brown	64.17 0.40	SS	1	66
Very fine to medium grained sand, brown/grey		SS	2	58
Fine to medium grained loamy sand and gravel, moist				
Gravelly loamy sand, some pieces of rock		SS	3	52
Wet				
SAND Fine to medium and trace grained sand, some gravel	60.00 4.57	SS	4	33
Coarse sand and gravel				
END OF BOREHOLE	58.47 6.10			

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

Photograph 1: BH 3-21 RC 11 and RC12

Photograph 2: BH 4-21 RC10

Photograph 3: BH 4-21 RC11.

Photograph 4: BH 5-21 RC10

Photograph 5: BH 6-21 RC9.

Photograph 6: BH 8-21 RC7.

Photograph 7: BH 8-21 RC8

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 - SLOPE STABILITY ANALYSIS CROSS-SECTIONS

FIGURES 3 & 4 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG6655-1 & PG6655-1A - TEST HOLE LOCATION PLAN

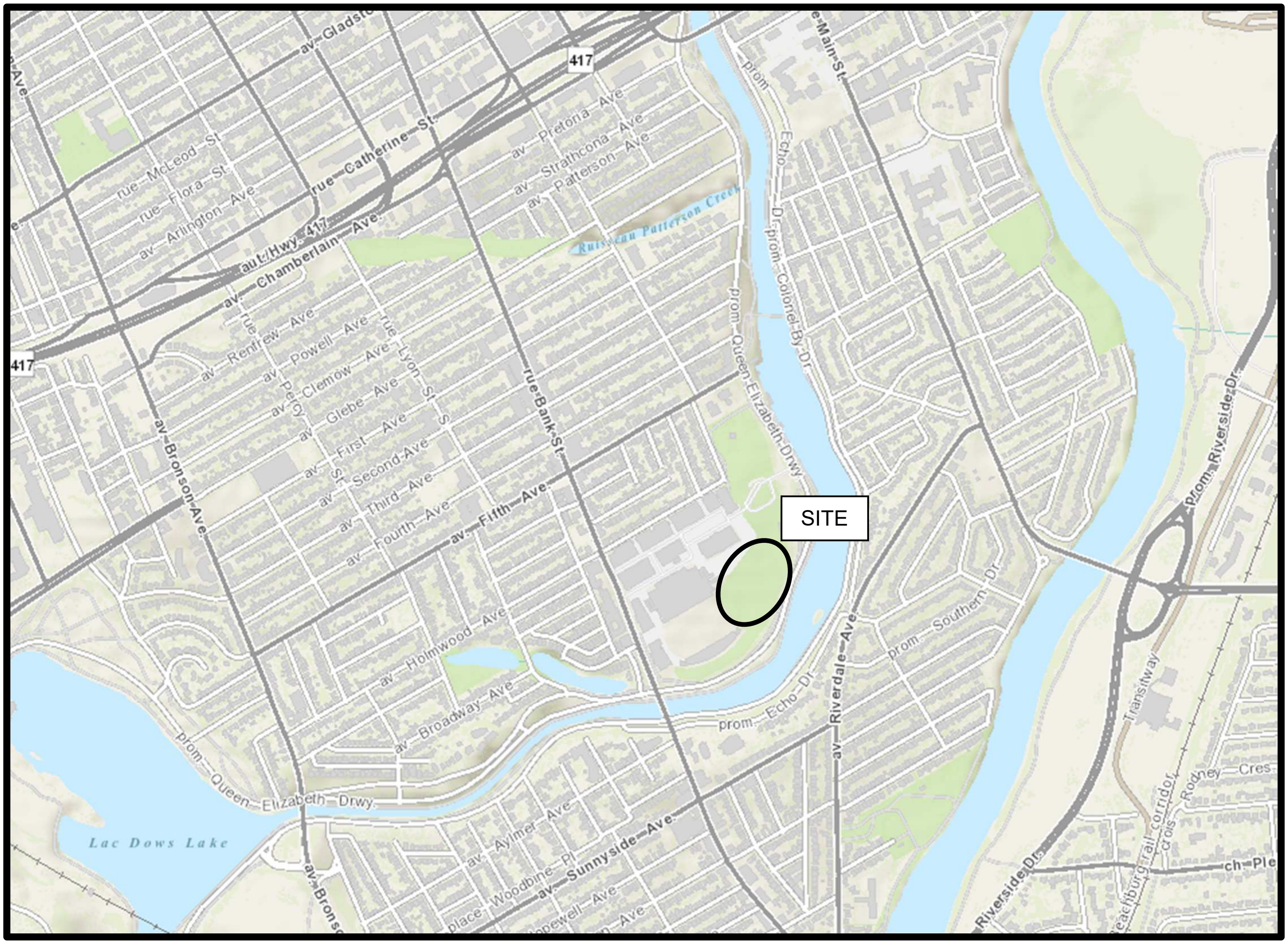
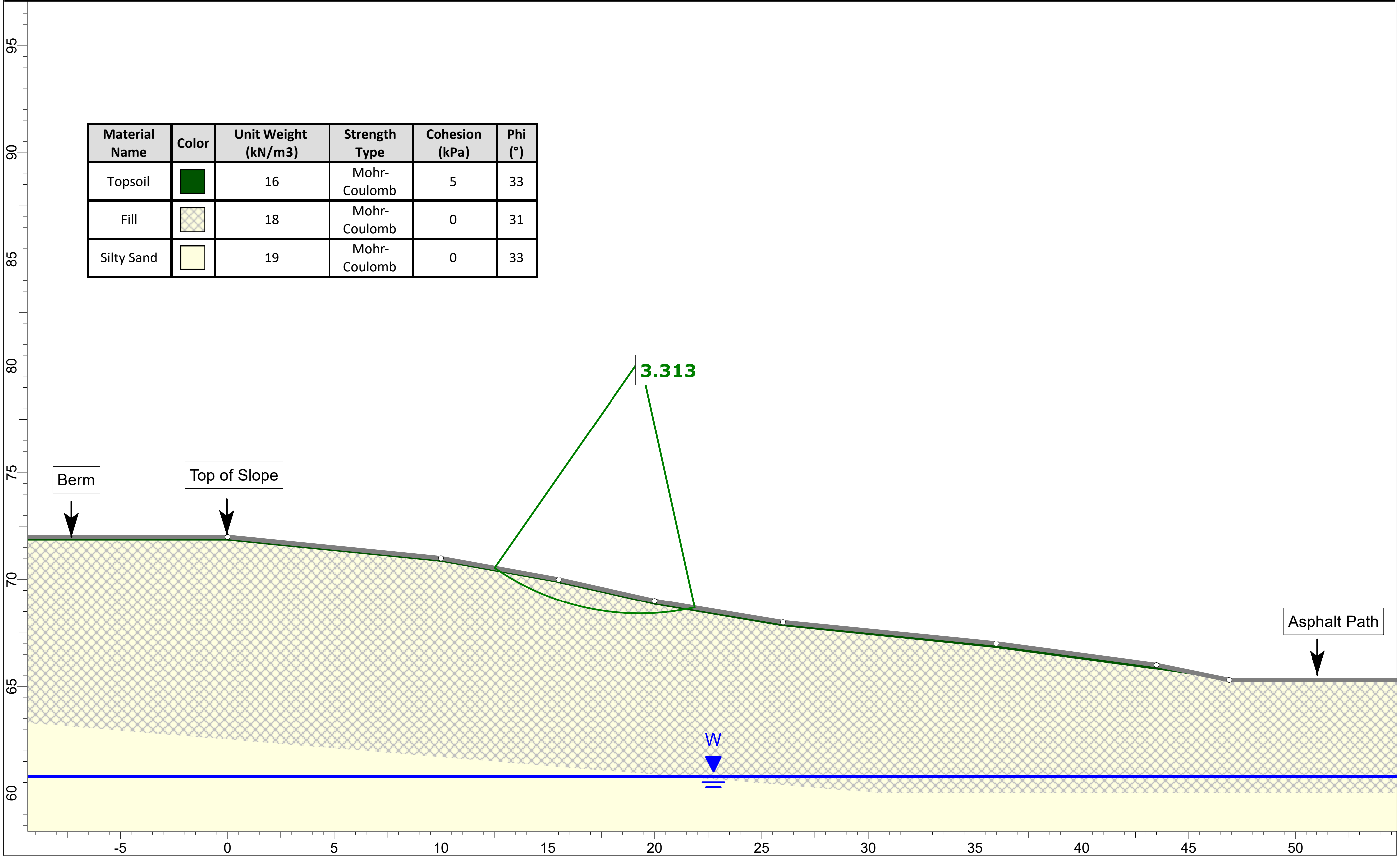


FIGURE 1

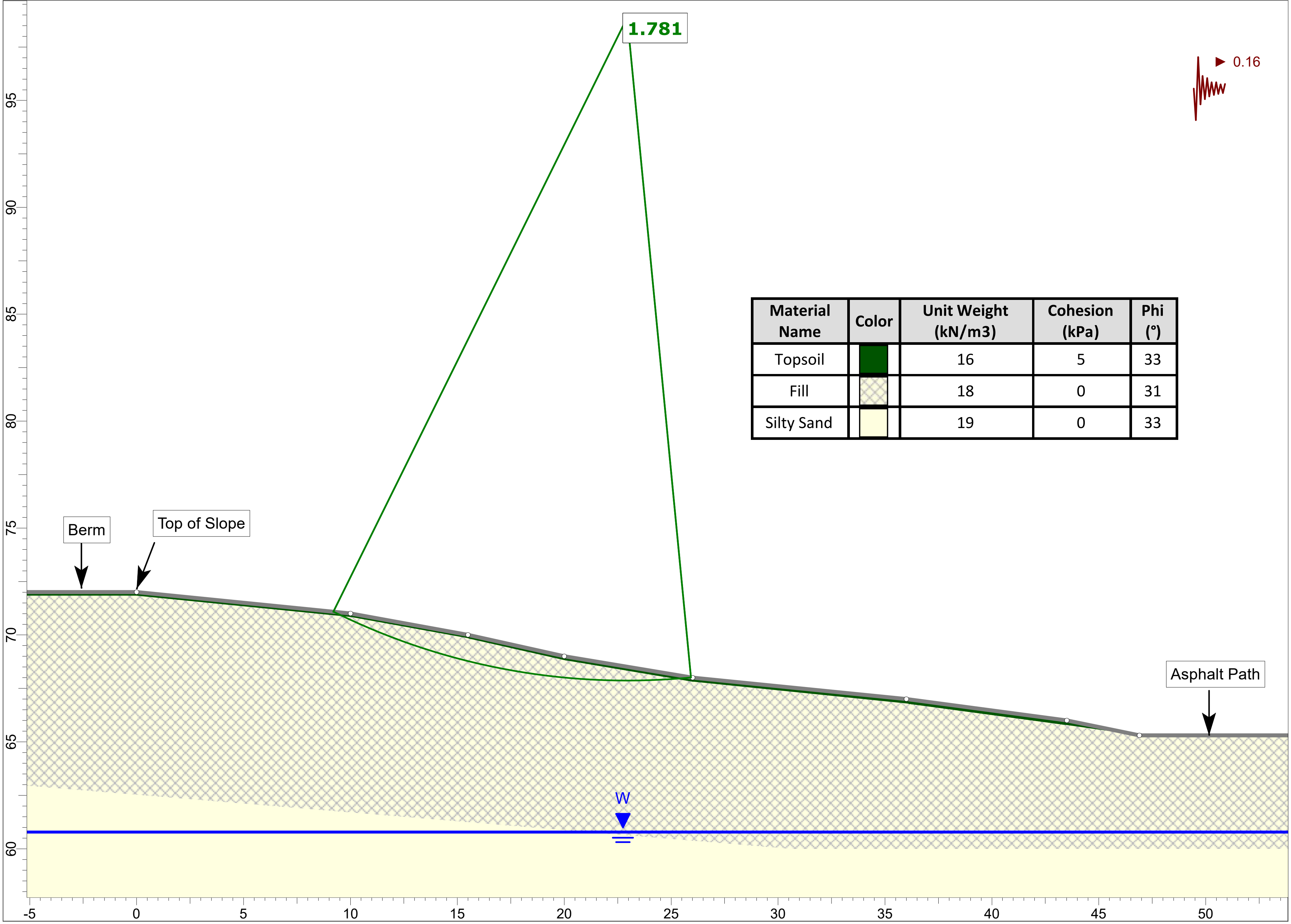
KEY PLAN

FIGURE 2A - SLOPE SECTION A - PROPOSED CONDITIONS - STATIC LOADING



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Topsoil	🟩	16	Mohr-Coulomb	5	33
Fill	▨	18	Mohr-Coulomb	0	31
Silty Sand	□	19	Mohr-Coulomb	0	33

FIGURE 2B - SLOPE SECTION A - PROPOSED CONDITIONS - SEISMIC LOADING



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Topsoil	🟩	16	5	33
Fill	▨	18	0	31
Silty Sand	🟨	19	0	33

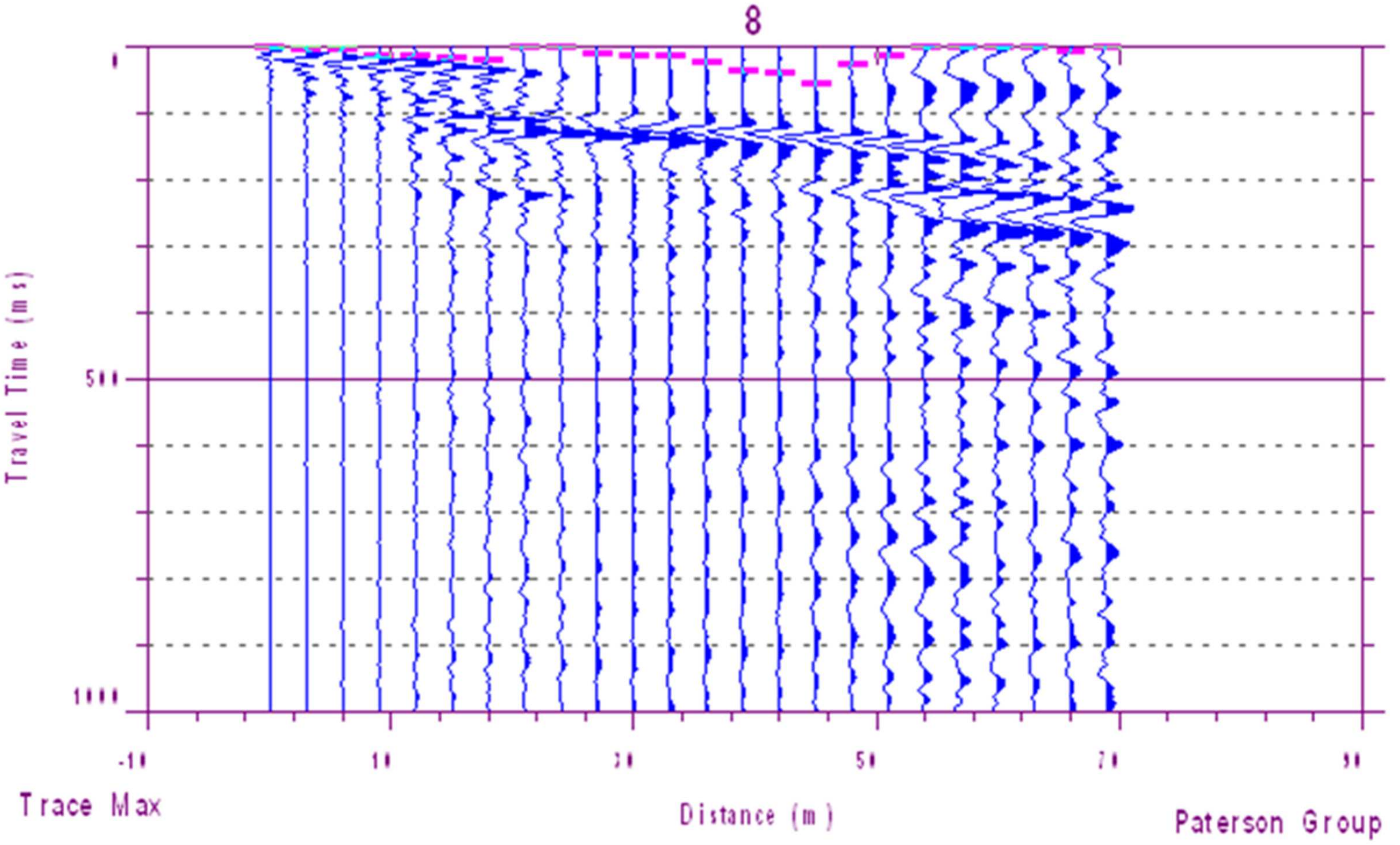


Figure 3 – Shear Wave Velocity Profile at Shot Location -3.0 m

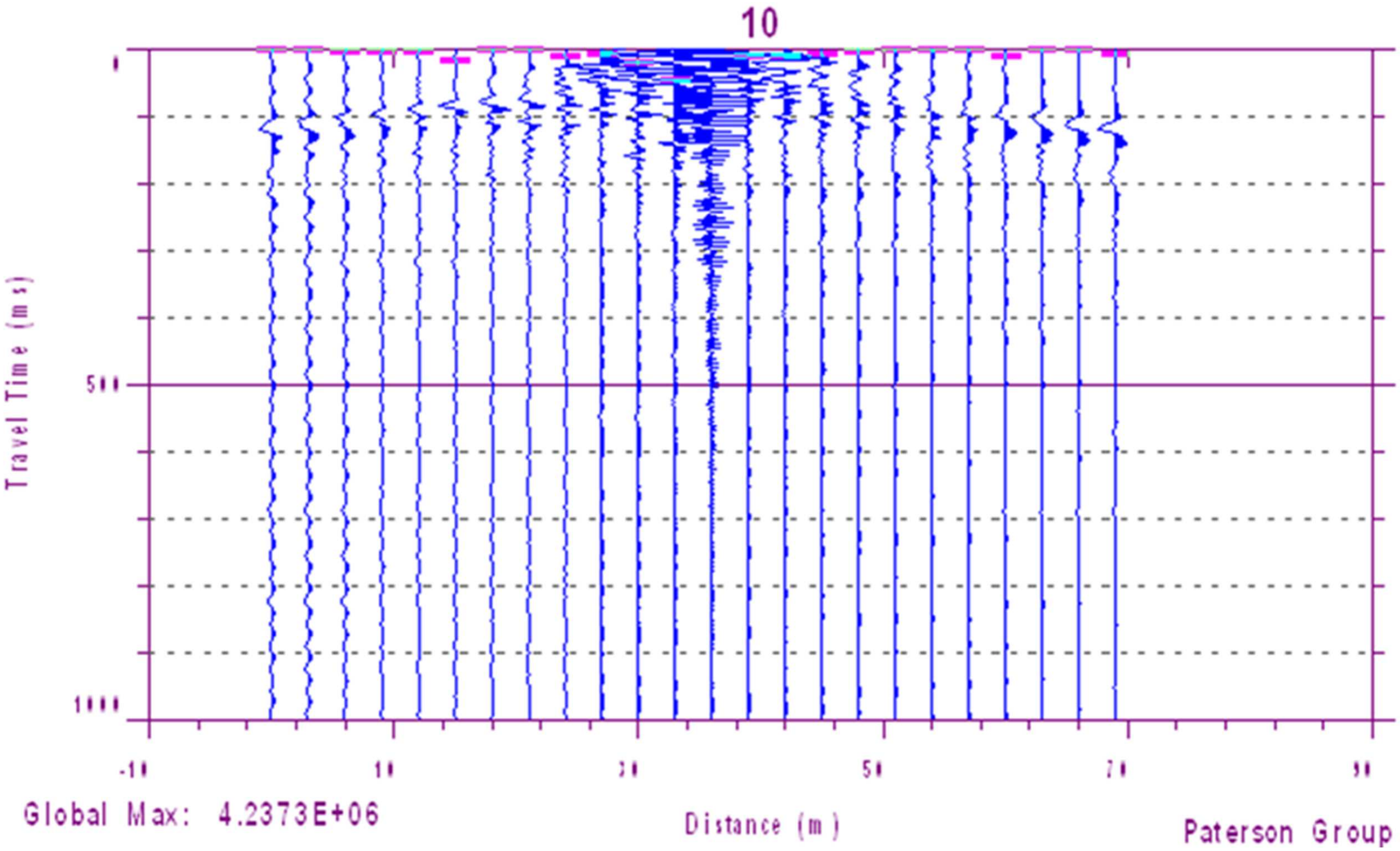
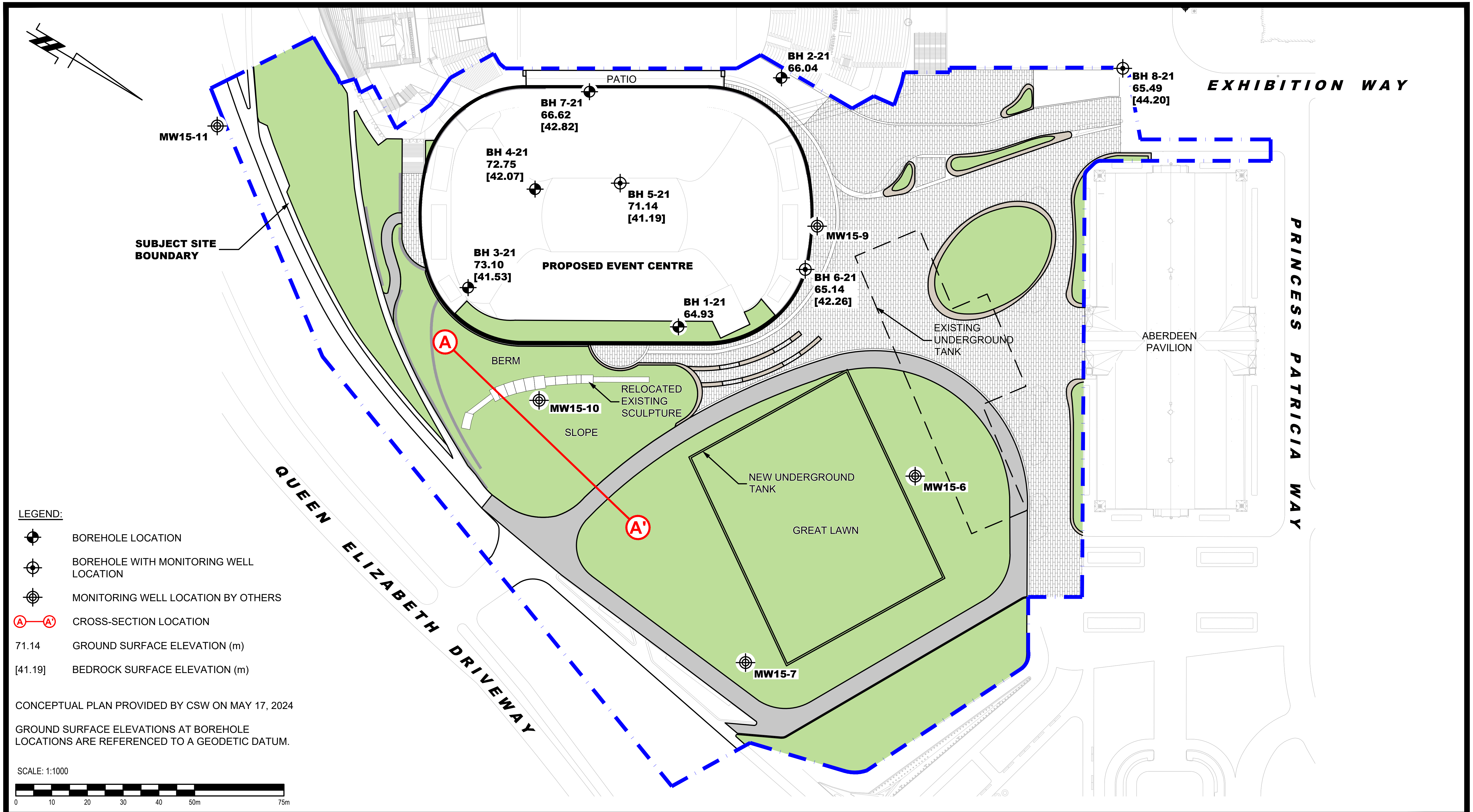


Figure 4 – Shear Wave Velocity Profile at Shot Location 34.5 m



LEGEND:

⯐	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
⊕	MONITORING WELL LOCATION BY OTHERS
Ⓐ—Ⓐ'	CROSS-SECTION LOCATION
71.14	GROUND SURFACE ELEVATION (m)
[41.19]	BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY CSW ON MAY 17, 2024

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

SCALE: 1:1000

0 10 20 30 40 50m 75m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED SITE BOUNDARY	06/08/2024	FC

CITY OF OTTAWA
GEOTECHNICAL INVESTIGATION
LANDSDOWNE PARK REDEVELOPMENT
PROPOSED EVENT CENTRE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 05/2024
Drawn by: ZS	Report No.: PG6655-1
Checked by: FC	Dwg. No.: PG6655-1
Approved by: DP	Revision No.: 1

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11x17

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

CONCEPTUAL PLAN PROVIDED BY CSW ON MAY 17, 2024

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

SCALE: 1:2000

0 25 50 75 100 125 150m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED SITE BOUNDARY	06/08/2024	FC

CITY OF OTTAWA
GEOTECHNICAL INVESTIGATION
LANDSDOWNE PARK REDEVELOPMENT
PROPOSED EVENT CENTRE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2000
Date: 05/2024
Drawn by: ZS
Report No.: PG6655-1
Checked by: FC
Dwg. No.: PG6655-1A
Approved by: DP
Revision No.: 1

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11x17