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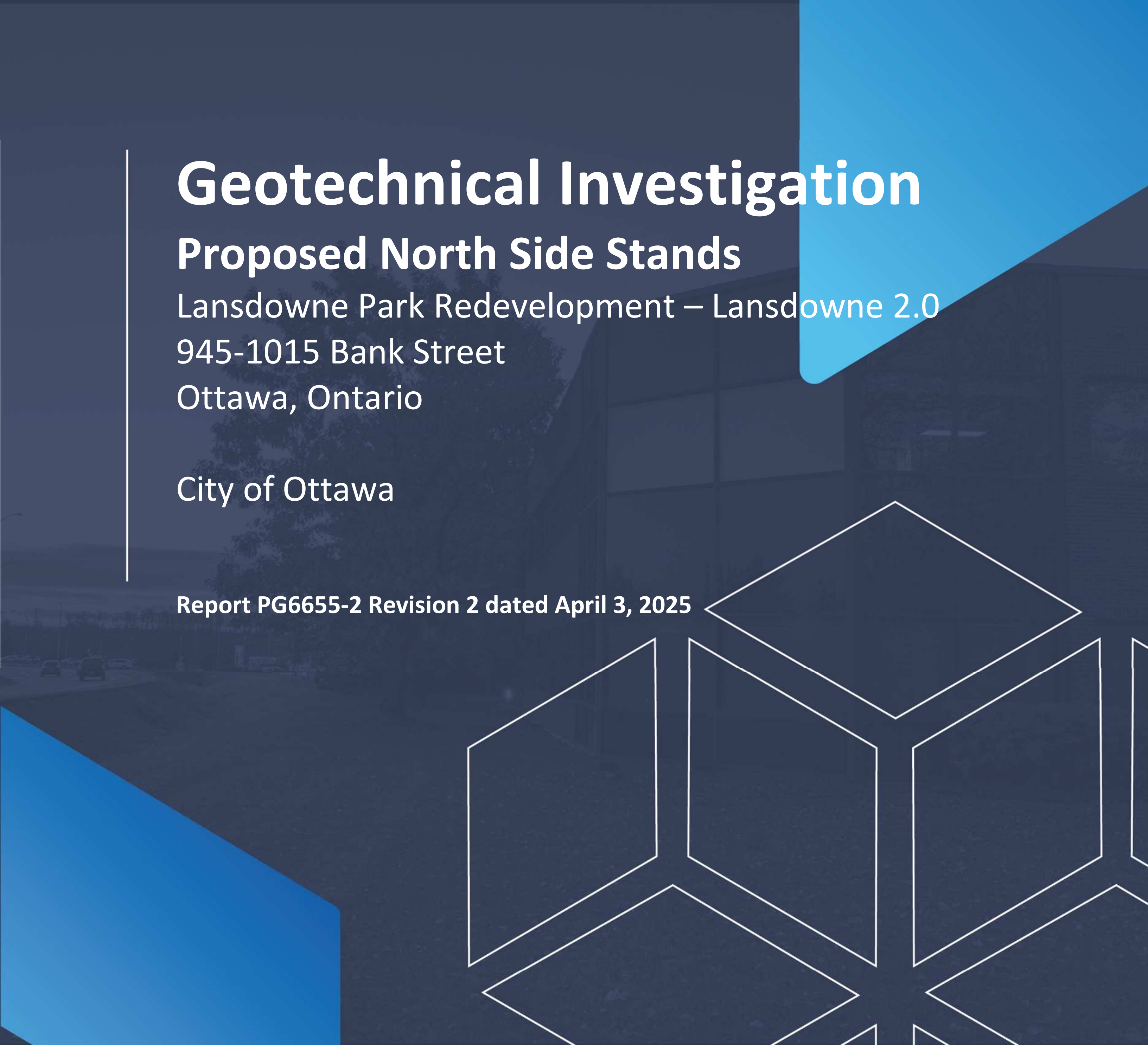

Geotechnical Investigation

Proposed North Side Stands

Lansdowne Park Redevelopment – Lansdowne 2.0
945-1015 Bank Street
Ottawa, Ontario

City of Ottawa

Report PG6655-2 Revision 2 dated April 3, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by City of Ottawa to conduct a geotechnical investigation for the proposed North Side Stands of the proposed Lansdowne Park Redevelopment (Lansdowne 2.0) 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 test holes.

- 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 available drawings, it is understood that the proposed project will consist of a proposed stadium stands structure which would host associated concourses, offices, operations and event spaces. Further, the stands' structure will be provided with one level of underground parking within its basement level.

The facility will be surrounded by landscaped and hardscaping areas, and a connection to the proposed arena located within Phase 1 of the proposed Lansdowne Park Redevelopment Project. It is also expected that the proposed building will be municipally serviced.

It is understood that the existing stands and associated structures will be demolished in support of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

A field investigation was completed at the subject site by Paterson between October 15 and November 1, 2024. At that time, a total of six (6) boreholes were advanced to a maximum depth of 27.0 m below existing ground surface. Previous investigations were completed by this firm on October 25, November 17, and November 18, 2021, and consisted of advancing a total of three (3) boreholes to a maximum depth of 24.1 m below existing grade. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG6655-2 - Test Hole Location Plan included in Appendix 2.

The 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 Paterson personnel under the direction of a senior engineer from our Geotechnical Division.

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 3-24, BH 4-24, BH 5-24, BH 6-24, BH 7-24, BH 8-21, and BH 9-21 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

All boreholes were fitted with monitoring wells to allow for groundwater level monitoring. 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.
- 32 or 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No.3 silica sand backfill within annular space around screen.
- Bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

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

3.2 Field Survey

The borehole locations were selected by Paterson personnel in a manner to provide general coverage of the proposed development, taking into consideration existing site features. The borehole locations and ground surface elevations were referenced to a geodetic datum. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG6655-2 - 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. A total of three (3) samples were submitted for grain size distribution analysis. The results are presented in Subsection 4.2 and on Grain Size Analysis Distribution Testing presented in Appendix 1.

Unconfined compressive strength testing was carried out by Paterson on bedrock samples from boreholes BH 5-24 and BH 7-24. The results of the testing by Paterson are discussed in section 4.2 and are provided in Appendix 1.

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless directed otherwise.

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.

3.5 Hydraulic Conductivity (Slug) Testing

Hydraulic conductivity (slug) testing was conducted at select borehole locations to evaluate the hydraulic properties of the overburden material within the anticipated saturated depth of excavation at the subject site. Slug testing (rising head) was completed in accordance with ASTM Standard Test Method D4404 - Field Procedure for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers. The slug testing results have been included in Appendix 1 of this report.

Assumptions inherent to the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter. The assumption regarding aquifer storage is considered to be appropriate for groundwater inflow through the overburden aquifer.

The assumption regarding screen length and well diameter is considered to be met based on a screen length of 3 m and a diameter of 0.03 to 0.05 m.

While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site.

The Hvorslev analysis is based on the line of best fit through the field data (hydraulic head recovery vs. time), plotted on a semi-logarithmic scale. The results of the testing are further discussed in Subsection 4.4.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by the existing north stands, rink and associated structures. The subject site within the Lansdowne Park Development is bound by a high-rise residential structure followed by Bank Street to the west, commercial units and buildings to the north, TD Place Stadium to the south, and commercial buildings followed by existing landscaped areas and Queen Elizabeth Driveway to the east.

4.2 Subsurface Profile

Overburden

Generally, the soil profile at the borehole locations consists of a layer of either asphalt or concrete underlain by fill material and further by a deposit of silty sand. The silty sand deposit is further underlain by a deposit of glacial till.

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. A 75 mm thick layer of asphaltic concrete was observed below the fill material at the location of BH 4-24. Based on the encountered fill thickness, the native, in-situ undisturbed soils were encountered at approximate geodetic elevations ranging between 61.8 and 64.4 m.

The fill layer was observed to be underlain by a compact, brown silty sand with varying amounts of clay and gravel. The silty sand layer was observed to extend to approximate geodetic elevations ranging between of 60.2 to 61.8 m.

The silty sand layer was underlain by a compact to very dense deposit of glacial till consisting of 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.

Grain Size Distribution

Grain size distribution was completed on three (3) selected soil samples. The results of the grain size analysis are summarized in Table 1 and presented on the Grain-Size Distribution Testing Results sheets in Appendix 1.

Table 1 - Summary of Grain Size Distribution Analysis

Test Hole	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 5-24	SS5	67.7	28.5	3.8	
BH 7-24	SS5	49.7	40.6	9.7	
BH 8A-24	SS7	45.1	42.2	12.7	

Bedrock

Bedrock was cored at the majority of the test holes and encountered at approximate elevations of 44.1 to 43.6 m within the subject site. The cored 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 limestone bedrock. Photographs of the recovered 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.

Unconfined Compressive Strength Testing on Bedrock Core Samples

Two (2) bedrock cores obtained by Paterson as part of the current field investigation were tested for unconfined compressive strength. The samples consisted of grey limestone bedrock as based on Paterson's observations. The results are summarized in Table 2 below and presented on Unconfined Compressive Strength Testing Results on Appendix 1.

Table 2 - Summary of Unconfined Bedrock Compressive Strength Testing Results

Test Hole	Sample	Test Core Depth (m)	Test Core Elevation (m)	Unconfined Compressive Strength (MPa)
BH 5-24	RC11	17.3	45.24	69.0
BH 7-24	RC8	18.7	43.84	75.6

4.3 Groundwater

Groundwater levels were recorded at each borehole location instrumented with a monitoring device. The groundwater level readings completed during the current investigation are presented in Table 3 and in the Soil Profile and Test Data Sheets in Appendix 1. It should be noted that groundwater levels are subject to seasonal fluctuations and the influence of the Rideau Canal, which is located south and southeast of the subject site.

Therefore, groundwater levels may vary at the time of construction.

Table 3 – Groundwater Elevation Summary

Test Hole	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 3-24	66.33	6.38	59.95	November 24, 2024
BH 3-24	66.33	6.17	60.16	November 24, 2024
BH 4-24	66.18	5.94	60.24	November 24, 2024
BH 5-24	62.54	2.28	60.26	November 24, 2024
BH 5A-24	62.54	2.20	60.34	November 24, 2024
BH 6-24	62.49	2.28	60.21	November 24, 2024
BH 6A-24	62.49	2.32	60.17	November 24, 2024
BH 7-24	62.54	2.41	60.13	November 24, 2024
BH 7A-24	62.54	2.23	60.31	November 24, 2024
BH 8A-24	66.05	6.04	60.01	November 24, 2024

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 undertaken during previous rounds of investigations and monitoring undertaken by Paterson.

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

4.4 Hydraulic Conductivity Testing Results

Hydraulic conductivity (slug) tests were conducted at five (5) monitoring well locations on December 8, 2021 and November 14, 2024, to evaluate the hydraulic properties of the overburden material at the test locations.

The measured hydraulic conductivity (K) values ranged between approximately 2.80 x 10^{-4} to 7.26 x 10^{-4} m/sec. The results are consistent with similar materials Paterson has encountered on other sites and typical published values for silty sand and glacial till with a silty sand matrix. The range in hydraulic conductivity values is due to the variability in the composition and compactness of the silty sand and glacial till deposit.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the construction of the proposed North Side Stands structure. In view of the anticipated building loads, the proposed structure may be founded on conventional spread footings placed on an undisturbed compact to dense silty sand or a very dense to compact glacial till bearing medium. All contractors should be prepared for handling and removing boulders and over-sized boulders throughout the subject site.

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

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

5.2 Site Grading and Preparation

Stripping Depth

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

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

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type I or 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 sandy soil fill could be placed as general landscaping fill and beneath exterior parking where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted with a suitably sized heavy vibratory roller. 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.

Footing Subgrade Preparation – Mud Slabs

It is anticipated the subgrade soils will become readily disturbed by construction traffic due to their in-situ saturated state and becoming dewatered for foundation construction. Therefore, it is recommended that a minimum 75 mm thick mud slab layer be placed over the prepared bearing medium for all footings once the bearing surface has been reviewed and approved by Paterson personnel. The mud slab concrete is recommended to consist of a minimum 15 MPa (28-day 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 design parameters have been provided on the assumption that foundation construction and subgrade preparation conditions would be undertaken in the dry and that groundwater levels would be maintained below the depth of the proposed works.

However, some options have been provided to accommodate conditions where this may not be feasible and where subgrade conditions differ in a localized area due to works impacted by either demolition activities or if soils that are not in accordance with the design assumptions are encountered at the design founding elevation for localized footings.

Further, 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.

Conventional Shallow Foundations – Native In-Situ Soils

Using continuously applied loads, footings for the proposed structure placed over an undisturbed, dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **250 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **400 kPa**. It should be understood that the glacial till deposit has been encountered below the silty sand deposit at test holes undertaken by Paterson throughout the subject site.

Footings placed over an undisturbed, compact silty sand bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**.

Conventional Shallow Foundations – Engineered Fill

Footings may be placed on suitably placed fill to raise the subgrade surface in areas where soils that are not in accordance with the design requirements are encountered at the design founding elevation for footings, or, where demolition works result in a bearing surface that is deeper than the design bearing surface elevation.

Where footings are placed upon a layer of engineered fill (i.e., OPSS Granular A, OPSS Granular B Type I or II crushed stone) capped with a minimum 300 mm thick layer of OPSS Granular A and founded upon either undisturbed, compact silty sand or dense glacial till may be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**.

Auxiliary footings (i.e., footings not associated within the main buildings foundation located within the basement level) placed upon site-generated and Paterson-reviewed and -approved sandy fill placed in maximum 300 mm thick loose lifts, compacted to a minimum of 98% of the materials SPMDD and capped with a minimum 300 mm thick layer of OPSS Granular A may be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**. This condition may be considered for footings supporting exterior columns supporting lightly-loaded structures located adjacent to or within the vicinity of the proposed stands structure.

All fill placed below footings must be placed in 300 mm maximum thick loose lifts and compacted to a minimum of 98% of the materials SPMDD. Mud slabs are not required where footings are placed on suitably prepared and approved engineered fill.

Conventional Shallow Foundations – Lean-Concrete In-Filled Trenches

In the event that the designed underside of footing (USF) elevation is located upon undisturbed, compact silty sand and is designed for undisturbed, dense glacial till, consideration could be given to placing the footing upon a trench of lean-concrete extending to the sought-bearing medium and up to the design USF elevation. Further, this option would be able to be considered for cases where consideration will not be given to either adjusting the foundation design for the compact, silty sand, or lowering the footing to the dense, glacial till.

This option would consist of sub-excavating the bearing surface to a depth corresponding to the appropriate bearing medium and using the sidewalls of the excavation as the temporary formwork. If the subsoils are drained up to the depth of the sub-excavation, it is expected the sidewalls would remain relatively vertical. Workers would not be permitted to enter the sub-excavations where near-vertical sidewalls are provided for this purpose.

Once the bearing surface has been reviewed and approved by Paterson personnel, lean concrete, consisting of minimum 15 MPa (28-day compressive strength) concrete may be used to raise the subgrade from the undisturbed, dense glacial till up to the design USF. The concrete may be cast below the water levels (if present) and cured in submerged conditions, if required and as assessed by Paterson at the time of construction. The lean concrete in-fill is recommended to extend a minimum of 150 mm horizontally beyond all faces of the overlying footing footprint.

Footings placed upon a trench of lean-concrete extending to the undisturbed, dense glacial till bearing surface may be designed using a bearing resistance value at SLS of **250 kPa** and a factored bearing resistance value at ULS of **400 kPa**.

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. A geotechnical resistance factor of 0.5 has been incorporated in the above-noted bearing resistance values.

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.

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.

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 with crushed stone fill) 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.

Deep Foundations – Drilled-Shaft Caissons

Where required, drilled shaft caissons can be considered for foundation support of auxiliary structures. Cast-in-place caissons should be installed by driving a temporary steel casing and excavating the soil through the casing. A minimum of 35 MPa concrete should be used to in-fill the caissons. The caissons are to be structurally reinforced over their entire length as advised by the structural design consultant. All caissons are to be verified to be clean of debris and soil prior to placement of concrete and by Paterson field personnel.

It is expected the caisson installation contractor will encounter cobbles and boulders throughout the installation process, therefore, the contractor should be prepared to advance past cobbles and boulders, including removing cobbles and boulders that accumulate within the caisson casing. Further, the contractor should be equipped to manage the associated groundwater influx within the casings due to the anticipated embedment depth below the local groundwater table.

The compressive resistance for such caissons is directly related to the point bearing resistance of the glacial till and the skin friction of the caisson. Table 4 below presents the estimated capacity for different typical caisson sizes founded within and upon an in-situ, dense glacial till bearing surface.

The minimum recommended centre-to-centre caisson spacing is 3 times the caisson diameter to minimize additional settlement from group effects. Group effects, or closer spacing, may be accommodated by reduced capacities to mitigate unacceptable long-term post-construction total and differential settlements. The bases of caissons that may be founded higher than adjacent caissons should be planned such that deeper caissons are not extended within a lateral support zone extending down and out at a 1.5H:1V from the base of the higher caisson.

It is anticipated the above-noted caissons will be considered to support the proposed elevator shaft at the south-stands connection and portions of the Event Centres foundation walls that will be interconnected to the permanent shoring system by headed shear connectors.

It should be understood that cased holes will be required to be advanced across subsoils being of permeable nature and located below the groundwater table. Casing will be required to prevent excessive caving and seepage during the caisson installation as well as to provide adequate support for removing soil to accommodate the caisson. Testing and inspections of caisson implementation, cleaning and capacities are recommended to be undertaken by Paterson personnel at the time of construction.

Table 4 – Caisson Axial Capacities at Serviceability Limit States (SLS) for Soil Bearing

Depth of Caisson Base Below Elevation 60.5 m	Underside of Caisson Elevation	Caisson Diameter (mm)											
		375	**450**	**500**	**600**	**775**	**850**	**900**	**1,000**	**1,100**	**1,200**	**1,350**	**1,500**
1	**59.5**	92	130	158	224	368	440	492	604	727	862	1,086	1,336
2	**58.5**	105	145	176	245	395	469	523	638	765	904	1,133	1,388
3	**57.5**	120	163	196	269	425	503	559	678	809	952	1,187	1,448
4	**56.5**	137	183	218	296	460	541	599	723	858	1,005	1,247	1,515
5	**55.5**	155	206	243	326	499	584	644	773	914	1,065	1,315	1,590
6	**54.5**	174	231	271	359	542	631	694	828	974	1,132	1,389	1,673
7	**53.5**	189	256	301	395	588	682	748	888	1,040	1,204	1,471	1,763
8	**52.5**	205	276	328	435	639	738	807	954	1,112	1,282	1,559	1,861
9	**51.5**	220	296	351	477	694	797	870	1,024	1,190	1,367	1,654	1,967
10	**50.5**	236	315	374	507	752	862	938	1,100	1,273	1,458	1,756	2,080
11	**49.5**	251	335	398	537	815	930	1,011	1,181	1,362	1,555	1,865	2,202

12	**48.5**	267	355	421	567	873	1,003	1,088	1,267	1,456	1,658	1,981	2,330
13	**47.5**	283	375	444	597	917	1,074	1,170	1,357	1,556	1,767	2,104	2,467
14	**46.5**	298	395	467	627	960	1,124	1,240	1,454	1,662	1,882	2,234	2,611
15	**45.5**	314	415	490	657	1,004	1,175	1,296	1,555	1,773	2,003	2,370	2,763
16	**44.5**	329	435	513	687	1,048	1,226	1,351	1,620	1,890	2,131	2,514	2,922
17	**43.5**	350	455	536	717	1,092	1,276	1,407	1,686	1,989	2,265	2,664	3,089

Notes:
- Reinforced caissons to be designed by others, capacities provided herein are considered geotechnical capacities for friction-end bearing caissons considered throughout Phase 1 of the proposed development.
- This design information is only considered applicable to Phase 1 of the proposed development.
- A geotechnical resistance factor of 0.4 has been applied to the above-noted capacities.
- The above-noted capacities derive resistance from a combination of skin friction and end-bearing resistance.
- The above-noted capacities are based on the bottom of the caisson being located below a geodetic elevation of 60.5 m. Higher elevations are not considered suitable for support of friction or end-bearing caissons due to the presence of loose to compact sand.
- Ultimate Limit States (ULS) resistance may be considered as 1.5 times the above-noted SLS resistance values.
- Capacities for caisson diameters not identified herein may be provided upon request.
- The above-noted capacities area based on founding the caissons with an in-situ, dense glacial till deposit reviewed and approved by Paterson personnel prior to the installation of reinforcing steel cages and concrete.

Deep Foundations – End-Bearing and Rock Socketed Caissons

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

Applicable pile resistance at SLS values and factored pile resistance at ULS values are provided in Table 5. Additional resistance values can be provided if available pile sizes vary from those detailed in Table 5. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

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

Table 5 - End Bearing Pile Foundation on Bedrock Design Data

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

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

Table 6 below presents the estimated capacity (factored ULS) for different typical caisson sizes for a rock bearing caisson and rock socketed caisson extending 3 m into sound rock.

Table 6 - Caisson Pile Capacities for Bedrock Embedment

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

Notes:
- 3 m rock socket in sound bedrock
- Reinforced caisson and rock socket, when applicable
-0.4 geotechnical factor applied to the shaft capacity

5.4 Design for Earthquakes

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

Field Program

The shear wave testing was located as presented in Drawing PG6655-2A - 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

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. It is expected the sand will become disturbed by constant construction traffic; therefore, provisions should be made to proof-rolling the soil subgrade using heavy vibratory compaction equipment under dry and above-freezing conditions prior to placing any fill in support of the basement slab.

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.

An underfloor drainage system will be advised to be incorporated in the design of the lowest level footprint. The system would consist of a series of perforated pipe subdrains throughout the basement footprint connected to the building’s sump pit, or, a nearby storm sewer outlets where a gravity connection may be facilitated. The design of this system would be prepared by Paterson for incorporation in the associated design drawings depicting the system.

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

Rigid Pavement Design – Basement Level

For design purposes, it is recommended that the rigid pavement structure for the lower level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 7 below.

Table 7 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
By Others	**32 MPa Concrete – Category C2 Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE Fill or OPSS Granular B Type I or II material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level.

The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Flexible Pavement Design – At-Grade Areas

The flexible pavement structure presented in Table 8 and Table 9 should be used for at-grade car parking areas and access lanes and heavy loading parking areas, if required.

Table 8 - 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 9 - 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
400	**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.	

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.).

These recommendations would be site- and situation specific and only able to be confirmed at the time of construction. It should be noted that the above-noted pavement structures are not intended to support construction traffic without carrying provisions for scarifying contaminated stone (i.e., stone mixed with non-crushed stone soils).

Temporary access roads that would be later used for permanent conditions should be underlain by a layer of woven geotextile layers to limit pumping of fines during the construction period.

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.

Landscaping and Pedestrian Pathways

It is recommended that cross-sections for landscaped and hardscaped areas intended for pedestrian traffic be reviewed by Paterson from a geotechnical perspective during the design phase to ensure adequate drainage and support is provided by the proposed fill layers.

5.8 Rock and Soil Anchor Design

Soil and Rock Anchors for Tiebacks

Typically, tiebacks in the Ottawa area are extended below the bedrock formation due to the higher available capacities and relatively shallow depth with respect to shoring system construction. However, given the presence of relatively dense glacial till throughout the subject site, consideration may be given to utilizing this deposit to support grouted tiebacks.

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

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed. Centre-to-centre spacing between anchors should be at least four times the anchor hole diameter and greater than 1/5 of the total anchor length (minimum of 1.2 m) to lower the group influence effects. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout does not flow from one hole to an adjacent empty one.

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

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic.

Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp.

The following design information may be considered for the design of soil and rock anchors to be used as tiebacks for the shoring system:

Soil Anchors

Soil anchors, or tiebacks, may be grouted in place by the use of a tremie tube (gravity) or under pressure. For gravity-grouted anchors, a factored grout-to-soil bond of **100 kPa** may be used for the dense glacial till encountered throughout the subject site. A factored grout-to-soil bond of **180 kPa** may be used if the anchors are grouted in a minimum pressure of 10 Bar.

It is recommended to use a minimum 40 MPa compressive strength non-shrink grout for this purpose and that a minimum unbonded length of 4.5 m be considered for these types of anchors.

At this time, it is not recommended to derive grout-to-soil bond capacity from the in-situ compact sand layer, and all capacity for soil anchors should be derived from bonds facilitated within the dense glacial till deposit.

Rock Anchors

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress for sound rock of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length of 3 m. Generally, the unconfined compressive strength of limestone bedrock ranges between 60 and 90 MPa, which is stronger than most routine grouts.

A unit weight of 15 kN/m^3 may be considered for the in-situ bedrock. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

A **Rock Mass Rating (RMR) of 65** is considered suitable for the bedrock formation throughout the subject site, and Hoek and Brown parameters (**m and s**) were taken as **0.575** and **0.00293**, respectively. For design purposes, all rock anchors are recommended to be placed at least 1.2 m apart to reduce group anchor effects. The above and additional design parameters are provided for reference below:

Table 10 – Parameters Used for Rock Anchor Design

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Fair Quality Shale Hoek and Brown parameters	44 m=0.575 and s=0.00293
Unconfined compressive strength - Shale bedrock	40 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Typical anchor lengths for a 75- and 125-mm diameter hole are provided in Table 11.

The anchor drill holes should be within 1.5 to 2 times the anchor tendon diameter, inspected by Paterson Geotechnical personnel and flushed clean with water prior to grouting.

A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

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

Table 11 – Typical Rock Anchor Lengths – Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Minimum Bonded Length	Minimum Unbonded Length	Minimum Total Length	
75	1.5	1.0	2.5	300
	2.5	2.0	4.5	500
	4.8	3.0	7.8	1,000
	9.2	4.0	13.5	2,000
125	1.1	1.5	2.6	300
	1.6	1.8	3.4	500
	2.9	2.4	5.3	1,000
	5.6	3.0	8.6	2,000

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

It is suggested that foundation waterproofing and drainage products be provided for the proposed perimeter foundation walls, and that the base of the excavation throughout the portion of the loading area of the structure (FFE equal to an elevation of 61.00 m) be tanked to minimize infiltration of groundwater into the buildings sump system. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by a minimum of 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structures where double-sided pours will be undertaken.

In areas where blind-sided pours will be considered, the perimeter drainage pipe should be placed along the interior side of the foundation wall and connected to sleeves placed within the foundation wall at a spacing advised upon by Paterson. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or building sump pit. 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 the waterproofing system, and provision should be carried for that purpose by the associated contractors accordingly. In a double-sided pour configuration, the exterior side of the foundation wall is expected to be exposed and prepared to install the waterproofing membrane and drainage board system.

It is expected that 150 mm diameter sleeves 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. Reference should be made to the mechanical and plumbing drawings prepared by TMP, complete in coordination with Paterson, depicting the proposed location of the sleeves within the subject site.

Perimeter Foundation System

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by a minimum of 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The perimeter drainage pipe would connect to a series of underfloor drainage lines which would direct water to sump pit(s) within the lower basement area.

Underfloor Drainage System

It is anticipated that underfloor drainage will be required to control water infiltration below the proposed basement level. The layout of the sleeves, perimeter and underfloor drainage systems has been coordinated with The Mitchell Partnership Inc. (TMP). Reference should be made to the mechanical and plumbing drawings prepared by TMP.

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 CCW MiraDrain 2000 or Delta-Teraxx or equivalent other reviewed and approved by Paterson, connected to the perimeter foundation drainage system. Placement of the material is recommended to be undertaken in accordance with the recommendations provided in Section 5.2. Imported granular materials, such as clean sand, OPSS Granular B Type I granular material or site-generated clean sand should otherwise be used for this purpose.

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. This requirement should be advised by Paterson during the design phase and based on review of architectural, structural and civil design drawings.

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. However, for most of the site, insufficient room will be available to permit the building excavation to be constructed by open-cut methods (i.e., unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back to 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 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.

Excavation side slopes around the building excavation should be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes, or other means of erosion protection along their footprint. The tarps should be anchored with stakes embedded a minimum of 600 mm below existing grade at the top of the excavation and on a maximum spacing of 3 m centres.

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 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. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included in the earth pressures described below.

Due to the non-cohesive nature of the in-situ soils, it is recommended that a rigid closed cell system, such as secant and/or sheet piles, be considered where the system will retain soils supporting settlement sensitive structures and/or infrastructure. Sheet pile embedment is expected to be limited by the dense nature of the underlying glacial till deposit and boulder content.

The remainder of the system may consist of a soldier pile and timber lagging system. The implementation of a soldier pile and lagging system is not recommended to be undertaken in excavations extending below the groundwater table due to the presence of running sand and overburden that can slough into the open excavation during installation. Management of groundwater will be critical in implementing a soldier pile and timber lagging system due to sandy nature of the in-situ subsoils. If it is sought to use the sidewalls as a cut-off from groundwater influx into the excavation, a cut off wall will be required to be implemented, and a soldier pile and timber lagging system would not suffice in this scenario.

Shoring designs should be planned to ensure adequate contact between lagging and retained soils is provided to minimize sloughing and disturbance of retained soils resulting in a void that would form without adequate lagging-overburden contact. Further, lift heights and bay widths of the excavation supported by a timber lagging and soldier pile system should be planned to consider the non-cohesive and loose nature of the in-situ fill and sandy subsoils.

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 12 - 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

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 through review of existing as-built drawings and historical reports available for all structures adjacent to the proposed structure. The requirement to temporarily support these structures using concrete underpinning or temporary shoring may be evaluated at that time. 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. Underpinning efforts should be undertaken in the dry and with drained subsoils given the sandy nature of the in-situ overburden.

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 and to undertake foundation construction works in the dry. Infiltration levels are anticipated to be high through the excavation for areas where sewers and foundations are located below the groundwater table level.

A hydrogeological assessment of the proposed redevelopment has been prepared by Paterson under a separate cover which quantifies the volume of water and rate of influx anticipated to be handled during the construction phase. Reference should be made to Paterson Hydrogeological Report PH5000-1 dated November 22, 2024.

Groundwater levels throughout the subject site have historically risen and lowered proportionally to the water level in the Rideau Canal. This may be observed in *Figure 4 – Groundwater Elevation Monitoring – Program Update* provided in Appendix 2 of this report which depicts the fluctuations in the water levels measured in monitoring wells located throughout the subject site and the overall Lansdowne Redevelopment Project area.

It is recommended that a specialized dewatering contractor be retained by the earthworks contractor for all excavations anticipated to be undertaken below the groundwater table. Dewatering methods advised by the specialist, such as well points, may be required for areas where excavations will advance below the groundwater table. Reference should be made to the aforementioned hydrogeological report to ascertain volumes and hydraulic conductivity of the in-situ soils as part of planning the associated dewatering and sewer and building excavation programs.

Permit to Take Water

A Ministry of the Environment, Conservation and Parks (MECP) Category 3 permit to take water (PTTW) is currently being prepared by Paterson in the event that construction activities take place during the seasonally high-water table. 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 – Temporary Construction Conditions

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

Long-term Groundwater Control

Based on our review, the founding elevation of the proposed structure will be such no issues are expected with respect to groundwater lowering that would cause long-term adverse effects to adjacent structures surrounding the proposed building, including the Rideau Canal.

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. Fill imported to the subject site and used to build up the subgrade below settlement sensitive structures, such as basement slabs and exterior paved areas, must be free of frost and cannot be exposed to freezing conditions during the construction phase. It will otherwise be susceptible to excessive post-thawing settlement that would require remedial efforts to resolve.

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 to 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.

These precautions would be required to be taken where excavation of side slopes is undertaken in close proximity to existing structures and substructures. 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 indicate 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 aggressive to very aggressive corrosive environment.

6.8 Landscaping Considerations

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.

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 building footprint, including the elevator shaft, 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.
- 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.

Fernanda Carozzi, PhD. Geoph.

Drew Petahtegoose, P.Eng.

LICENSED PROFESSIONAL ENGINEER
April 3, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

PHOTOGRAPHS OF ROCK CORE

GRAIN SIZE DISTRIBUTION TESTING RESULTS

UNCONFINED COMPRESSIVE STRENGTH TESTING RESULTS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY TESTING RESULTS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368752.26 **NORTHING:** 5029083.13 **ELEVATION:** 66.33

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: October 15, 2024

FILE NO. : PG6655

HOLE NO. : BH 3-24

SAMPLE DESCRIPTION

GROUND SURFACE

ASPHALT

0.06m [66.27m]

FILL: Brown silty sand, with crushed stone and gravel

- Grey by 0.7 m depth

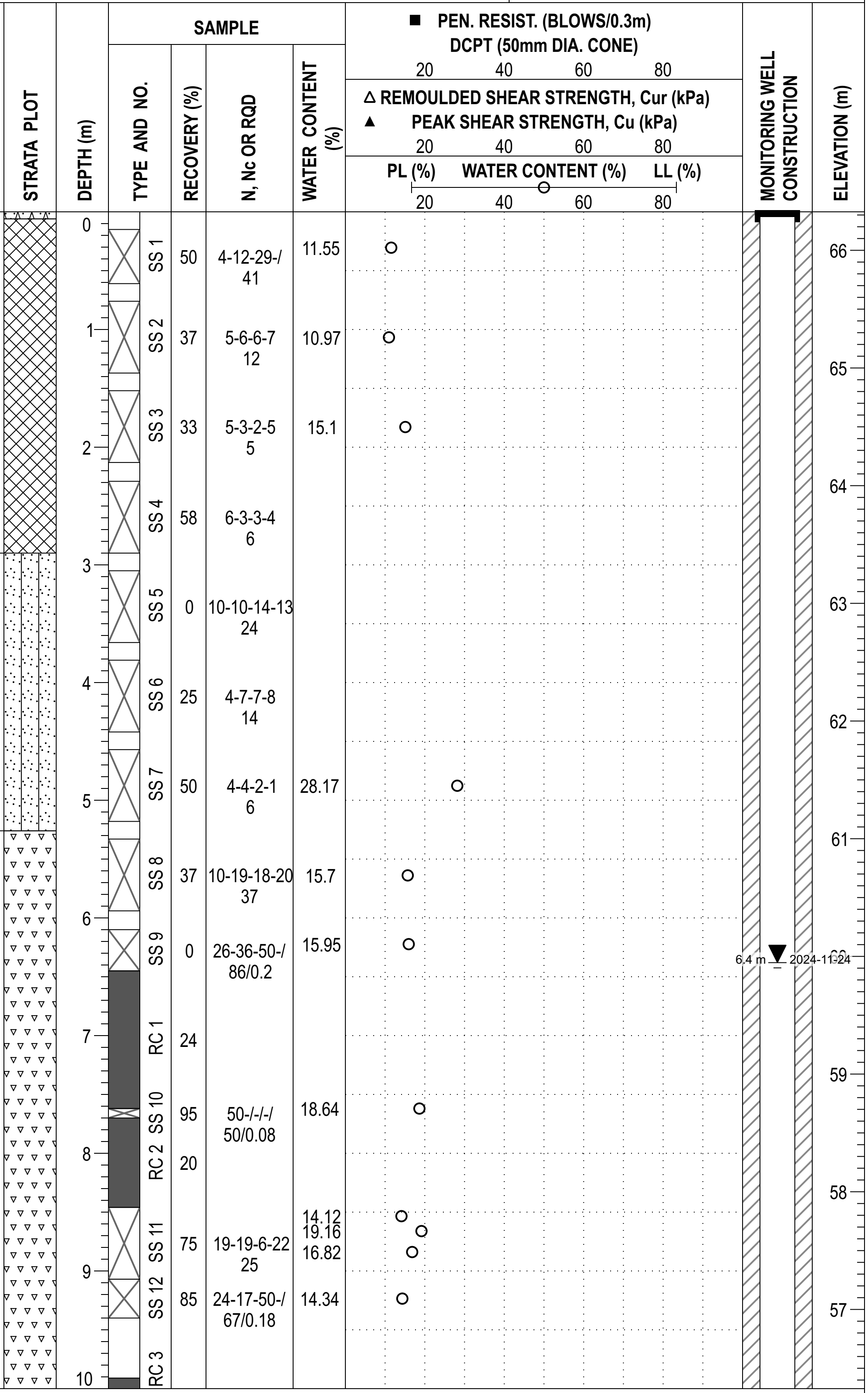
2.90m [63.43m]

Compact, brown **SILTY SAND**, trace gravel

5.26m [61.07m]

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

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	50	4-12-29-/ 41	11.55
SS 2	37	5-6-6-7 12	10.97
SS 3	33	5-3-2-5 5	15.1
SS 4	58	6-3-3-4 6	
SS 5	0	10-10-14-13 24	
SS 6	25	4-7-7-8 14	
SS 7	50	4-4-2-1 6	28.17
SS 8	37	10-19-18-20 37	15.7
SS 9	0	26-36-50-/ 86/0.2	15.95
RC 1	24		
SS 10	95	50-/-/-/ 50/0.08	18.64
RC 2	20		
SS 11	75	19-19-6-22 25	14.12 19.16 16.82
SS 12	85	24-17-50-/ 67/0.18	14.34
RC 3			



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368752.26 **NORTHING:** 5029083.13 **ELEVATION:** 66.33

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: October 15, 2024

FILE NO. : PG6655

HOLE NO. : BH 3-24

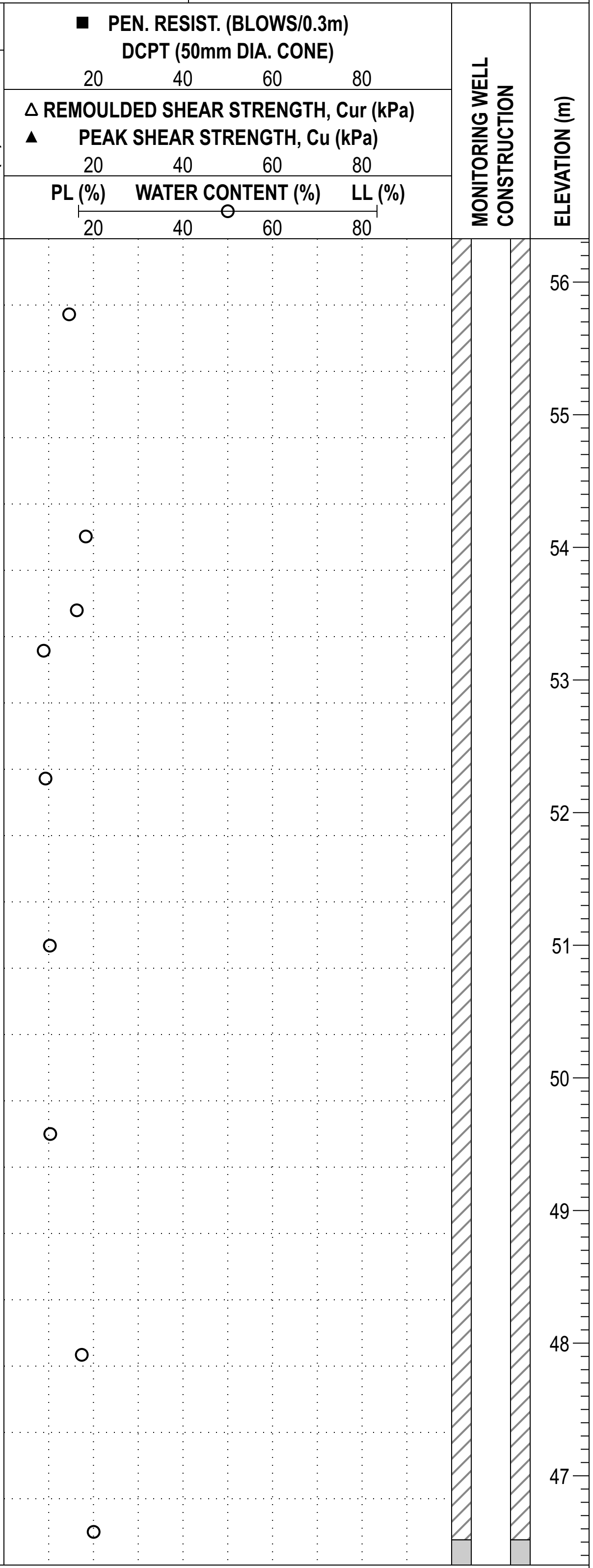
SAMPLE DESCRIPTION

GROUND SURFACE

- Grey by 12.2 m depth
- Trace clay by 13.9 m depth
- Silt content increasing with depth

SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
RC 3	75		
SS 13	102	50-/-/-/ 50/0.1	14.57
RC 4	2		
SS 14		50-/-/-/ 50/0.05	18.29
RC 5	51		
SS 15	50	20-15-7-8 22	16.28 8.88
SS 16	76	10-12-50-/ 62/0.25	9.29
RC 6	28		
SS 17	100	51-50-/-/ 50/0.13	10.26
RC 7	49		
SS 18	100	41-50-/-/ 50/0.13	10.35
RC 8	29		
SS 19	100	6-15-28-50 43	17.39
RC 9	20		
SS 20	54	48-50-/-/ 50/0.13	20.04

DEPTH (m): 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20



SOIL PROFILE AND TEST DATA

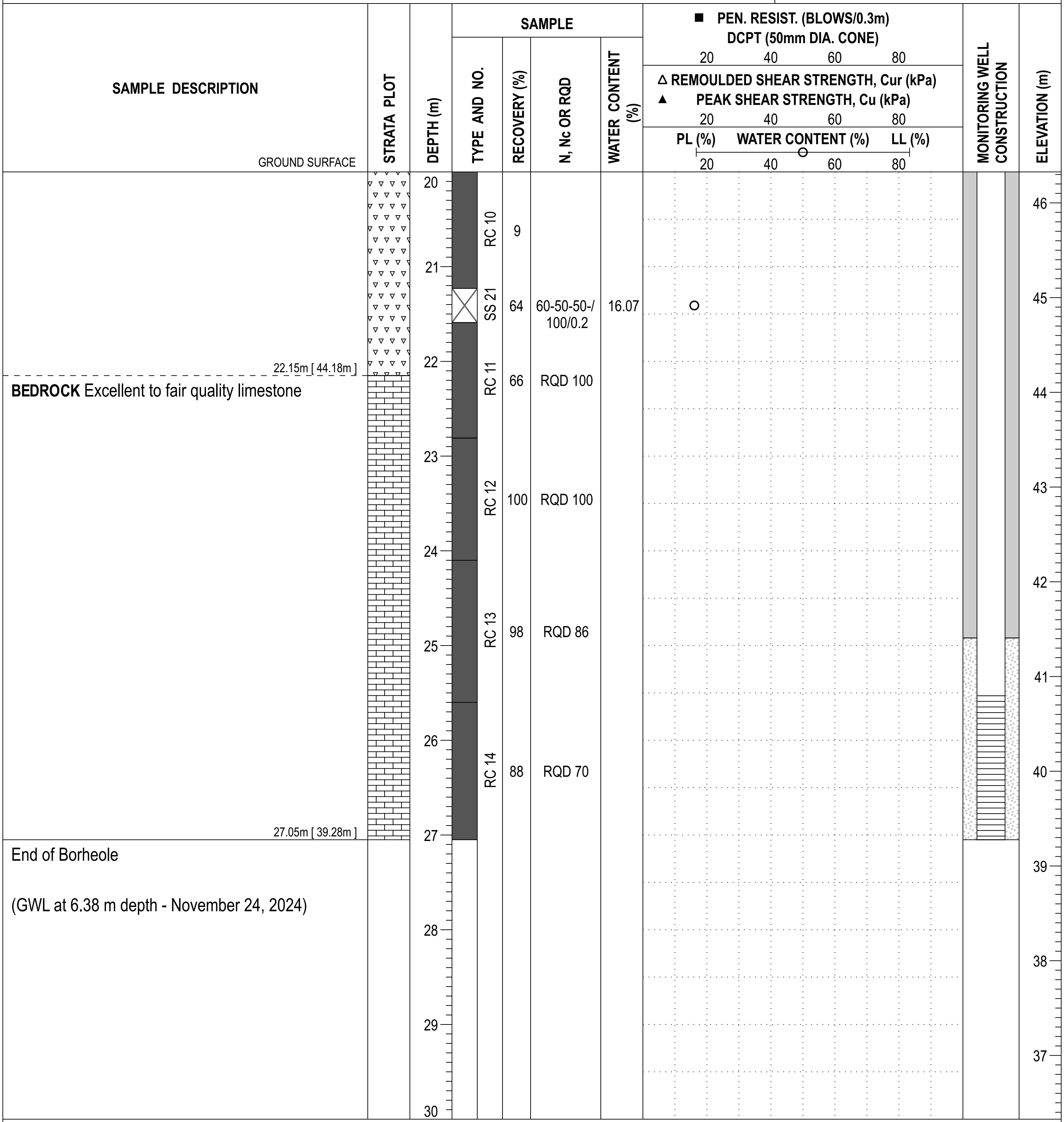
Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368752.26 **NORTHING:** 5029083.13 **ELEVATION:** 66.33

PROJECT: Proposed North Stands
BORINGS BY: CME-55 Low Clearance Drill
REMARKS:
DATE: October 15, 2024

FILE NO. : PG6655
HOLE NO. : BH 3-24



22.15m [44.18m]

BEDROCK Excellent to fair quality limestone

27.05m [39.28m]

End of Borheole

(GWL at 6.38 m depth - November 24, 2024)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
RC 10	9		
SS 21	64	60-50-50-/ 100/0.2	16.07
RC 11	66	RQD 100	
RC 12	100	RQD 100	
RC 13	98	RQD 86	
RC 14	88	RQD 70	

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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368752.30 **NORTHING:** 5029083.00 **ELEVATION:** 66.33

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: October 16, 2024

FILE NO. : PG6655

HOLE NO. : BH 3A-24

SAMPLE DESCRIPTION

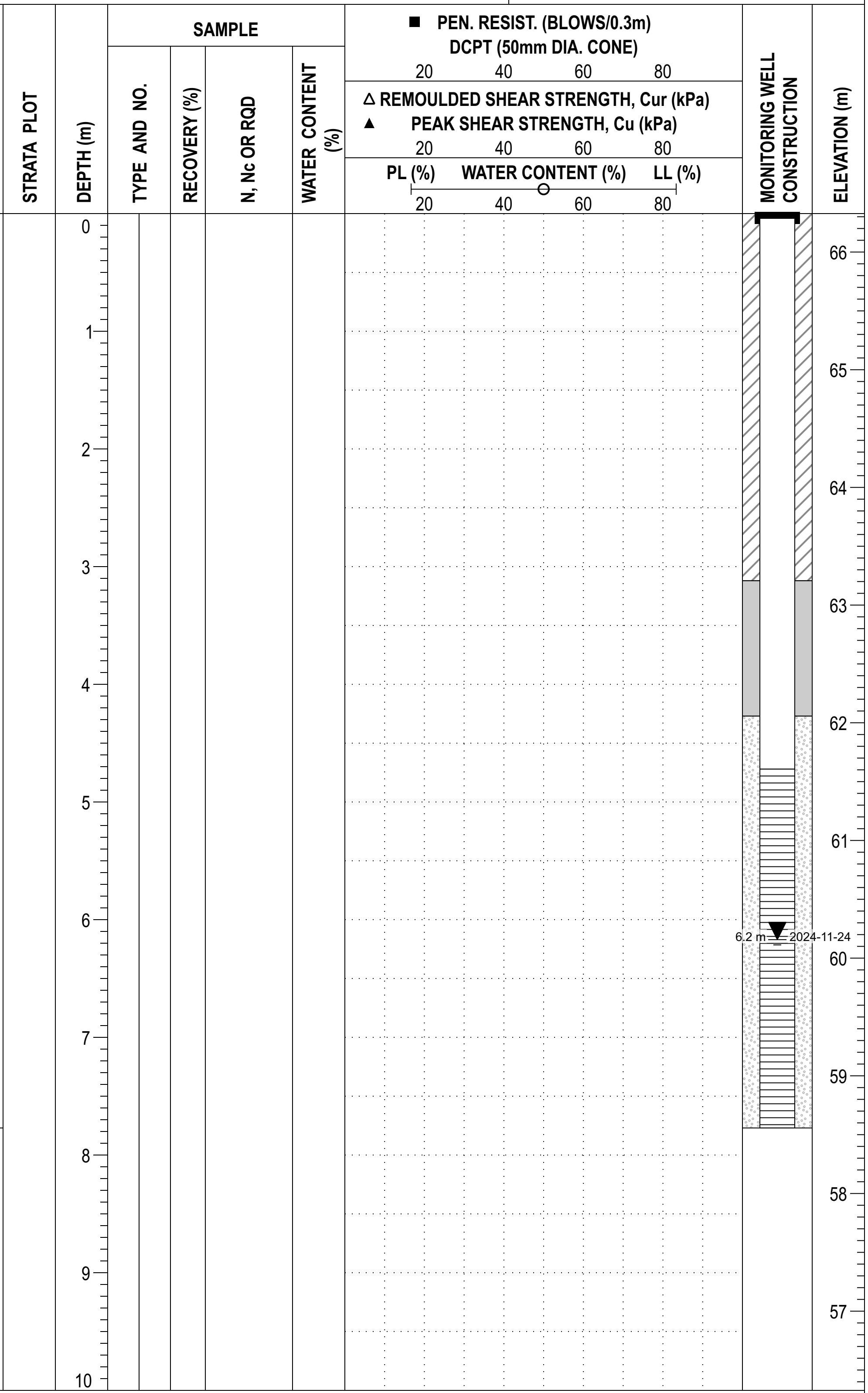
GROUND SURFACE

Refer to BH 3-24 for soil profile

7.77m [58.56m]

End of Borehole

(GWL at 6.17 m depth - November 24, 2024



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368655.15 **NORTHING:** 5029026.59 **ELEVATION:** 66.18

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: October 18, 2024

FILE NO. : **PG6655**

HOLE NO. : **BH 4-24**

SAMPLE DESCRIPTION

GROUND SURFACE

ASPHALT 0.06m [66.12m]

FILL: Brown silty sand, with gravel

- Thin asphalt layer by 0.7 m depth

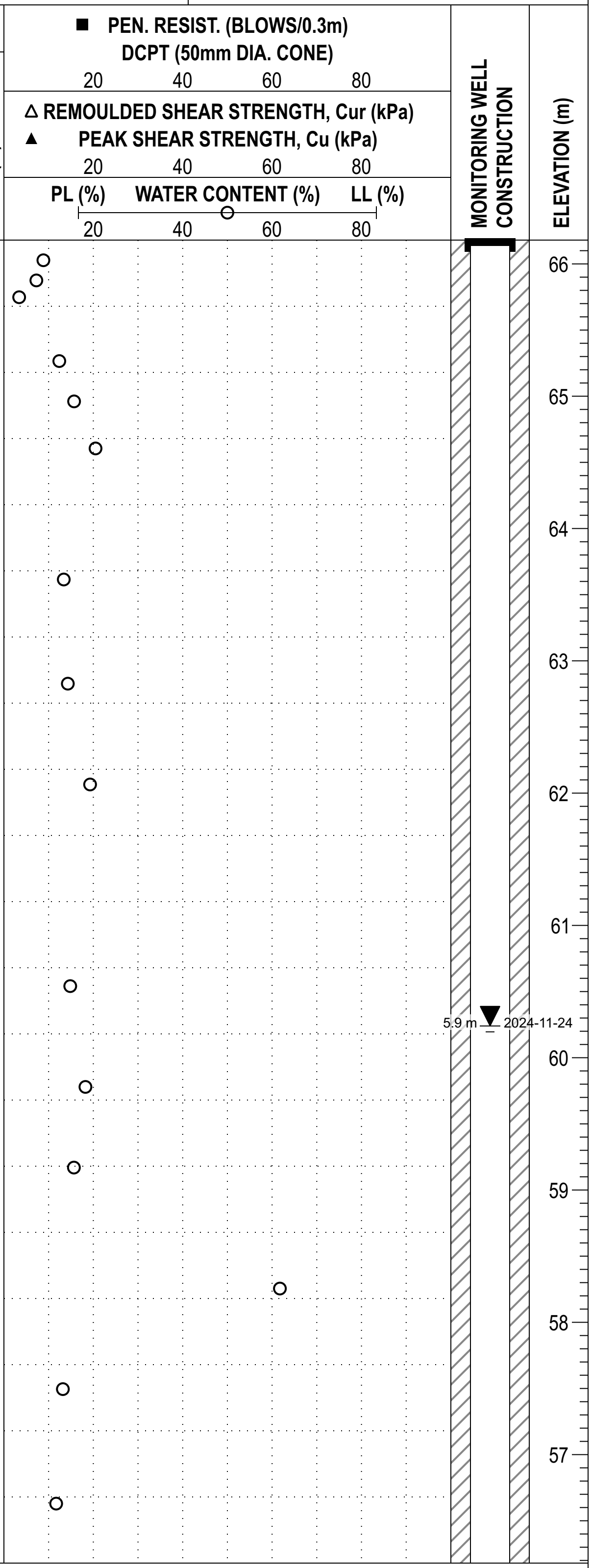
1.73m [64.45m]

Dense, brown **SILTY SAND**, with gravel, cobbles and boulders

5.26m [60.92m]

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

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	67	14-21-27-29 48	8.83 7.26 3.41
SS 2	66	21-11-10-5 21	12.39 15.7
SS 3	97	10-50-/-/ 50/0.05	20.5
RC 1	69		
SS 4	67	15-20-17-20 37	13.39
SS 5	58	16-16-17-18 33	14.3
SS 6	58	11-12-17-21 29	19.28
SS 7	58	19-18-17-23 35	
SS 8	67	22-18-16-16 34	14.84
SS 9	75	18-19-19-11 38	18.24
SS 10	102	42-50-/-/ 50/0.05	15.66
RC 2	69		
SS 11	83	38-33-38-44 71	61.75
SS 12	58	33-37-73-51 110	13.19
SS 13	75	47-40-42-50 82	11.69
RC 3			



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368655.15 **NORTHING:** 5029026.59 **ELEVATION:** 66.18

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

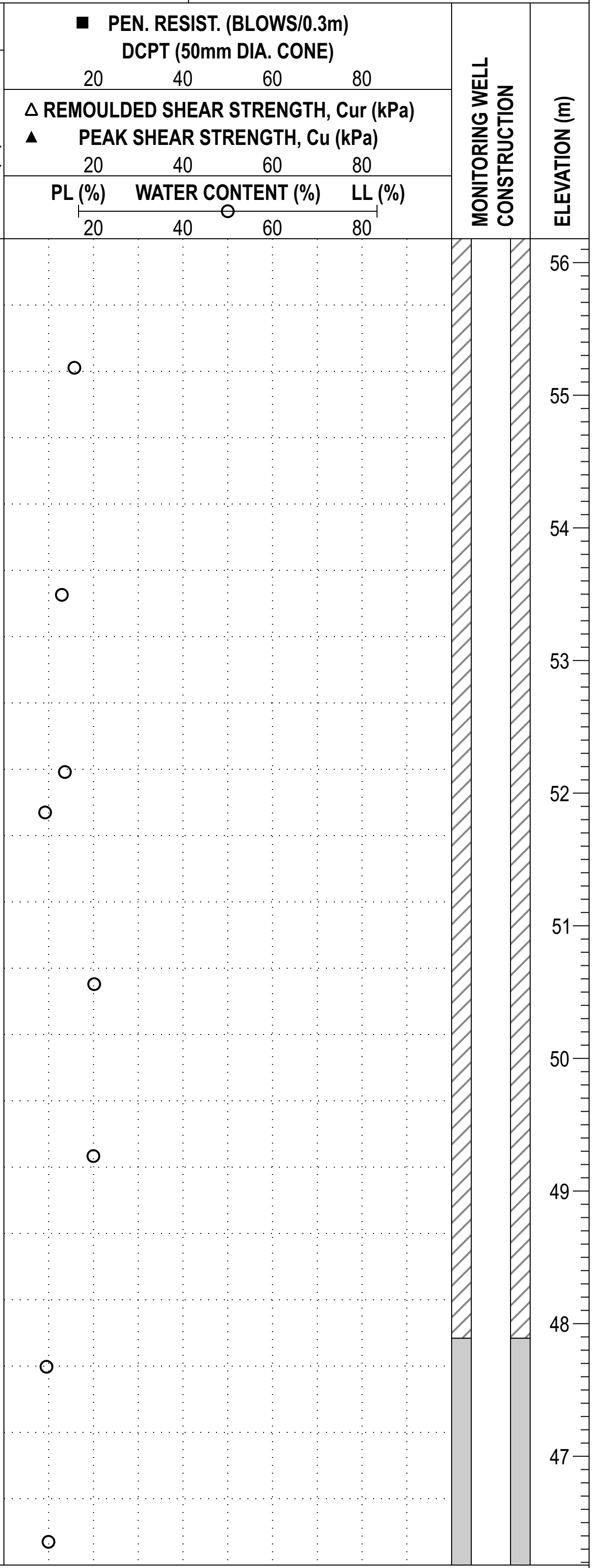
REMARKS:

DATE: October 18, 2024

FILE NO. : PG6655

HOLE NO. : BH 4-24

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE					
	10	RC 3	46		
	11	SS 14	100	89-50-/-/ 50/0.08	15.74
		RC 4	35		
	12				
		SS 15		40-46-38-49 84	12.94
	13	RC 5	21		
	14				13.62
		SS 16	83	24-27-29-37 56	9.2
	15	RC 6	0		
		SS 17	100	7-44-50-/ 94/0.1	20.16
	16				
	17	SS 18	92	27-28-23-22 51	19.98
- Grey by 17.7 m depth		RC 7	22		
	18	SS 19	100	28-39-50-/ 89/0.28	9.51
	19				
	20	SS 20	67	38-40-60-50 100	9.95



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368655.15 **NORTHING:** 5029026.59 **ELEVATION:** 66.18

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

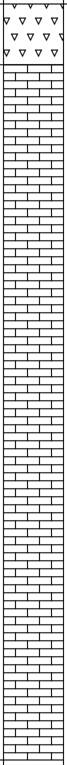
DATE: October 18, 2024

FILE NO. : PG6655

HOLE NO. : BH 4-24

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE	20	SS 20				46
20.45m [45.73m]						
BEDROCK Excellent to good quality limestone	21	RC 8	80	RQD 94		45
	22	RC 9	100	RQD 85		44
	23	RC 10	100	RQD 100		43
	24					42
25.60m [40.58m]	25	RC 11	100	RQD 98		41
End of Borehole	26					40
(GWL at 5.94 m depth - November 24, 2024)	27					39
	28					38
	29					37
	30					

STRATA PLOT

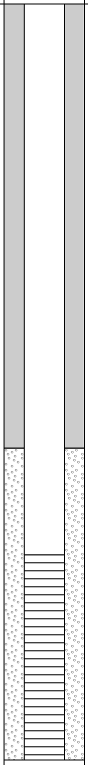


■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
20 40 60 80

△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

PL (%) WATER CONTENT (%) LL (%)
20 40 60 80

MONITORING WELL CONSTRUCTION



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 368654.17 **NORTHING:** 5029070.94 **ELEVATION:** 62.54

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: Borehole Drilled Indoors

DATE: October 21, 2024

FILE NO. : PG6655

HOLE NO. : BH 5-24

SAMPLE DESCRIPTION

GROUND SURFACE

CONCRETE SLAB

0.20m [62.34m]

FILL: Crushed stone, layer of insulation

0.41m [62.13m]

Compact, brown **SILTY SAND**, trace gravel

1.68m [60.86m]

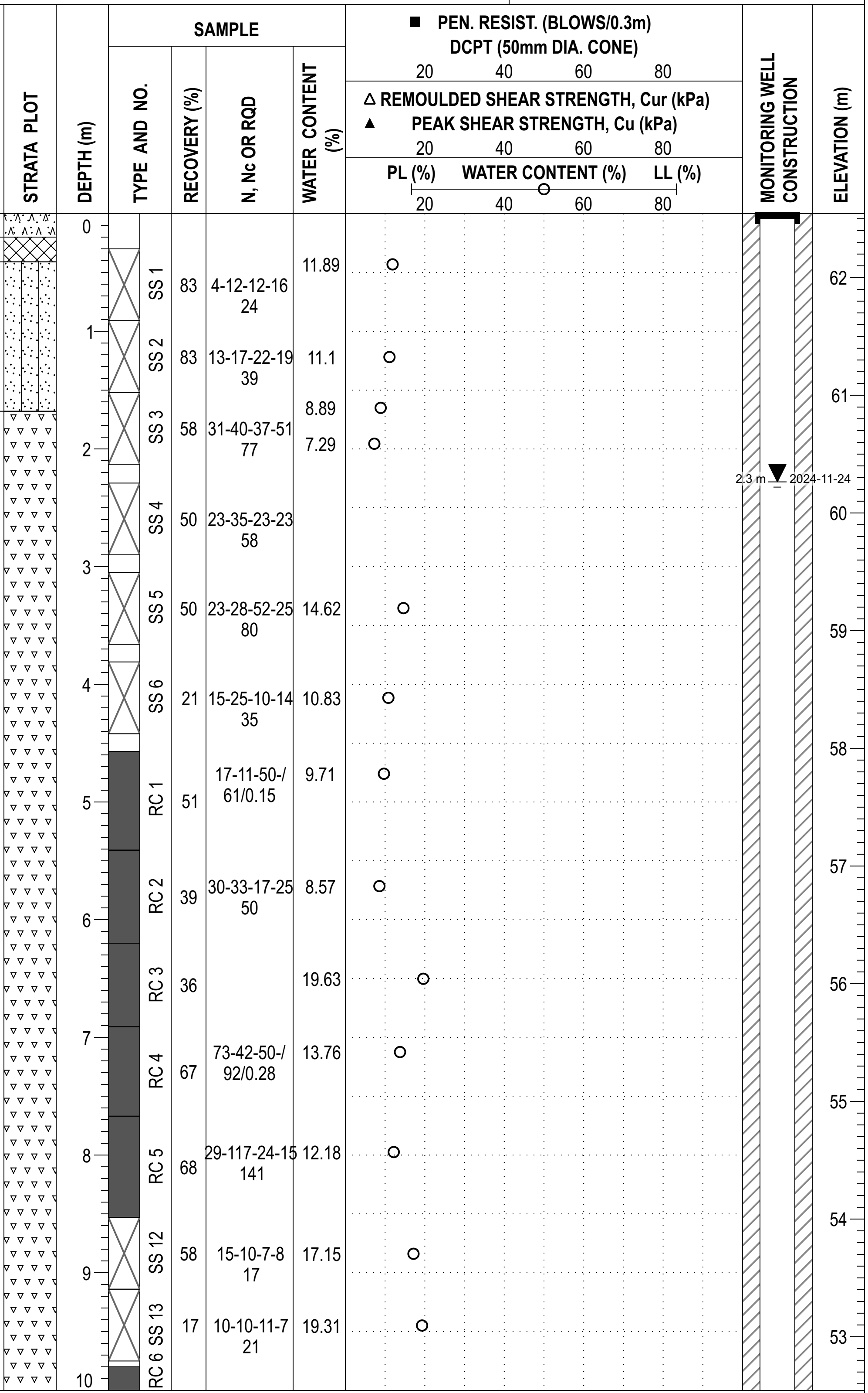
GLACIAL TILL: Very dense, brown silty sand, with gravel, cobbles and boulders

- Grey by 3.0 m depth

- Trace clay by 3.7 m depth

- Compact by 8.5 m depth

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	83	4-12-12-16 24	11.89
SS 2	83	13-17-22-19 39	11.1
SS 3	58	31-40-37-51 77	8.89 7.29
SS 4	50	23-35-23-23 58	
SS 5	50	23-28-52-25 80	14.62
SS 6	21	15-25-10-14 35	10.83
RC 1	51	17-11-50-/ 61/0.15	9.71
RC 2	39	30-33-17-25 50	8.57
RC 3	36		19.63
RC 4	67	73-42-50-/ 92/0.28	13.76
RC 5	68	29-117-24-15 141	12.18
SS 12	58	15-10-7-8 17	17.15
SS 13	17	10-10-11-7 21	19.31
RC 6			



COORD. SYS.: MTM ZONE 9 **EASTING:** 368654.17 **NORTHING:** 5029070.94 **ELEVATION:** 62.54

PROJECT: Proposed North Stands
BORINGS BY: CME-55 Low Clearance Drill
REMARKS: Borehole Drilled Indoors
DATE: October 21, 2024

FILE NO. : PG6655
HOLE NO. : BH 5-24

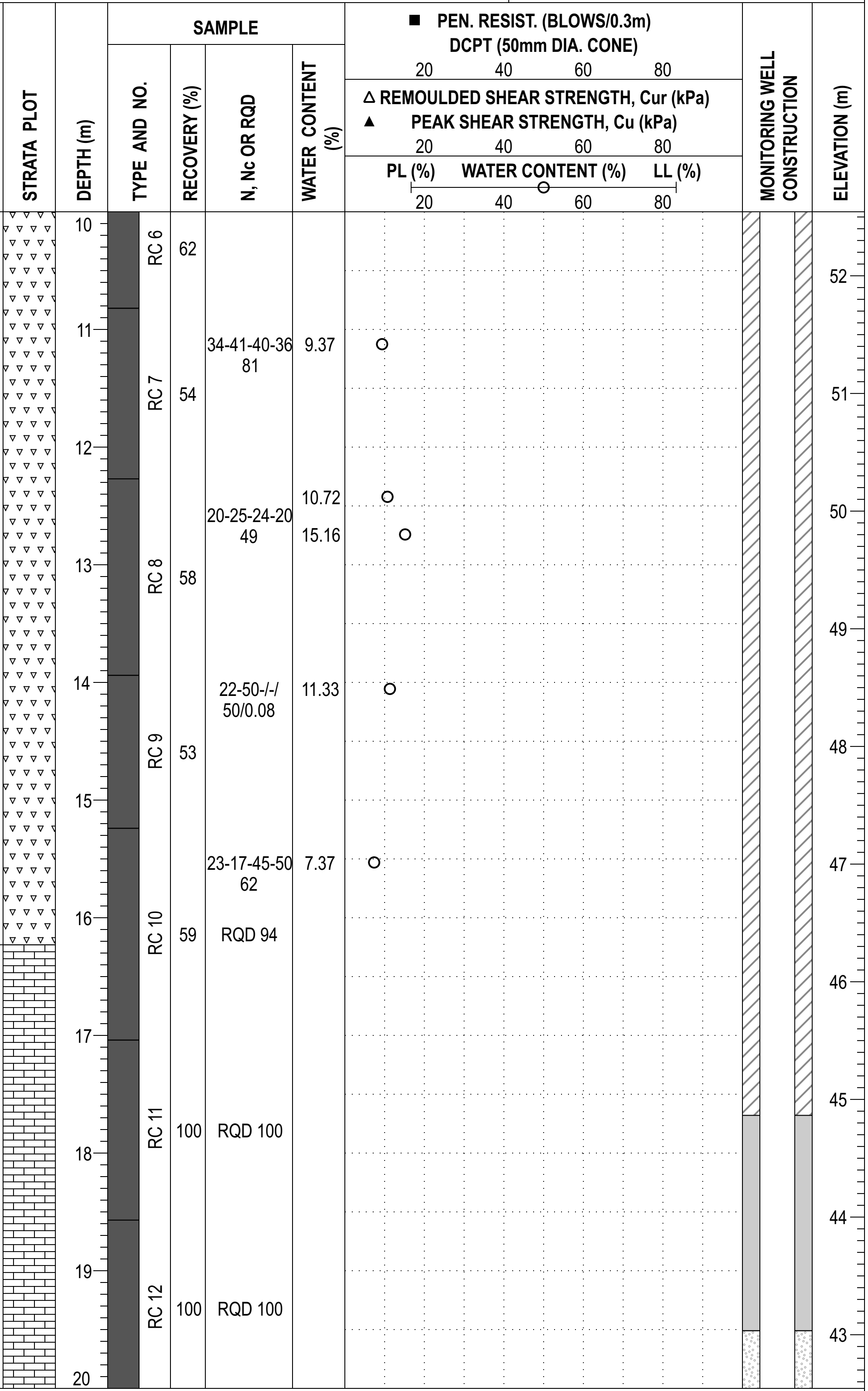
SAMPLE DESCRIPTION

GROUND SURFACE

16.23m [46.31m]

BEDROCK: Good to excellent quality limestone

SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
RC 6	62		
		34-41-40-36 81	9.37
RC 7	54		
		20-25-24-20 49	10.72 15.16
RC 8	58		
		22-50-/-/ 50/0.08	11.33
RC 9	53		
		23-17-45-50 62	7.37
RC 10	59	RQD 94	
RC 11	100	RQD 100	
RC 12	100	RQD 100	



COORD. SYS.: MTM ZONE 9 **EASTING:** 368654.17 **NORTHING:** 5029070.94 **ELEVATION:** 62.54

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: Borehole Drilled Indoors

DATE: October 21, 2024

FILE NO. : PG6655

HOLE NO. : BH 5-24

SAMPLE DESCRIPTION GROUND SURFACE	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH, Cur (kPa) ▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
		20	RC 12						42
21.56m [40.98m]		21	RC 13	100	RQD 97				41
End of Borheole (GWL at 2.28 m depth - November 24, 2024)		22							40
		23							39
		24							38
		25							37
		26							36
		27							35
		28							34
		29							33
		30							

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368654.17 **NORTHING:** 5029070.94 **ELEVATION:** 62.54

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

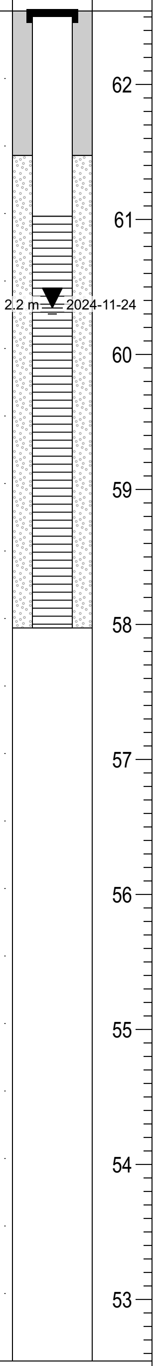
REMARKS: Borehole Drilled Indoors

DATE: October 22, 2024

FILE NO. : PG6655

HOLE NO. : BH 5A-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N, Nc OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH, Cur (kPa) ▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
Refer to BH 5-24 for soil profile 4.57m [57.97m]		0 – 4.57							62.54 – 57.97
End of Borehole (GWL at 2.2 m depth - November 24, 2024)									



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368726.10 **NORTHING:** 5029111.91 **ELEVATION:** 62.49

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: Borehole Drilled Indoors

DATE: October 28, 2024

FILE NO. : **PG6655**

HOLE NO. : **BH 6-24**

SAMPLE DESCRIPTION

GROUND SURFACE

CONCRETE SLAB 0.10m [62.39m]

FILL: Crushed stone 0.23m [62.26m]

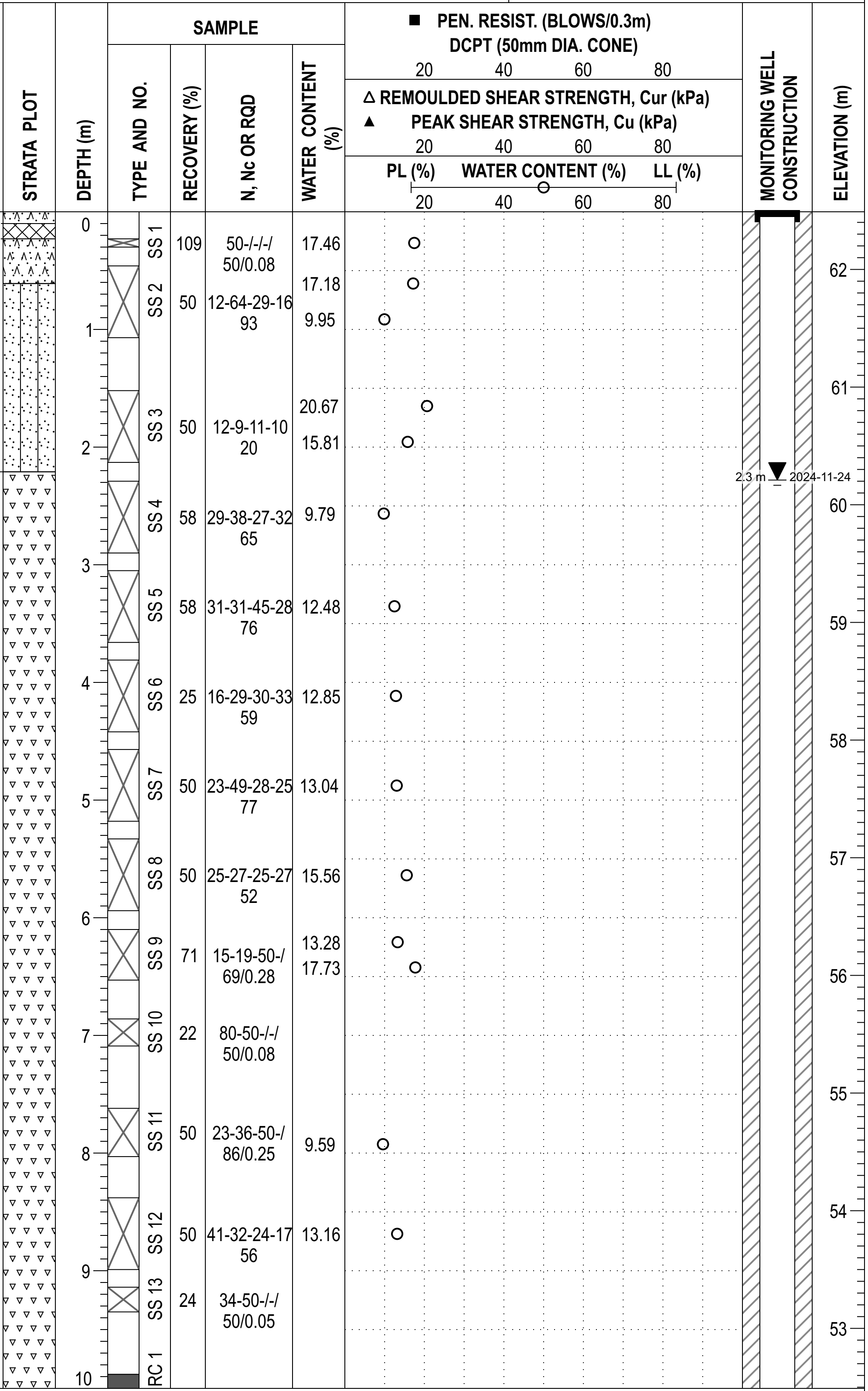
REINFORCED CONCRETE 0.61m [61.88m]

Very dense to compact, brown **SILTY SAND**, trace gravel 2.21m [60.28m]

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

- Grey by 6.0 m depth

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	109	50-/-/-/ 50/0.08	17.46
SS 2	50	12-64-29-16 93	17.18 9.95
SS 3	50	12-9-11-10 20	20.67 15.81
SS 4	58	29-38-27-32 65	9.79
SS 5	58	31-31-45-28 76	12.48
SS 6	25	16-29-30-33 59	12.85
SS 7	50	23-49-28-25 77	13.04
SS 8	50	25-27-25-27 52	15.56
SS 9	71	15-19-50-/ 69/0.28	13.28 17.73
SS 10	22	80-50-/-/ 50/0.08	
SS 11	50	23-36-50-/ 86/0.25	9.59
SS 12	50	41-32-24-17 56	13.16
SS 13	24	34-50-/-/ 50/0.05	
RC 1			



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368726.10 **NORTHING:** 5029111.91 **ELEVATION:** 62.49

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: Borehole Drilled Indoors

DATE: October 28, 2024

FILE NO. : PG6655

HOLE NO. : BH 6-24

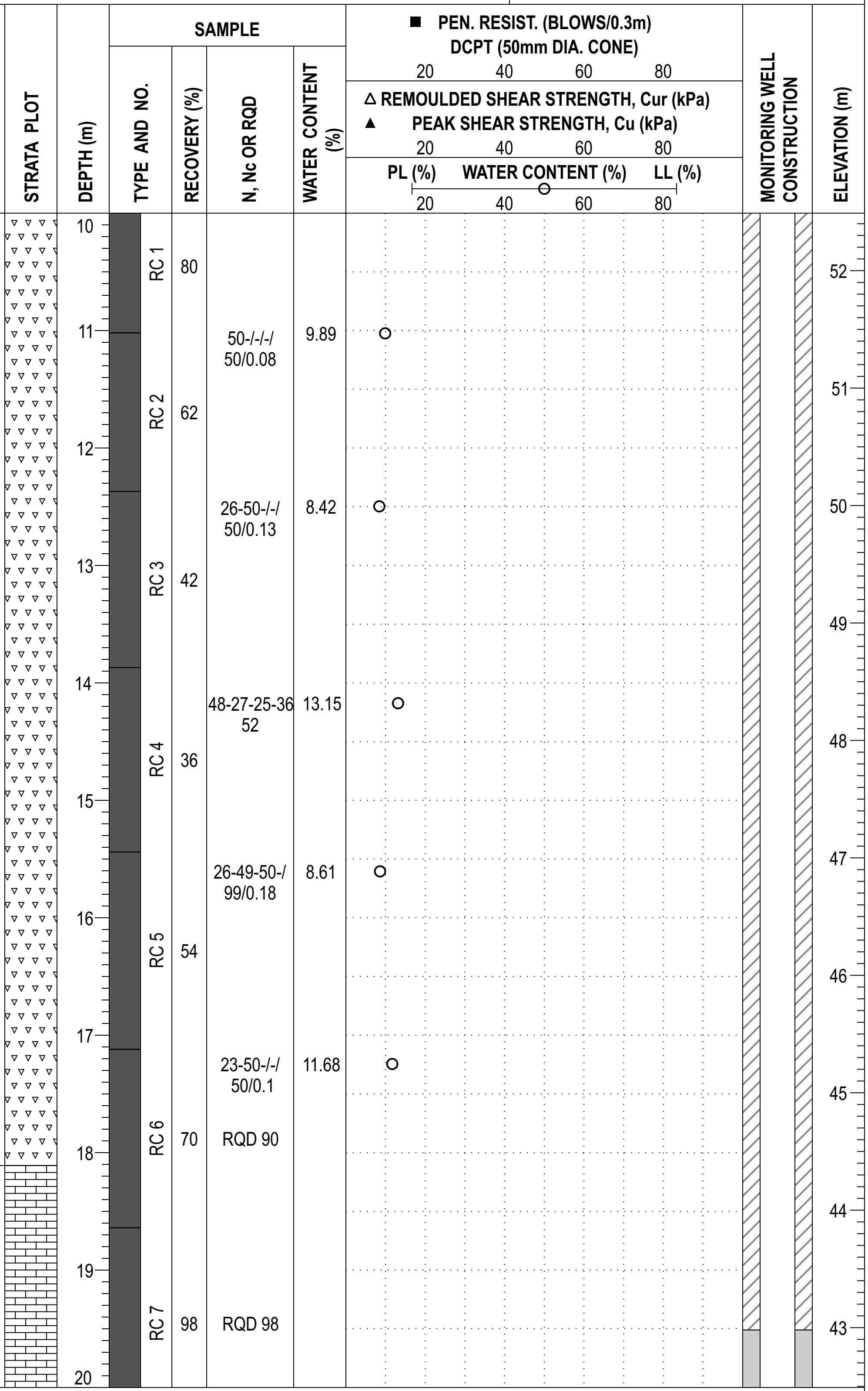
SAMPLE DESCRIPTION

GROUND SURFACE

- Trace clay by 11.0 m depth

18.11m [44.38m]

BEDROCK: Excellent quality limestone



TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
RC 1	80		
		50-/-/-/ 50/0.08	9.89
RC 2	62		
		26-50-/-/ 50/0.13	8.42
RC 3	42		
		48-27-25-36 52	13.15
RC 4	36		
		26-49-50-/ 99/0.18	8.61
RC 5	54		
		23-50-/-/ 50/0.1	11.68
RC 6	70	RQD 90	
RC 7	98	RQD 98	

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368726.10 **NORTHING:** 5029111.91 **ELEVATION:** 62.49

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: Borehole Drilled Indoors

DATE: October 28, 2024

FILE NO. : PG6655

HOLE NO. : BH 6-24

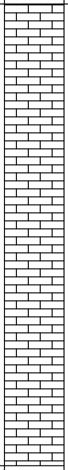
■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80

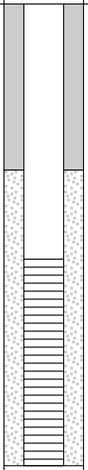
PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE						
	20	RC 7				42
	21	RC 8	100	RQD 95		41
	22	RC 9	100	RQD 84		40
23.42m [39.07m]	23					39
End of Borehole (GWL at 2.28 m depth - November 24, 2024)	24					38
	25					37
	26					36
	27					35
	28					34
	29					33
	30					

STRATA PLOT



MONITORING WELL CONSTRUCTION



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368726.10 **NORTHING:** 5029111.91 **ELEVATION:** 62.49

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: Borehole Drilled Indoors

DATE: October 28, 2024

FILE NO. : **PG6655**

HOLE NO. : **BH 6A-24**

SAMPLE DESCRIPTION

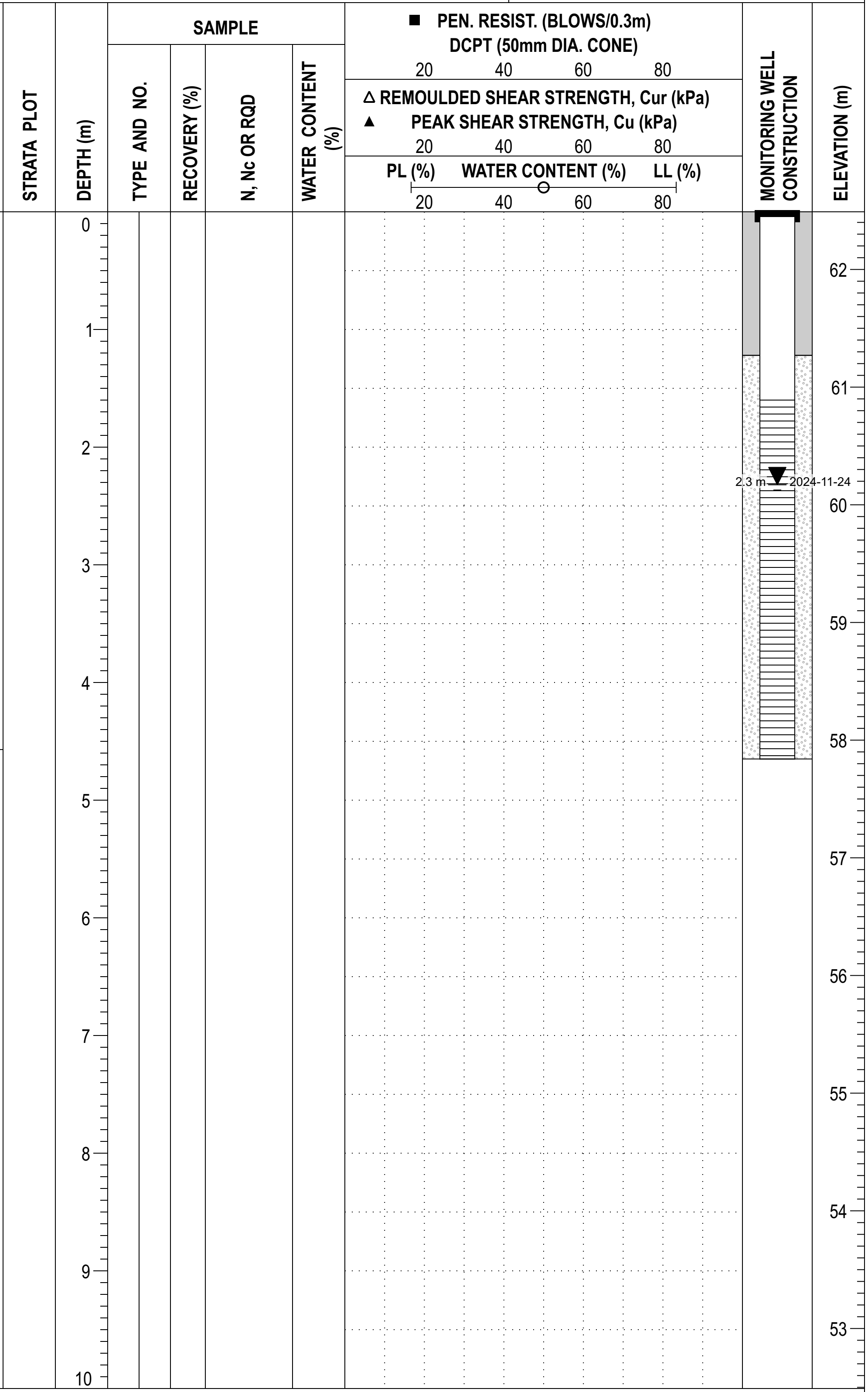
GROUND SURFACE

Refer to BH 6-24 for soil profile

4.57m [57.92m]

End of Borehole

(GWL at 2.32 m depth - November 24, 2024)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

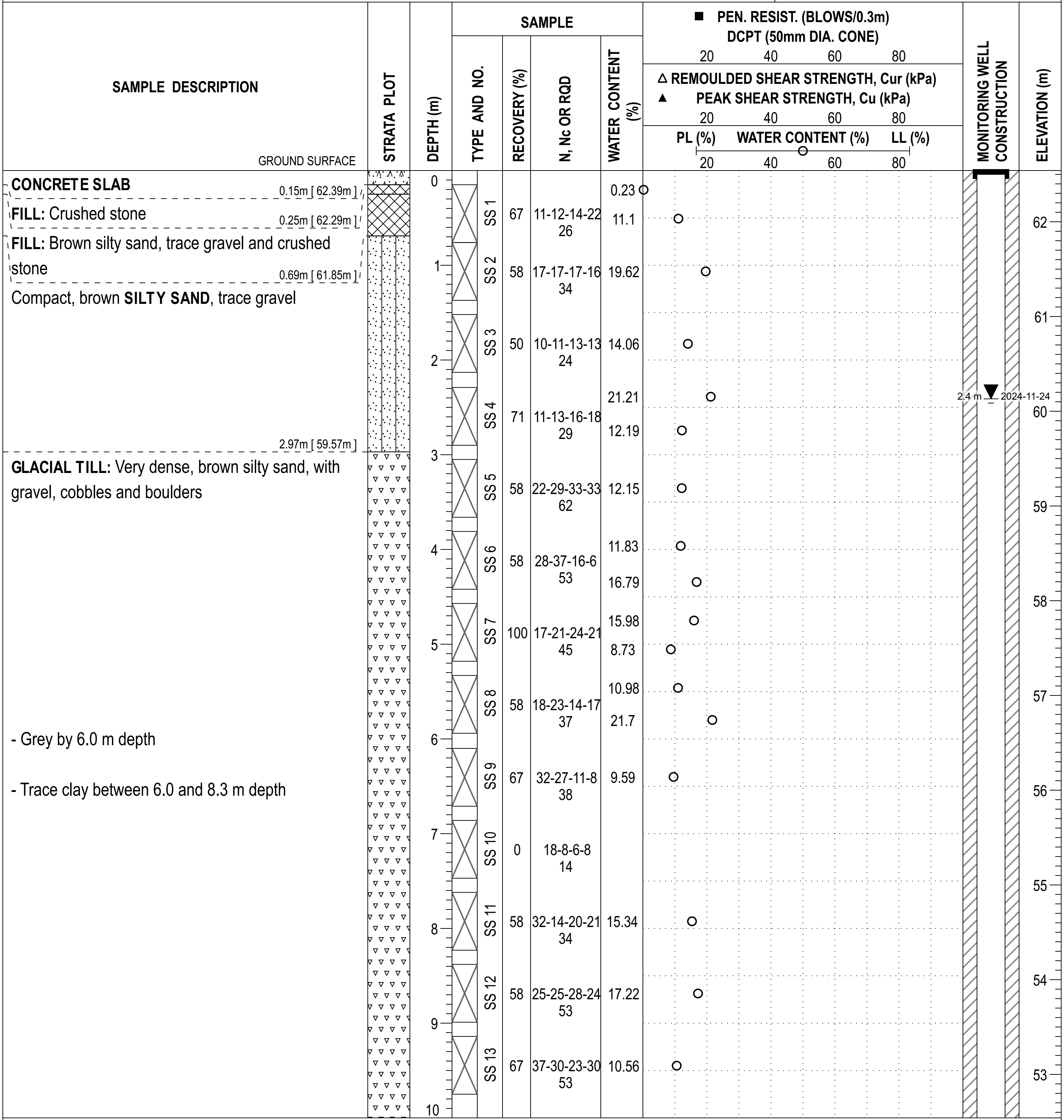
Lansdowne Park Redevelopment

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 368708.35 **NORTHING:** 5029132.04 **ELEVATION:** 62.54

PROJECT: Proposed North Stands
BORINGS BY: CME-55 Low Clearance Drill
REMARKS: Borehole Drilled Indoors
DATE: October 31, 2024

FILE NO. : PG6655
HOLE NO. : BH 7-24



SAMPLE DESCRIPTION

GROUND SURFACE

CONCRETE SLAB 0.15m [62.39m]

FILL: Crushed stone 0.25m [62.29m]

FILL: Brown silty sand, trace gravel and crushed stone 0.69m [61.85m]

Compact, brown **SILTY SAND**, trace gravel 2.97m [59.57m]

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

- Grey by 6.0 m depth

- Trace clay between 6.0 and 8.3 m depth

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
			0.23
SS 1	67	11-12-14-22 26	11.1
SS 2	58	17-17-17-16 34	19.62
SS 3	50	10-11-13-13 24	14.06
SS 4	71	11-13-16-18 29	21.21 12.19
SS 5	58	22-29-33-33 62	12.15
SS 6	58	28-37-16-6 53	11.83 16.79
SS 7	100	17-21-24-21 45	15.98 8.73
SS 8	58	18-23-14-17 37	10.98 21.7
SS 9	67	32-27-11-8 38	9.59
SS 10	0	18-8-6-8 14	
SS 11	58	32-14-20-21 34	15.34
SS 12	58	25-25-28-24 53	17.22
SS 13	67	37-30-23-30 53	10.56

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368708.35 **NORTHING:** 5029132.04 **ELEVATION:** 62.54

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

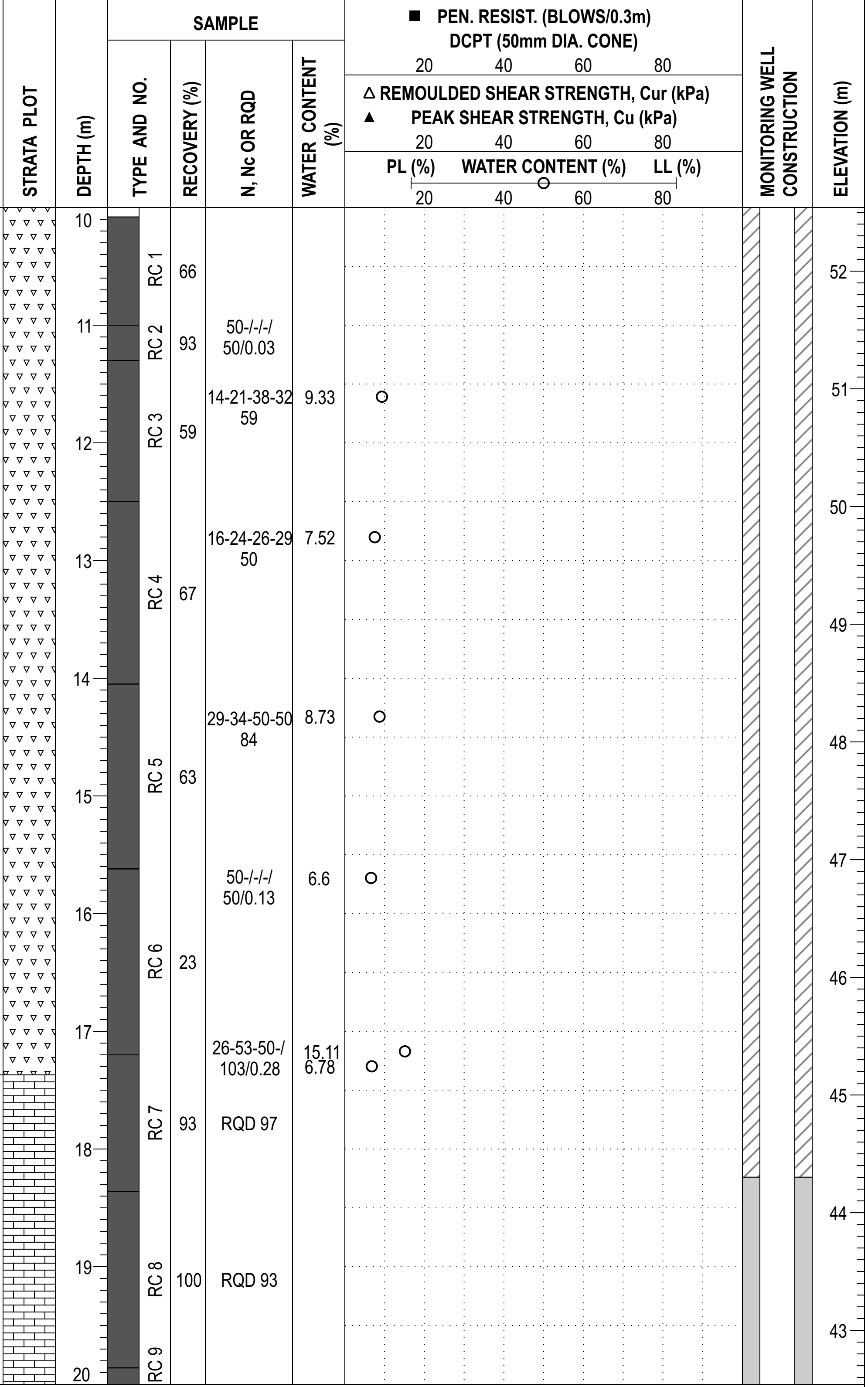
REMARKS: Borehole Drilled Indoors

DATE: October 31, 2024

FILE NO. : PG6655

HOLE NO. : BH 7-24

SAMPLE DESCRIPTION GROUND SURFACE	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
	RC 1	66		
	RC 2	93	50-/-/-/ 50/0.03	
			14-21-38-32 59	9.33
	RC 3	59		
- Increasing clay content by 12.5 m depth				
			16-24-26-29 50	7.52
	RC 4	67		
			29-34-50-50 84	8.73
	RC 5	63		
			50-/-/-/ 50/0.13	6.6
	RC 6	23		
			26-53-50-/ 103/0.28	15.11 6.78
17.37m [45.17m]				
BEDROCK: Excellent to good quality limestone	RC 7	93	RQD 97	
	RC 8	100	RQD 93	
	RC 9			



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368708.35 **NORTHING:** 5029132.04 **ELEVATION:** 62.54

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

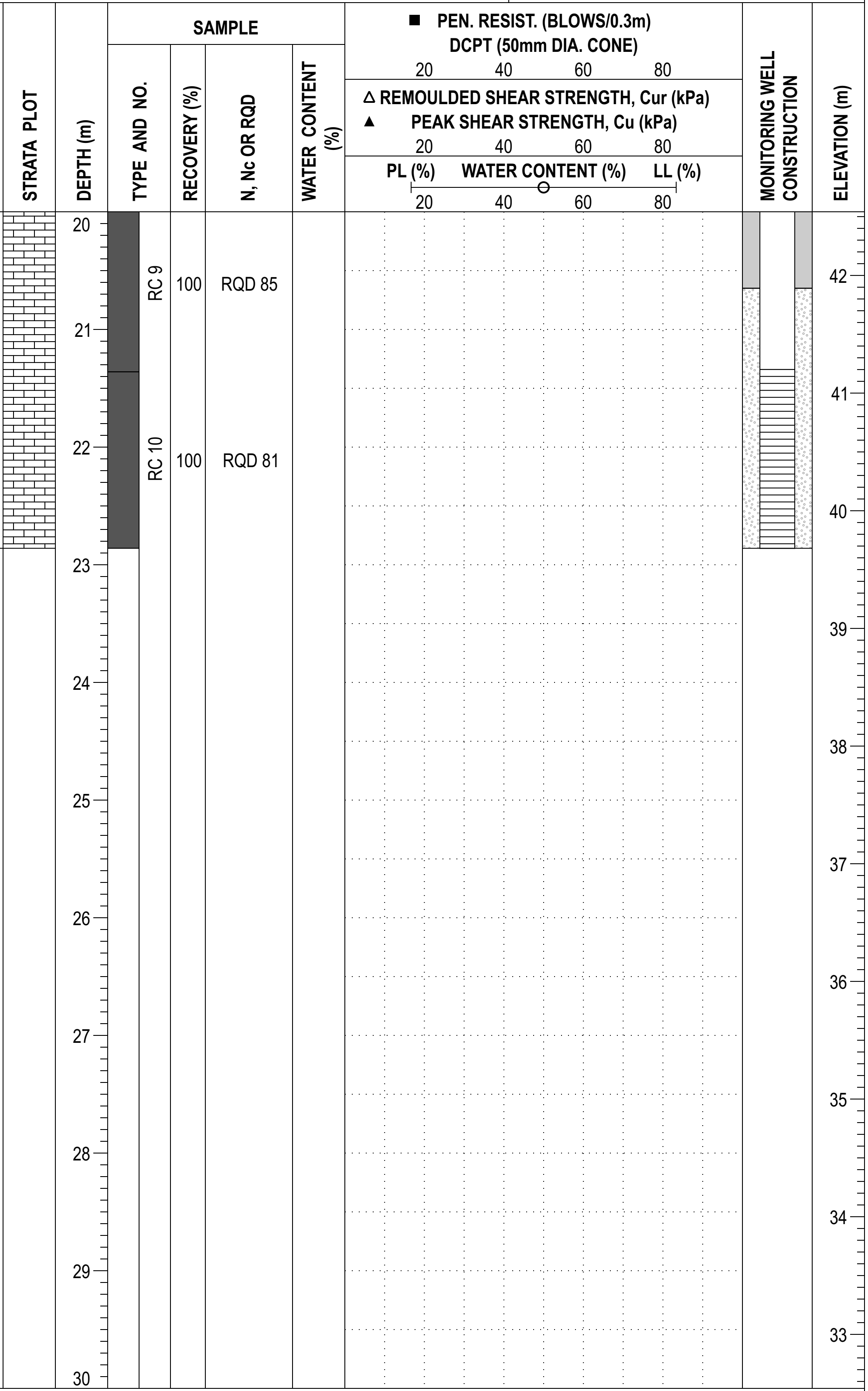
REMARKS: Borehole Drilled Indoors

DATE: October 31, 2024

FILE NO. : PG6655

HOLE NO. : BH 7-24

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N, Nc OR RQD	SAMPLE WATER CONTENT (%)
GROUND SURFACE					
	20–21.36	RC 9	100	RQD 85	
22.86m [39.68m]	21.36–22.86	RC 10	100	RQD 81	
End of Borehole (GWL at 2.41 m depth - November 24, 2024)					



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368708.35 **NORTHING:** 5029132.04 **ELEVATION:** 62.54

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

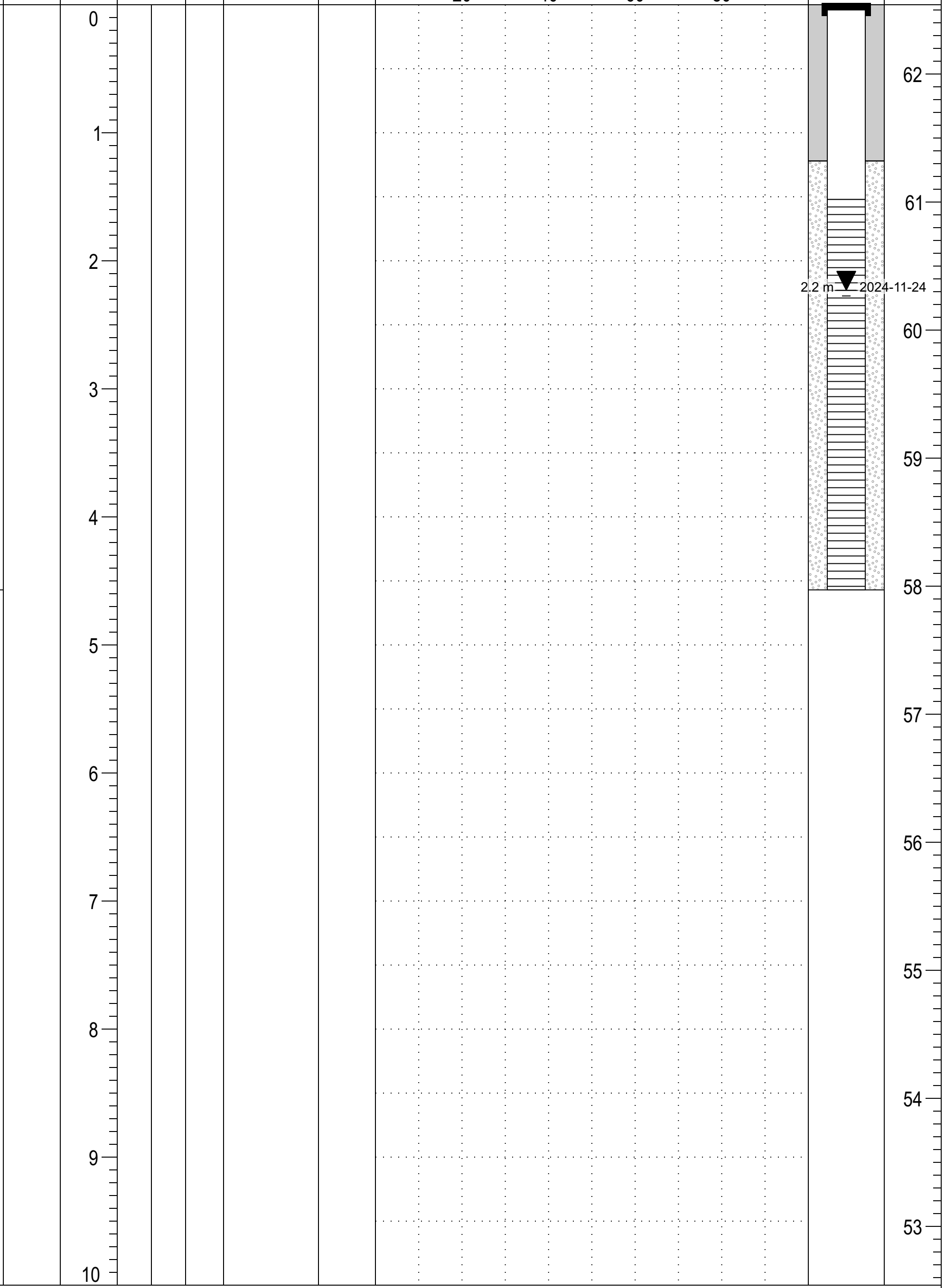
REMARKS: Borehole Drilled Indoors

DATE: October 31, 2024

FILE NO. : PG6655

HOLE NO. : BH 7A-24

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N, Nc OR RQD	SAMPLE WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH, Cur (kPa) ▲ PEAK SHEAR STRENGTH, Cu (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
Refer to BH 7-24 for soil profile 4.57m [57.97m]									
End of Borehole (GWL at 2.23 m depth - November 24, 2024)									



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COORD. SYS.: MTM ZONE 9 **EASTING:** 368758.14 **NORTHING:** 5029099.16 **ELEVATION:** 66.05

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS: Borehole Drilled Indoors

DATE: November 01, 2024

FILE NO. : PG6655

HOLE NO. : BH 8-24

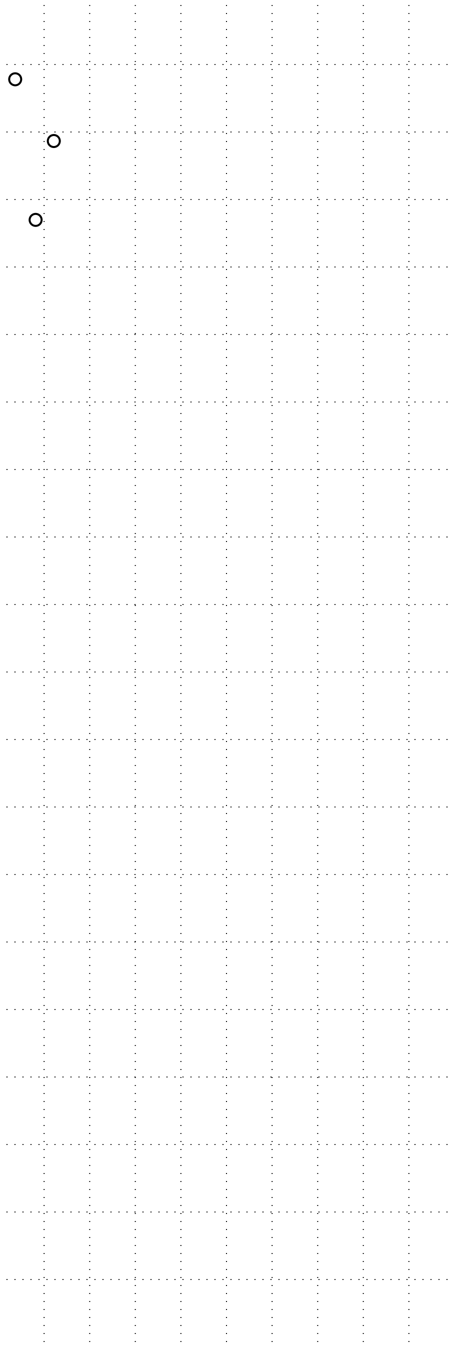
SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE	0					66
ASPHALT 0.06m [65.99m]						
FILL: Brown silty sand, with crushed stone and asphalt		SS 1	58	18-12-9-4 21	3.66	
	1	SS 2	50	5-5-8-8 13	12.13	65
1.91m [64.14m]		SS 3	71	7-8-6-50 14	8.14	
End of Borehole Practical refusal to augering at 1.91 m depth	2					64
	3					63
	4					62
	5					61
	6					60
	7					59
	8					58
	9					57
	10					

STRATA PLOT | PIEZOMETER CONSTRUCTION

■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
20 40 60 80

△ REMOULDED SHEAR STRENGTH, Cur (kPa)
▲ PEAK SHEAR STRENGTH, Cu (kPa)
20 40 60 80

PL (%) WATER CONTENT (%) LL (%)
20 40 60 80



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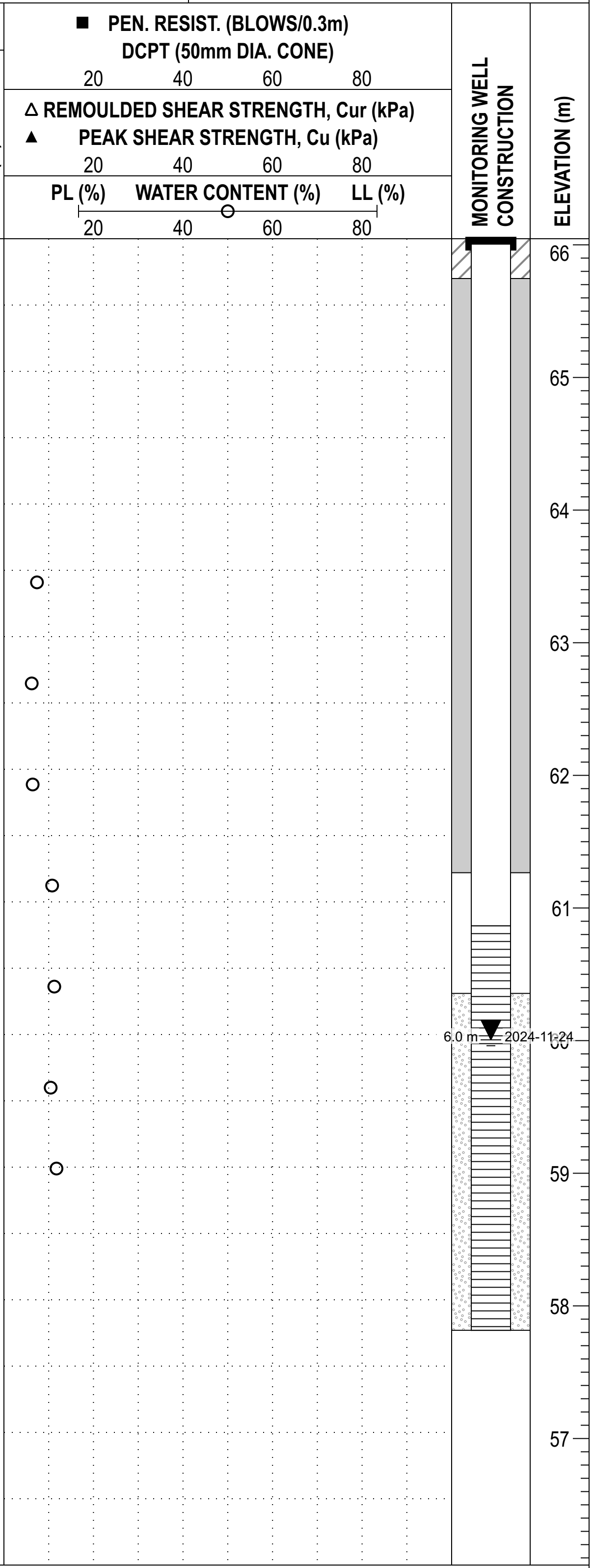
SOIL PROFILE AND TEST DATA
Geotechnical Investigation
Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368757.19 **NORTHING:** 5029100.00 **ELEVATION:** 66.05

PROJECT: Proposed North Stands
BORINGS BY: CME-55 Low Clearance Drill
REMARKS: Borehole Drilled Indoors
DATE: November 01, 2024

FILE NO. : PG6655
HOLE NO. : BH8A-24

SAMPLE DESCRIPTION GROUND SURFACE	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
Refer to BH 8-24 for soil profile	0				
2.13m [63.92m]					
Loose, brown **SILTY SAND**, trace gravel		SS 1	54	4-3-3-2 6	7.38
		SS 2	75	1-1-1-1 2	6.21
		SS 3	58	3-4-4-5 8	6.42
		SS 4	79	3-3-4-4 7	10.77
5.26m [60.79m]					
GLACIAL TILL: Compact to very dense, brown silty sand, with gravel, cobbles and boulders		SS 5	67	18-11-16-30 27	11.25
		SS 6	67	26-34-33-35 67	10.44
		SS 7	67	26-20-19-30 39	11.74
7.62m [58.43m]					
End of Borehole (GWL at 6.04 m depth - November 24, 2024)	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 2-21

BORINGS BY CME-55 Low Clearance Drill

DATE October 25, 2021

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

DEPTH (m)	ELEV. (m)
0	66.04
1	65.04
2	64.04
3	63.04
4	62.04
5	61.04
6	60.04
7	59.04
8	58.04
9	57.04
10	56.04

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

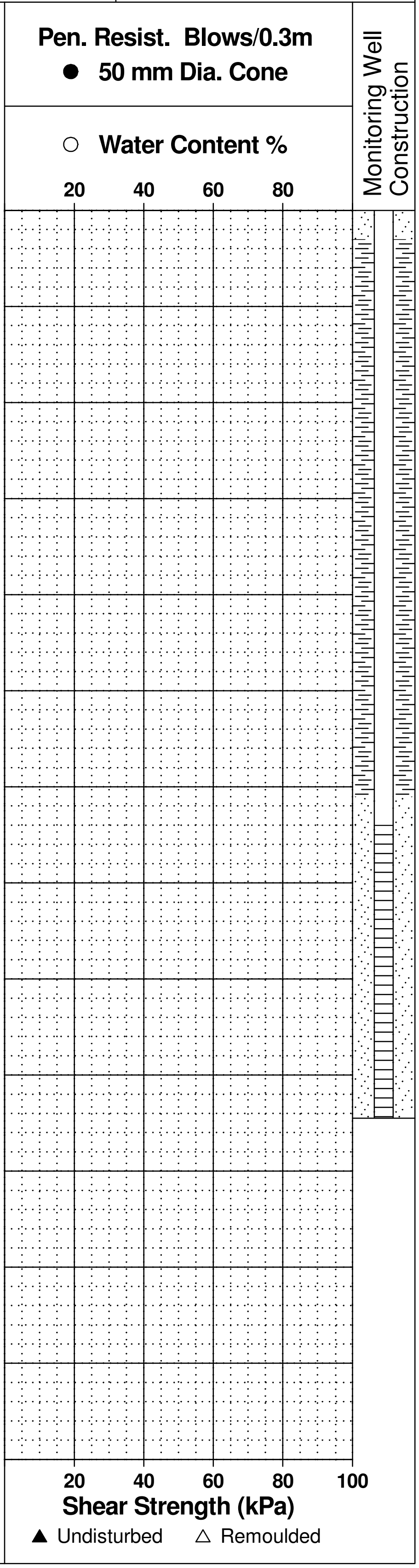
BORINGS BY CME-55 Low Clearance Drill

DATE November 17, 2021

FILE NO. PG5792

HOLE NO. BH 8-21

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



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

GROUND SURFACE

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	RC	1	55	
	RC	2	30	
	SS	18	58	28
	RC	3	0	
	SS	19	0	50+
	RC	4	36	
	SS	20	25	50+
	RC	5	50	
	SS	21	0	
	RC	6	35	
21.28	SS	22		50+
BEDROCK: Excellent quality, grey limestone with occasional shale partings	RC	7	100	90
	RC	8	100	95
24.10				
End of Borehole				

DEPTH (m)	ELEV. (m)
13	52.45
14	51.45
15	50.45
16	49.45
17	48.45
18	47.45
19	46.45
20	45.45
21	44.45
22	43.45
23	42.45
24	41.45

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

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 18, 2021

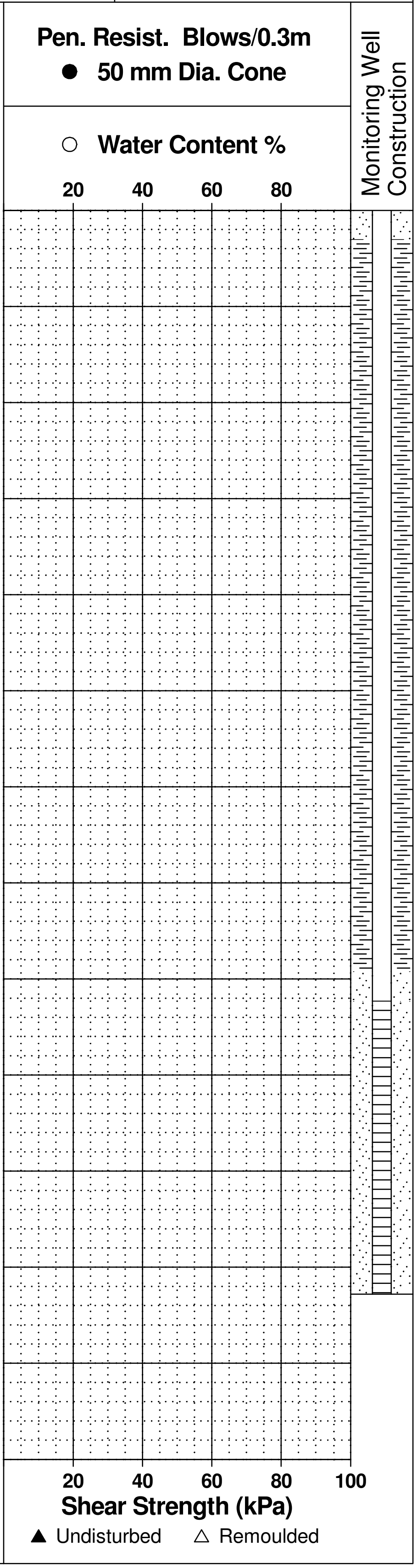
FILE NO. PG5792

HOLE NO. BH 9-21

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

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

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



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SOIL 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 18, 2021

FILE NO. PG5792

HOLE NO. BH 9-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	SS	12	22	50+
	RC	6	70	
	RC	7	37	
	RC	8	25	
	SS	13	0	50+
	RC	9	48	
	RC	10	11	
21.36				
BEDROCK: Excellent quality, grey limestone with occasional shale partings	RC	11	100	90
	RC	12	100	100
24.08 End of Borehole				

DEPTH (m)	ELEV. (m)
13	54.07
14	53.07
15	52.07
16	51.07
17	50.07
18	49.07
19	48.07
20	47.07
21	46.07
22	45.07
23	44.07
24	43.07

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)

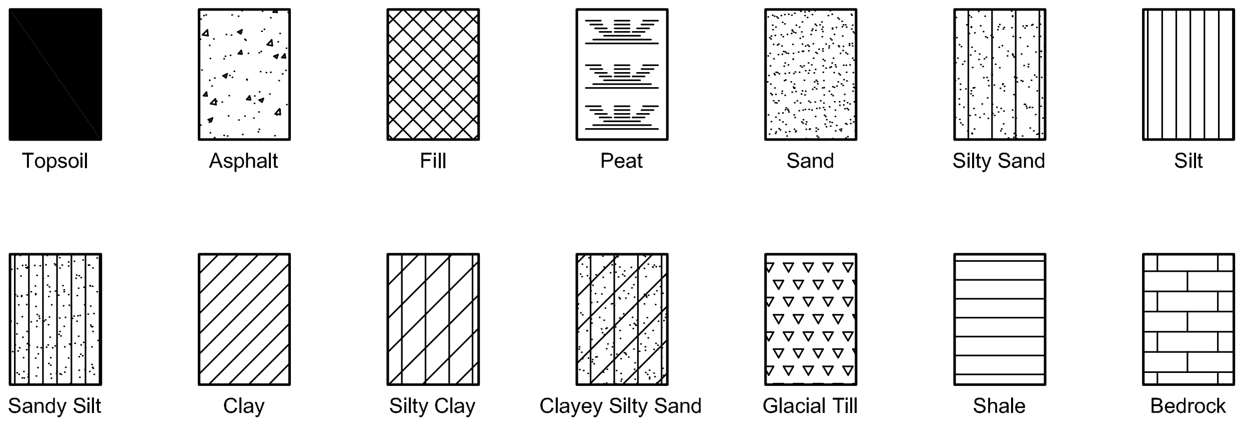
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

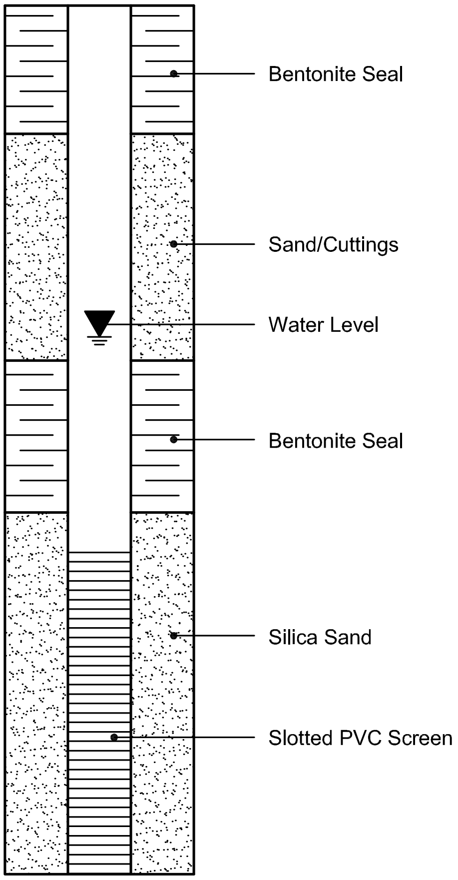
SYMBOLS AND TERMS (continued)

STRATA PLOT

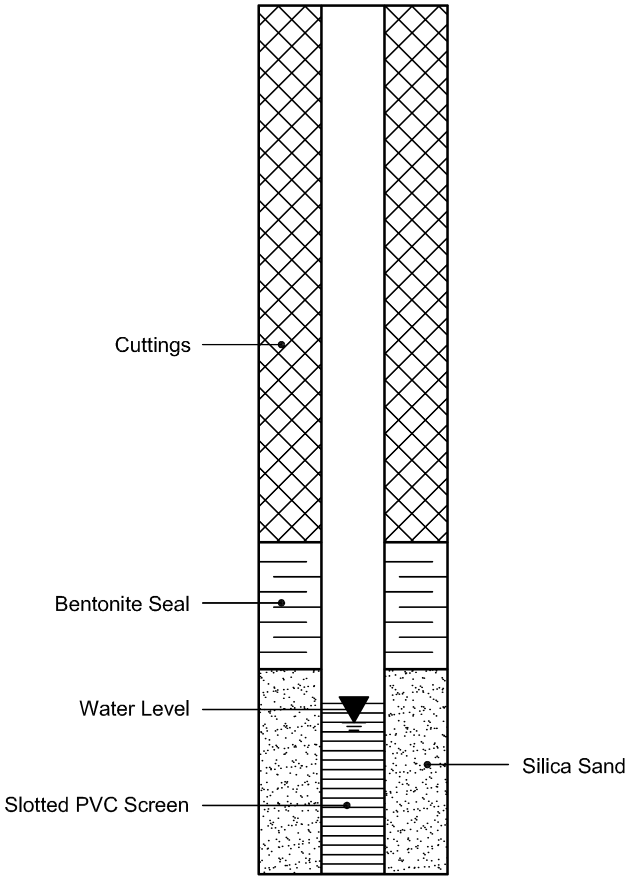


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION

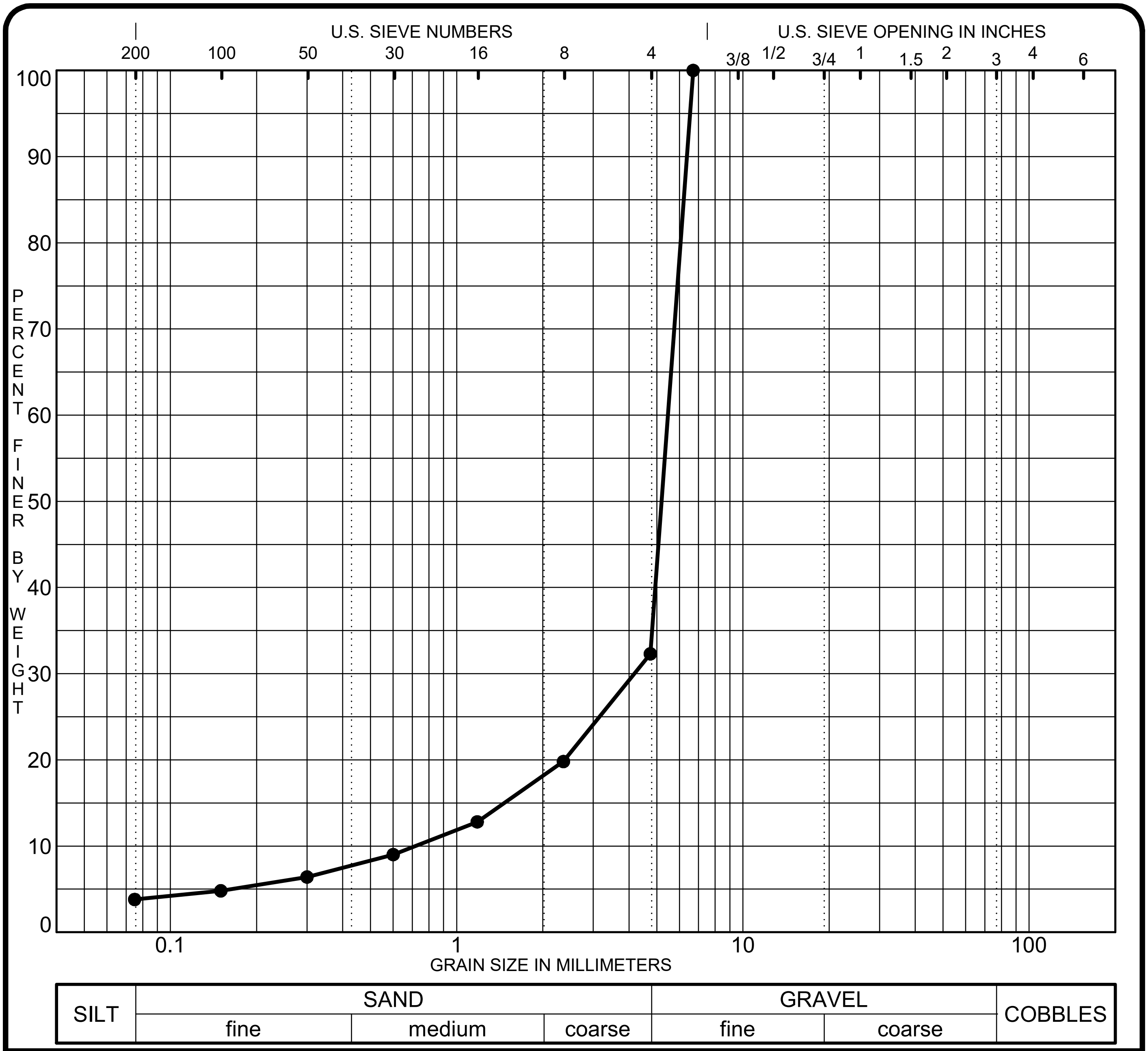


Photograph 1: BH 8-21 RC7.

Photograph 2: BH 8-21 RC8.

Photograph 3: BH 9-21 RC12.

Photograph 4: BH 9-21 RC12.



Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 5-24 SS5**						**4.45**	**7.6**
⊠								
▲								
★								

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 5-24 SS5**	**6.70**	**5.47**	**4.176**	**0.7169**	**67.7**	**28.5**	**3.8**	
⊠									
▲									
★									

CLIENT **City of Ottawa**

FILE NO. **PG6655**

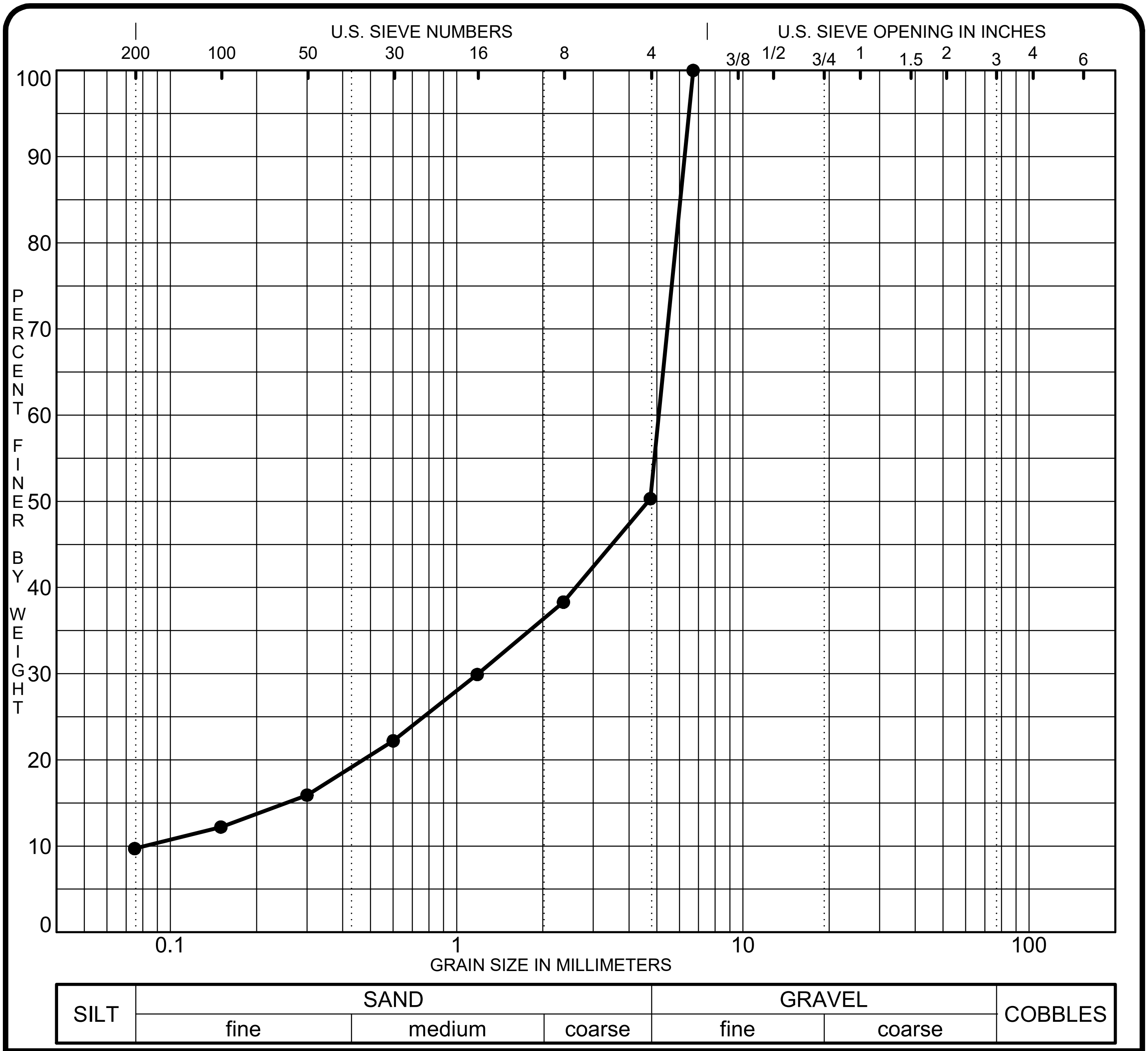
PROJECT **Geotechnical Investigation - Lansdowne Park Redevelopment, Proposed North Stands**

DATE **15 Nov 24**

PATERSON GROUP

9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

GRAIN SIZE DISTRIBUTION



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 7-24**	**SS5**						**3.42**	**62.3**
☒									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 7-24**	**SS5**	**6.70**	**5.08**	**1.190**	**0.0815**	**49.7**	**40.6**	**9.7**	
☒										
▲										
★										

CLIENT **City of Ottawa**

FILE NO. **PG6655**

PROJECT **Geotechnical Investigation - Lansdowne Park Redevelopment, Proposed North Stands**

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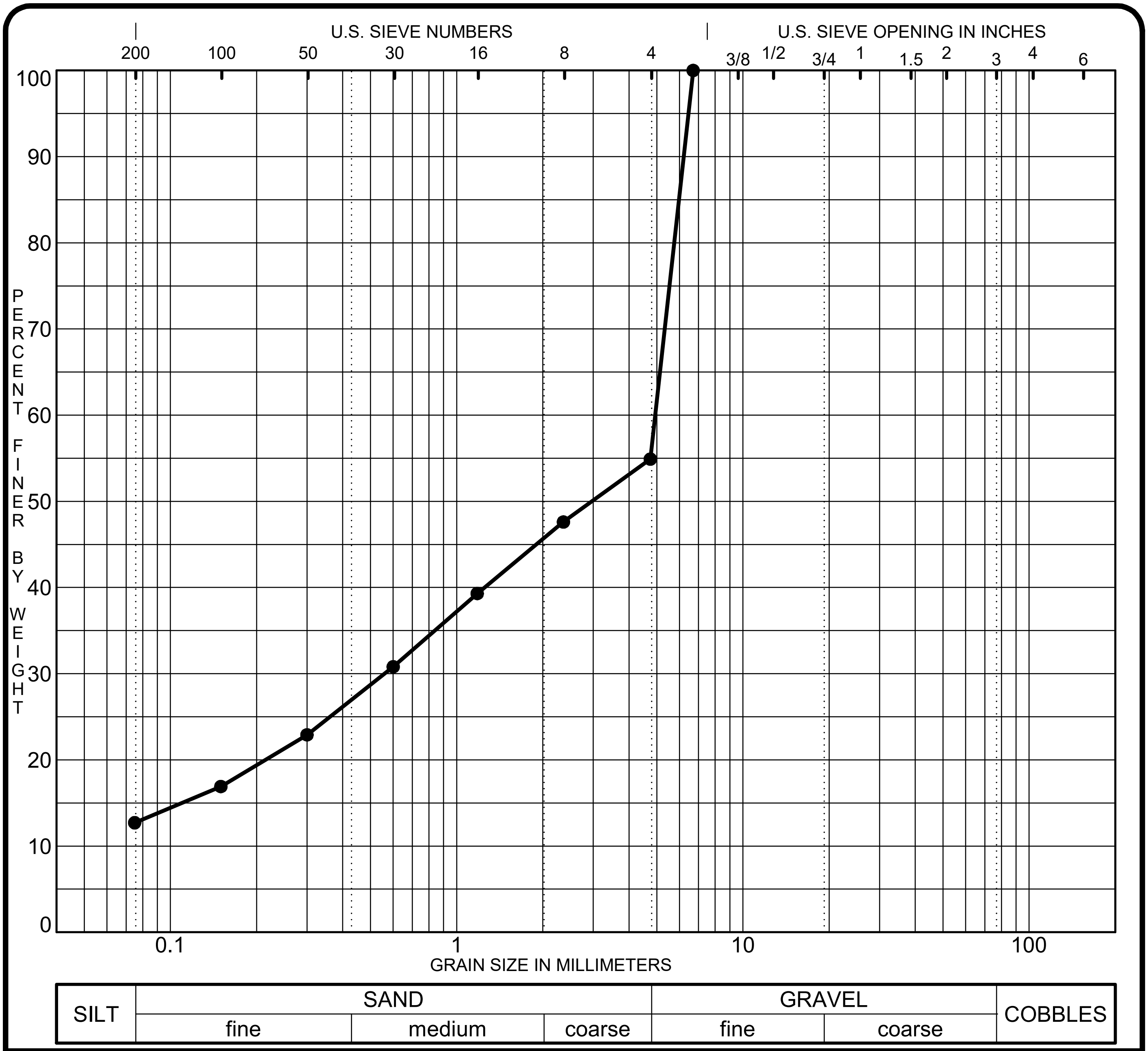
GRAIN SIZE DISTRIBUTION



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 8A-24 SS7**							
☒								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 8A-24 SS7**	**6.70**	**4.94**	**0.559**		**45.1**	**42.2**	**12.7**	
☒									
▲									
★									

CLIENT **City of Ottawa**

PROJECT **Geotechnical Investigation - Lansdowne Park Redevelopment, Proposed North Stands**

FILE NO. **PG6655**

DATE **15 Nov 24**

PATERSON GROUP

9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

GRAIN SIZE DISTRIBUTION

PATERSON GROUP

CONCRETE CORE COMPRESSIVE STRENGTH CSA A23.2-14C

CLIENT:	**City of Ottawa**	FILE No.:	**PG6655**
PROJECT:	Lab Testing	REPORT No.:	1
SITE ADDRESS:	**Lansdowne Redevelopment**	DATE REPT'D:	13-Nov-24
STRUCTURE TYPE & LOCATION:			

SAMPLE INFORAMTION			
LAB NO.:	**58237**	**58238**	
SAMPLE NO.:	RC11	RC8	
LOCATION:	BH5 - 24 / 56'8" - 57'2'	BH7 - 24 / 61'3" - 61'9"	
SAMPLE DATES			
DATE CAST	-	-	
DATE CORED	31-Oct-24	31-Oct-24	
DATE RECEIVED	11-Nov-24	11-Nov-24	
DATE TESTED	12-Nov-24	12-Nov-24	
SAMPLE DIMENSIONS			
AVERAGE DIAMETER (mm)	63.00	63.00	
HEIGHT (mm)	118.00	124.00	
WEIGHT (g)	1020	1060	
AREA (mm^2)	3117	3117	
VOLUME (cm^3)	368	387	
UNIT WEIGHT (kg/m^3)	**2773**	**2742**	
TEST RESULTS			
H / D RATIO	1.87	1.97	
CORRECTION FACTOR	0.989	0.996	
LOAD (lbs)	48900	53200	
GROSS Mpa	69.8	75.9	
MPa CORRECTED	**69.0**	**75.6**	
FORM OF BREAK	**Type A**	**Type A**	
DIRECTION OF LOADING	PARALLEL	PARALLEL	
CURING CONDITIONS	SITE→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→		

REMARKS

TECHNICAL PERSONNEL

TECHNICIAN:	**VERIFIED BY:**	**C. Beadow**	**APPROVED BY:**	**Joe Forsyth, P. Eng.**

CERTIFIED LAB

John D. Paterson & Associates Ltd., 28 Concourse Gate, Nepean, ON

Certificate of Analysis

Report Date: 25-Nov-2024

Client: **Paterson Group Consulting Engineers (Ottawa)**

Order Date: 19-Nov-2024

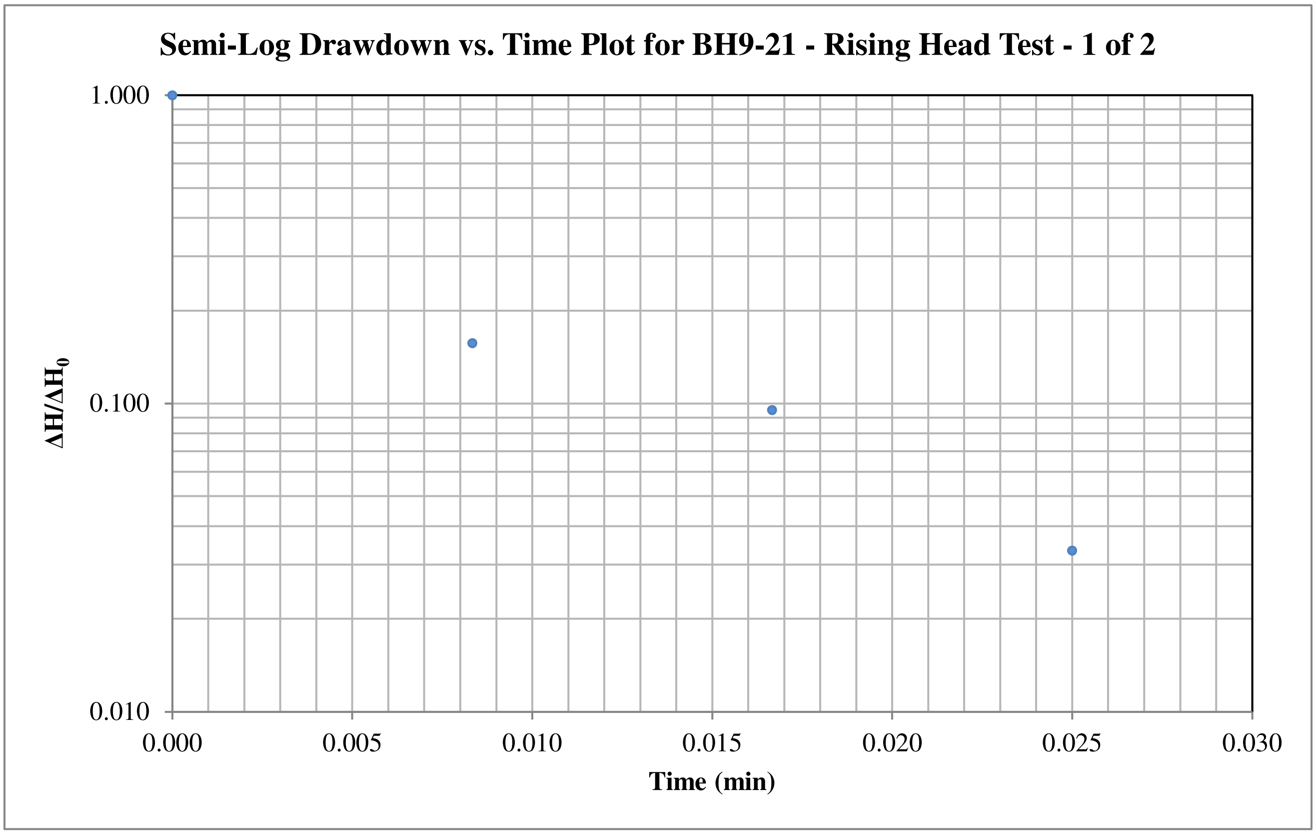
Client PO: 61790

Project Description: PG6655

	Client ID:	BH8A-24-SS1	-	-	-		
	Sample Date:	01-Nov-24 09:00	-	-	-	-	-
	Sample ID:	2447213-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	94.7	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.85	-	-	-	-	-
Resistivity	0.1 Ohm.m	14.2	-	-	-	-	-
Anions							
Chloride	10 ug/g	243	-	-	-	-	-
Sulphate	10 ug/g	220	-	-	-	-	-

Hvorslev Hydraulic Conductivity Analysis

Project: Lansdowne - Trinity
Test Location: BH9-21
Test: Rising Head - 1 of 2
Date: December 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.59613

Well Parameters:

L	3 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

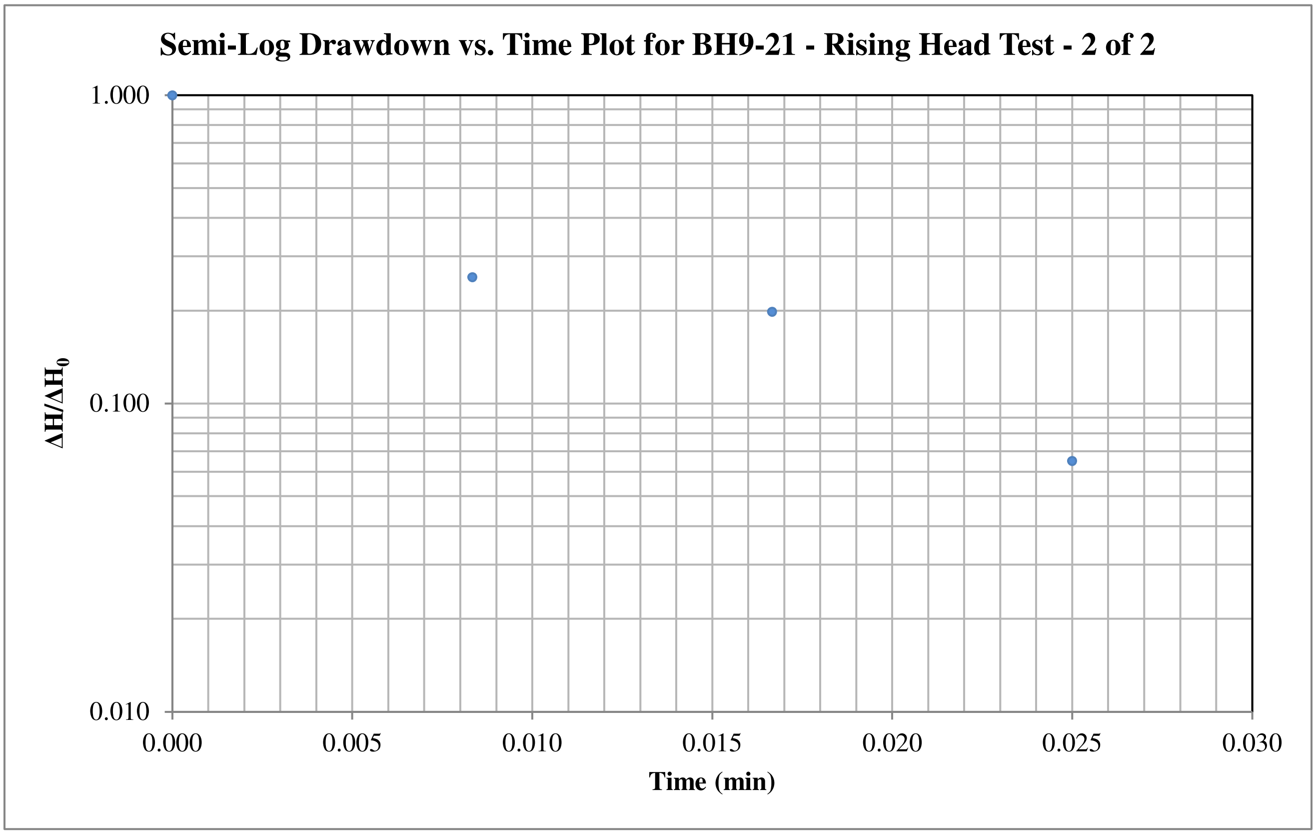
Data Points (from plot):

t^*: 0.006 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 5.86E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Lansdowne - Trinity
Test Location: BH9-21
Test: Rising Head - 2 of 2
Date: December 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.59613

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

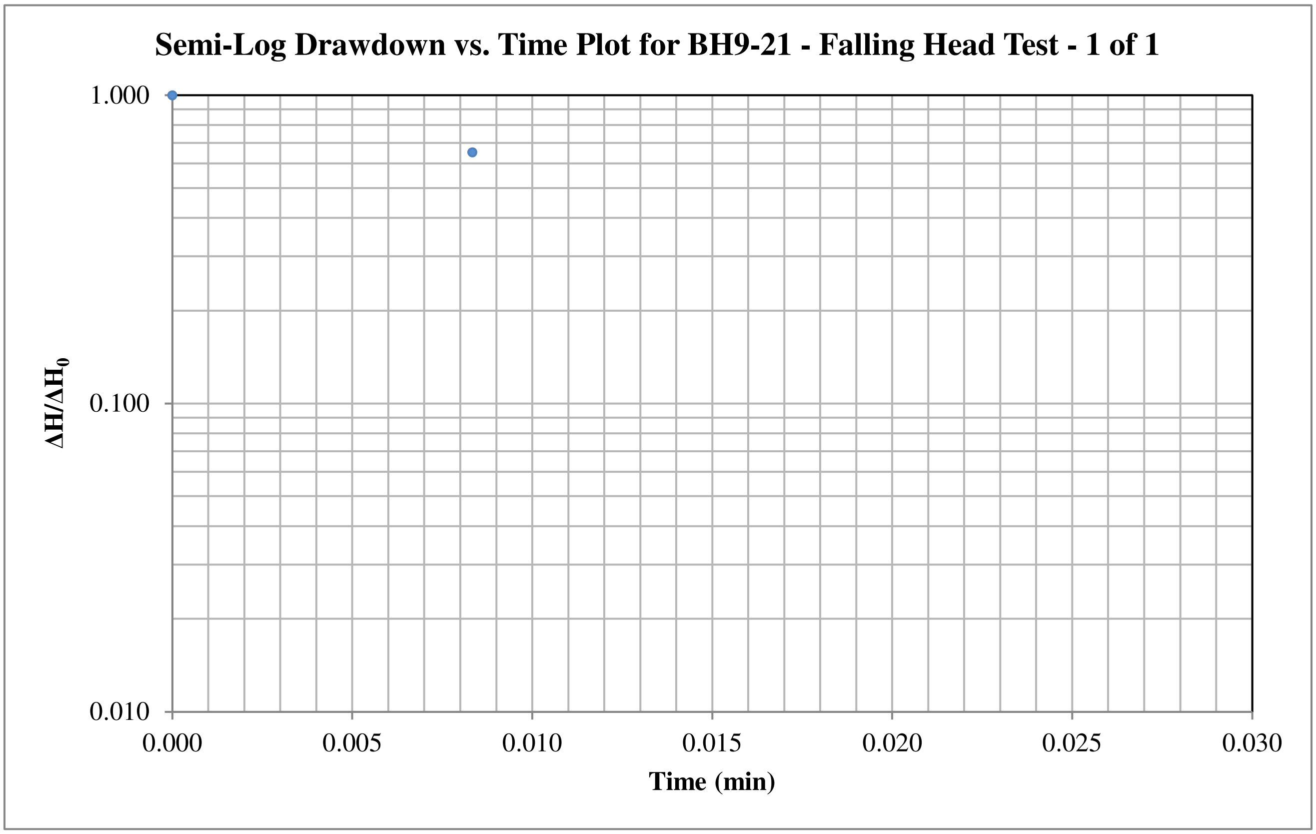
Data Points (from plot):

t*: 0.007 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 5.16E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Lansdowne - Trinity
Test Location: BH9-21
Test: Falling Head Test - 1 of 1
Date: December 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.59613

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.03175	m	Diameter of well
r_c	0.01588	m	Radius of well

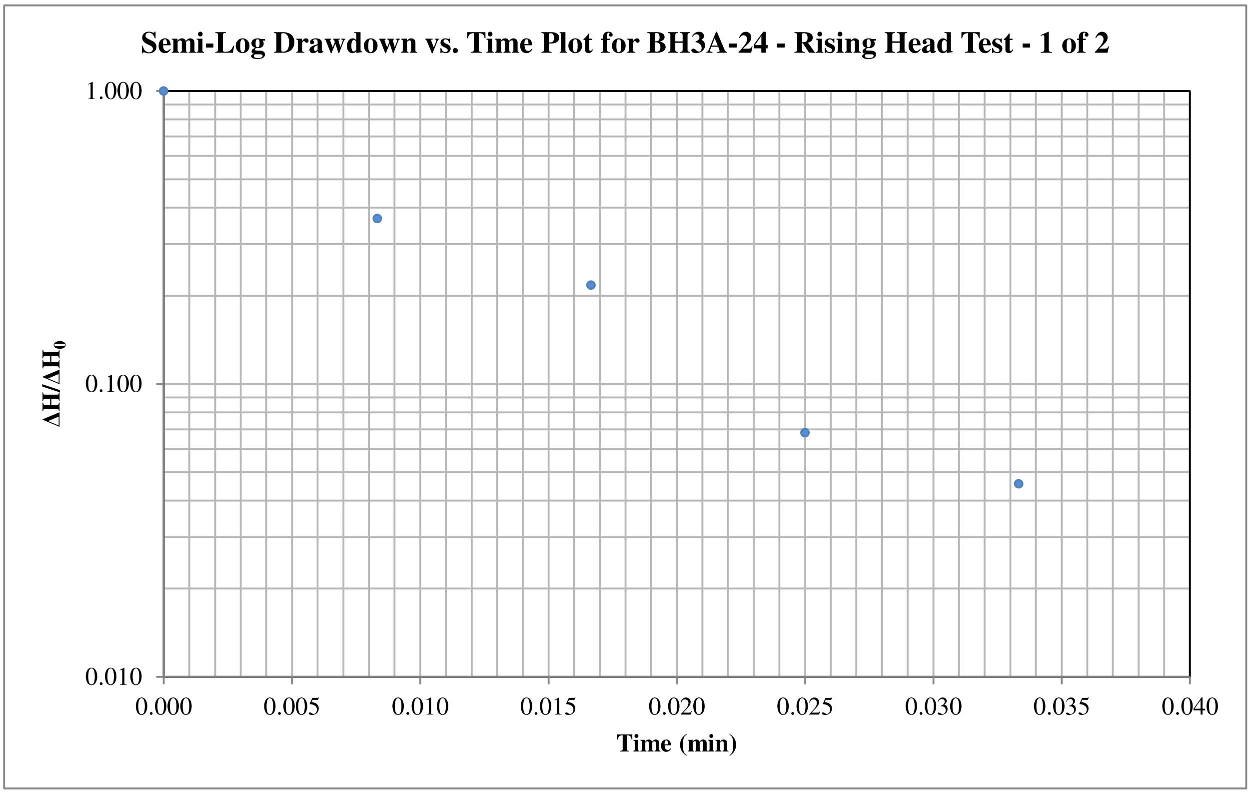
Data Points (from plot):

t*: 0.012 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.05E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: City of Ottawa - Lansdowne
Test Location: BH3A-24
Test: Rising Head - 1 of 2
Date: November 14, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.17929

Well Parameters:

L	1.6 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

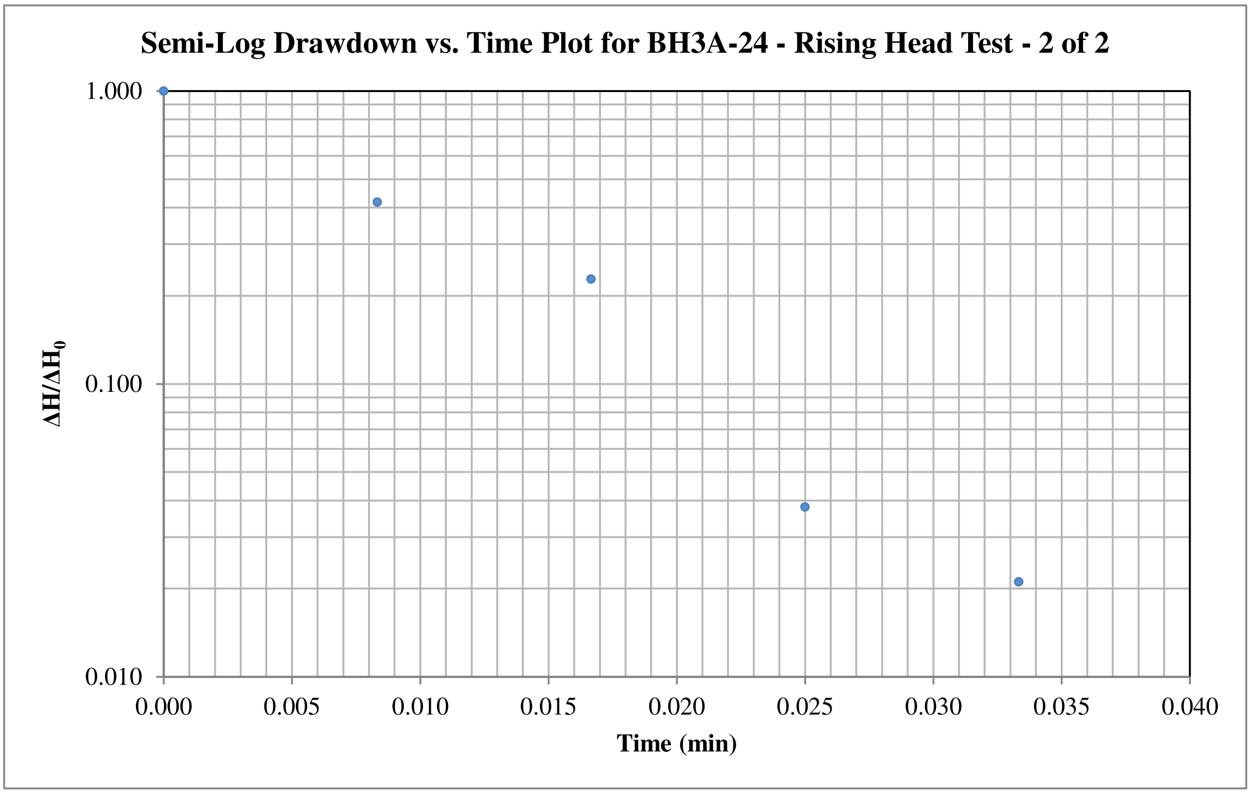
Data Points (from plot):

t*: 0.008 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 7.26E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: City of Ottawa - Lansdowne
Test Location: BH3A-24
Test: Rising Head - 2 of 2
Date: November 14, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.17929

Well Parameters:

L	1.6 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

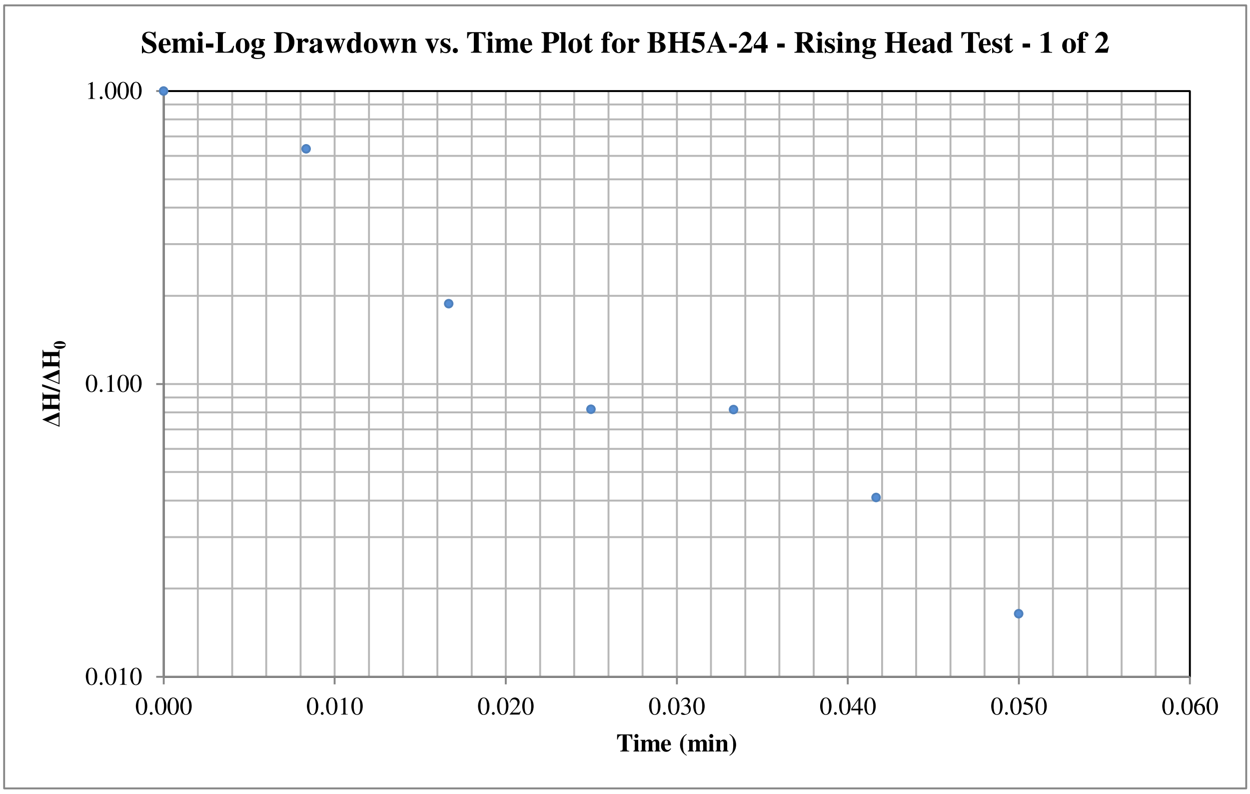
Data Points (from plot):

t*: 0.010 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 5.77E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: City of Ottawa - Lansdowne
Test Location: BH5A-24
Test: Rising Head - 1 of 2
Date: November 14, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.00482

Well Parameters:

L	2.4 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

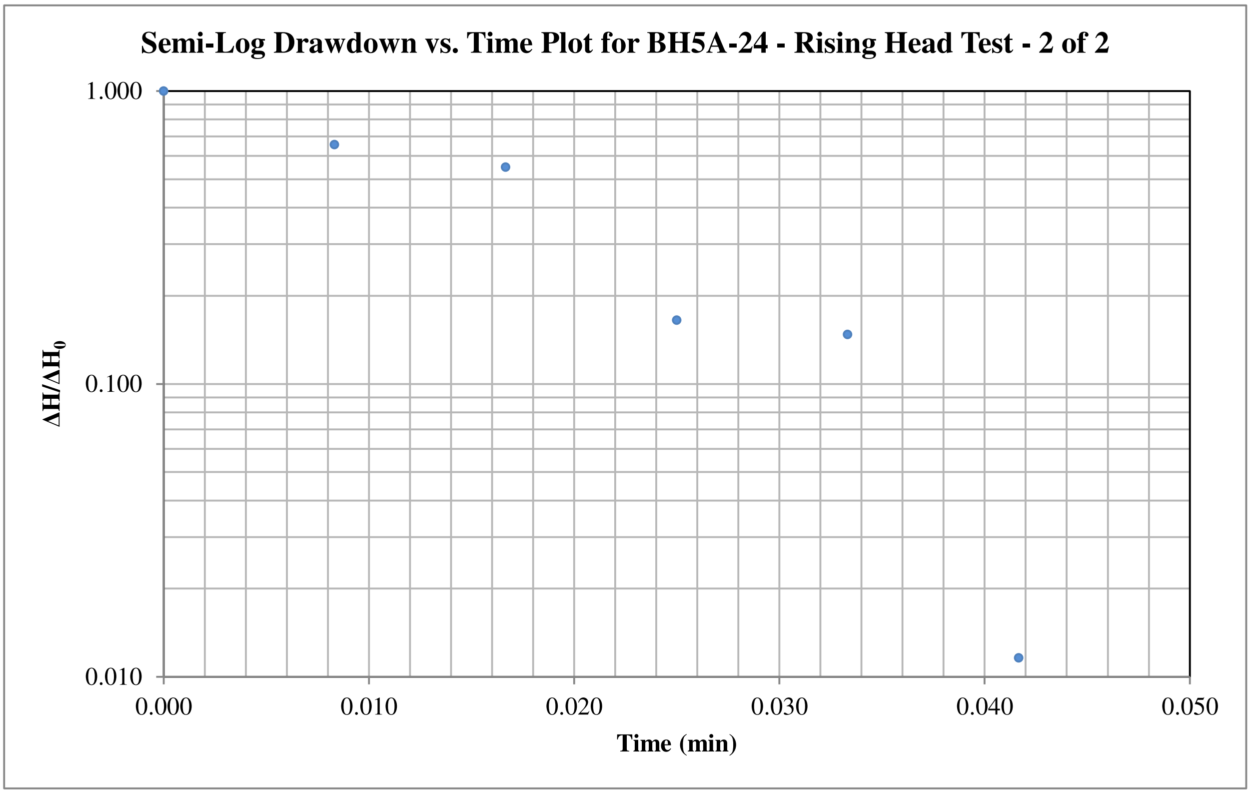
Data Points (from plot):

t*: 0.013 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.29E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: City of Ottawa - Lansdowne
Test Location: BH5A-24
Test: Rising Head - 2 of 2
Date: November 14, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.00482

Well Parameters:

L	2.4 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

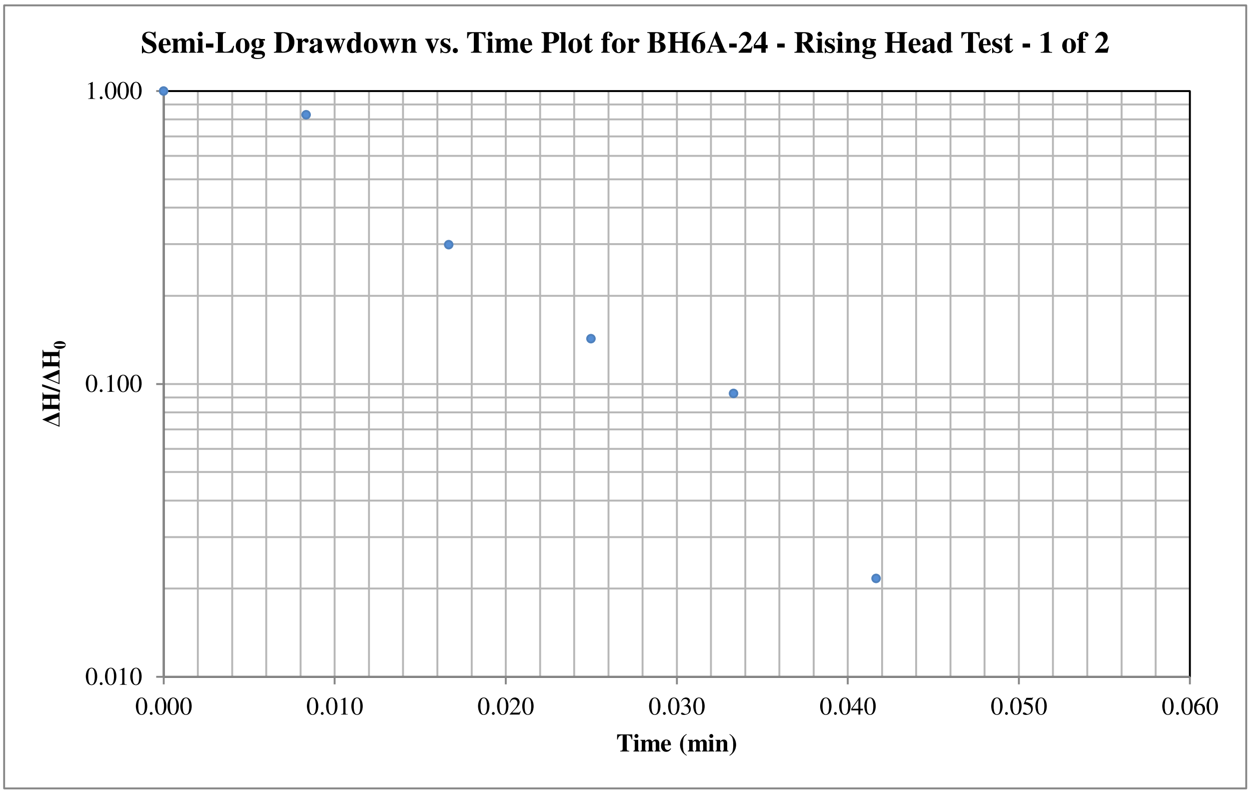
Data Points (from plot):

t*: 0.013 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.29E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: City of Ottawa - Lansdowne
Test Location: BH6A-24
Test: Rising Head - 1 of 2
Date: November 14, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.01485

Well Parameters:

L	2.41 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

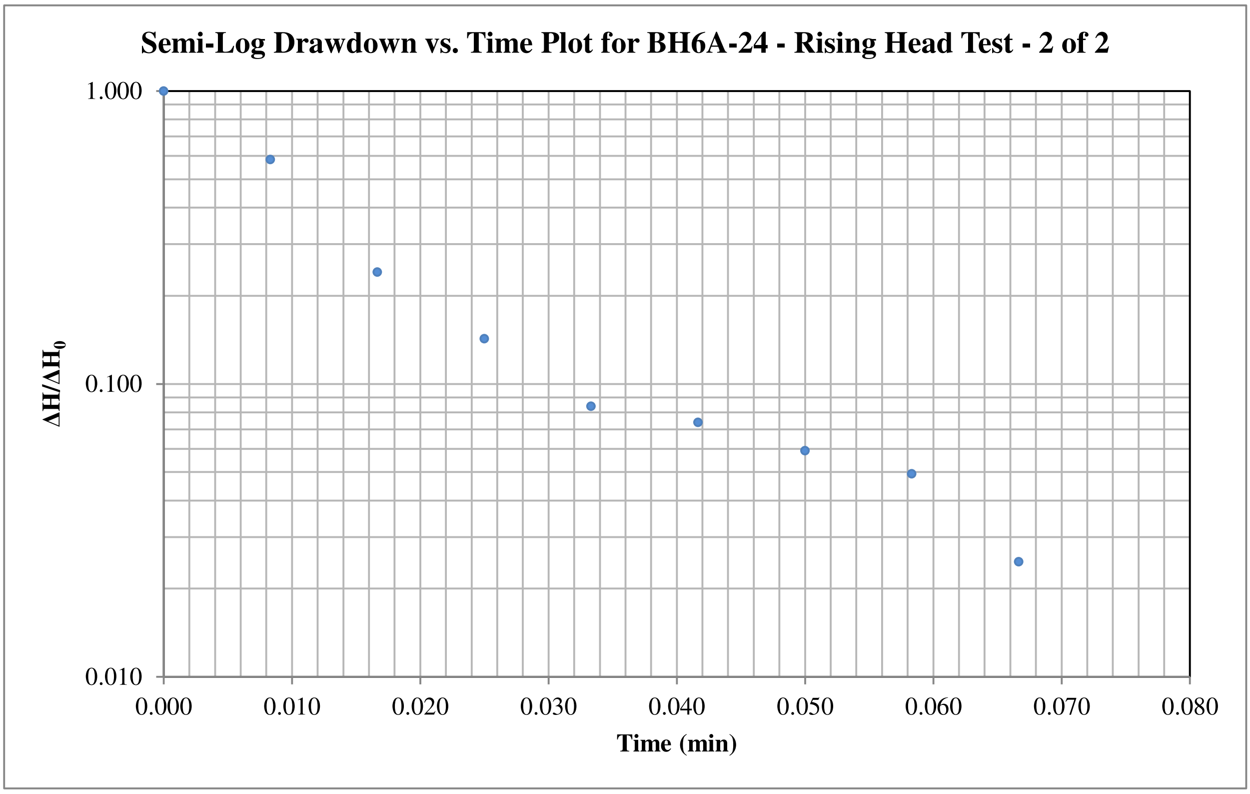
Data Points (from plot):

t*: 0.016 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.80E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: City of Ottawa - Lansdowne
Test Location: BH6A-24
Test: Rising Head - 2 of 2
Date: November 14, 2024


Semi-Log Drawdown vs. Time Plot for BH6A-24 - Rising Head Test - 2 of 2

Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.01485

Well Parameters:

L	2.41 m	Saturated length of screen or open hole
D	0.03175 m	Diameter of well
r_c	0.01588 m	Radius of well

Data Points (from plot):

t*: 0.014 minutes

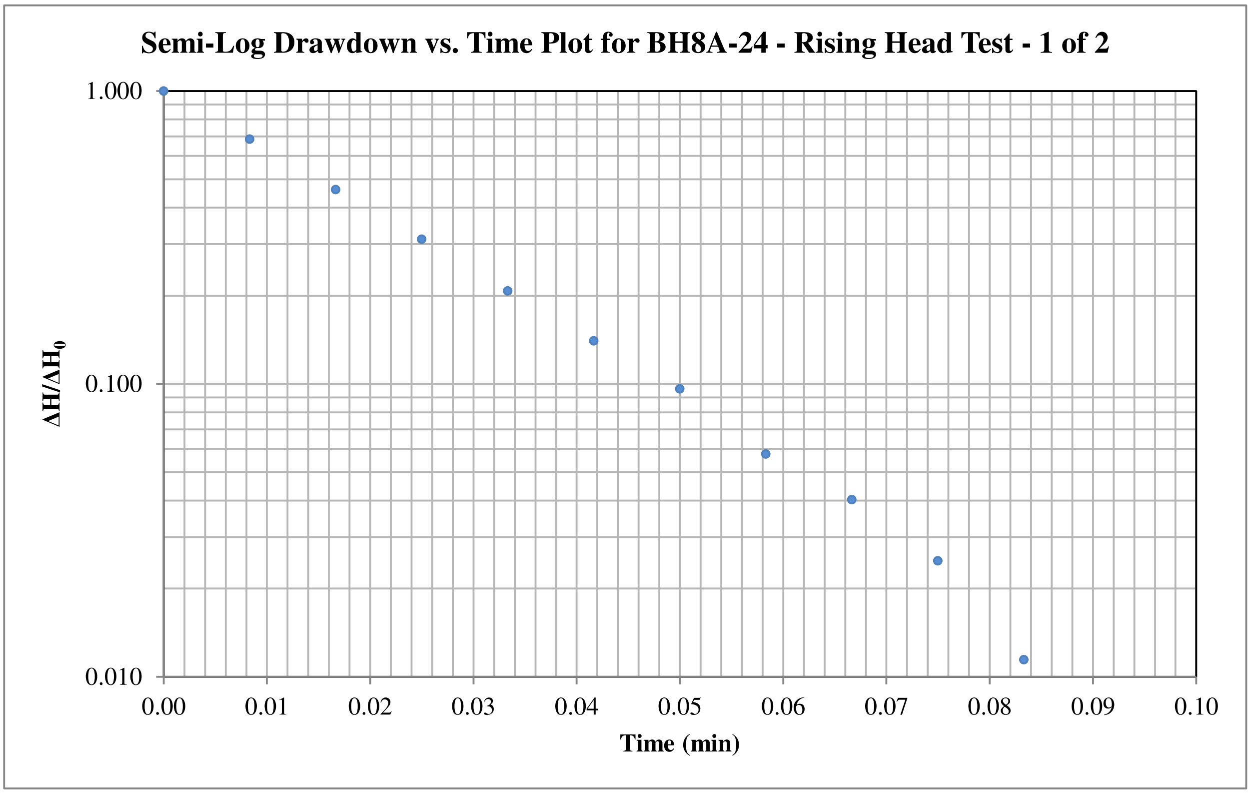
$\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.22E-04 m/sec

File: PH5000

Hvorslev Hydraulic Conductivity Analysis

Project: City of Ottawa - Lansdowne
Test Location: BH8A-24
Test: Rising Head - 1 of 2
Date: November 14, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.08738

Well Parameters:

L	2.19 m	Saturated length of screen or open hole
D	0.0508 m	Diameter of well
r_c	0.0254 m	Radius of well

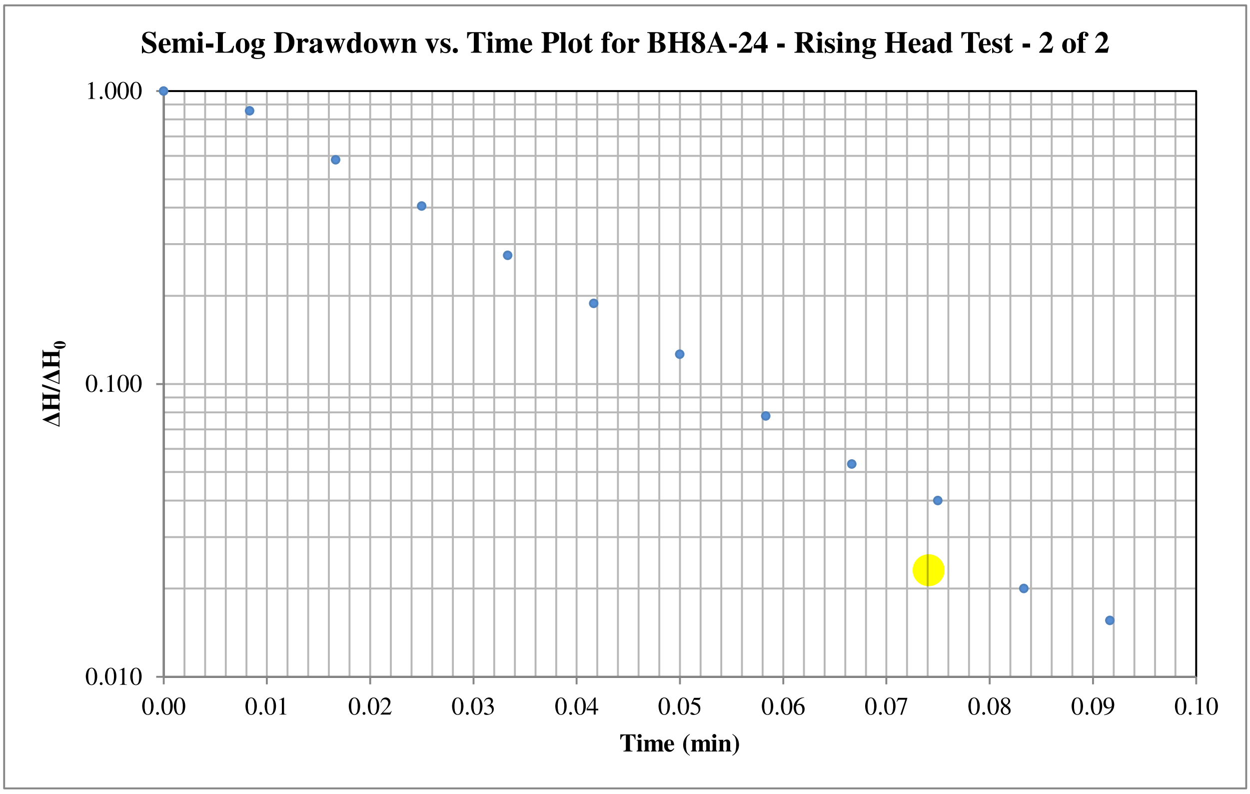
Data Points (from plot):

t*: 0.022 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 5.00E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: City of Ottawa - Lansdowne
Test Location: BH8A-24
Test: Rising Head - 2 of 2
Date: November 14, 2024



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.08738

Well Parameters:

L	2.19 m	Saturated length of screen or open hole
D	0.0508 m	Diameter of well
r_c	0.0254 m	Radius of well

Data Points (from plot):

t*: 0.027 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.99E-04 m/sec

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 - GROUNDWATER ELEVATION MONITORING - PROGRAM UPDATE

DRAWING PG6655-2 & PG6655-2A - TEST HOLE LOCATION PLAN

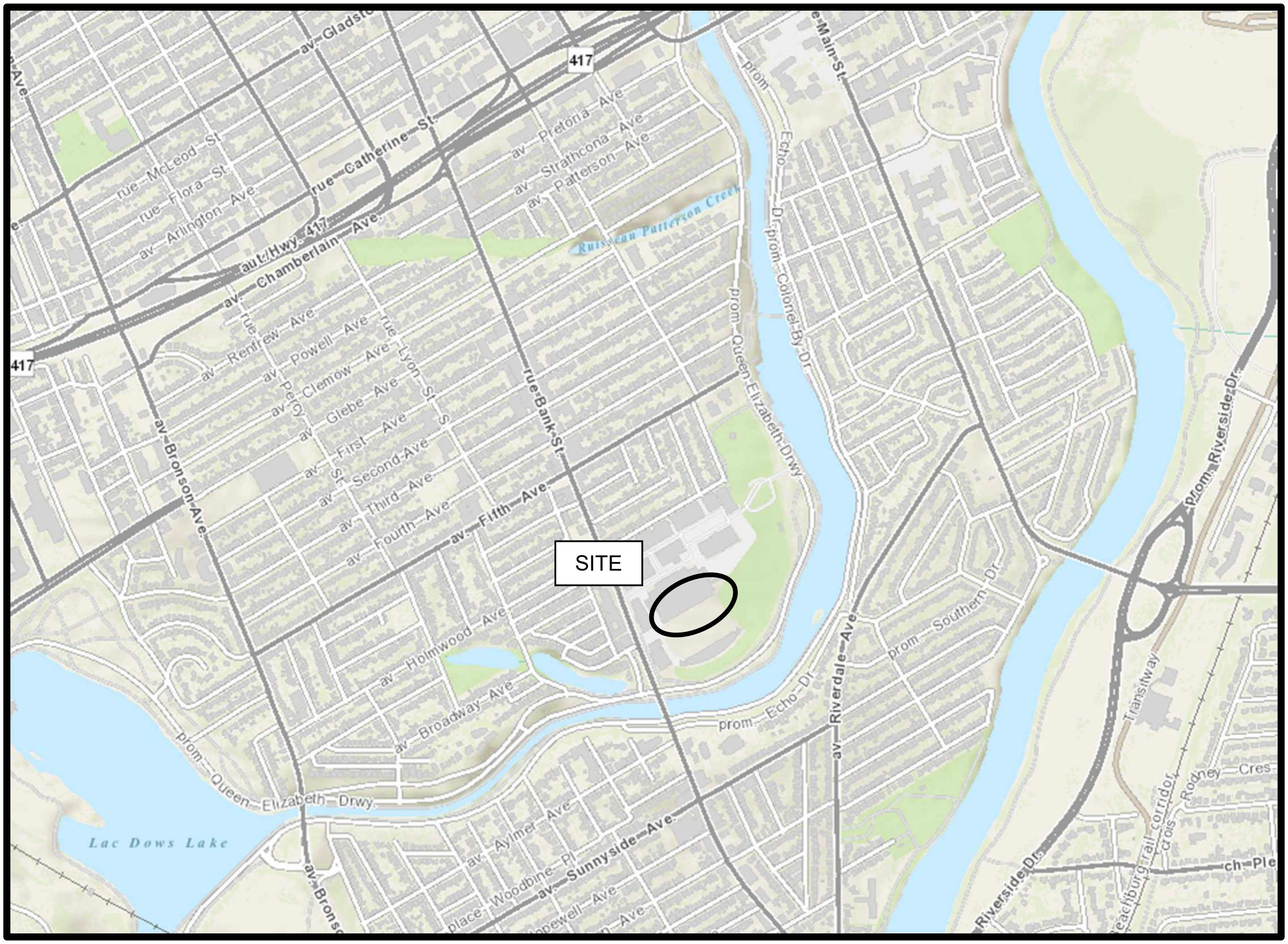


FIGURE 1

KEY PLAN

PATERSON GROUP

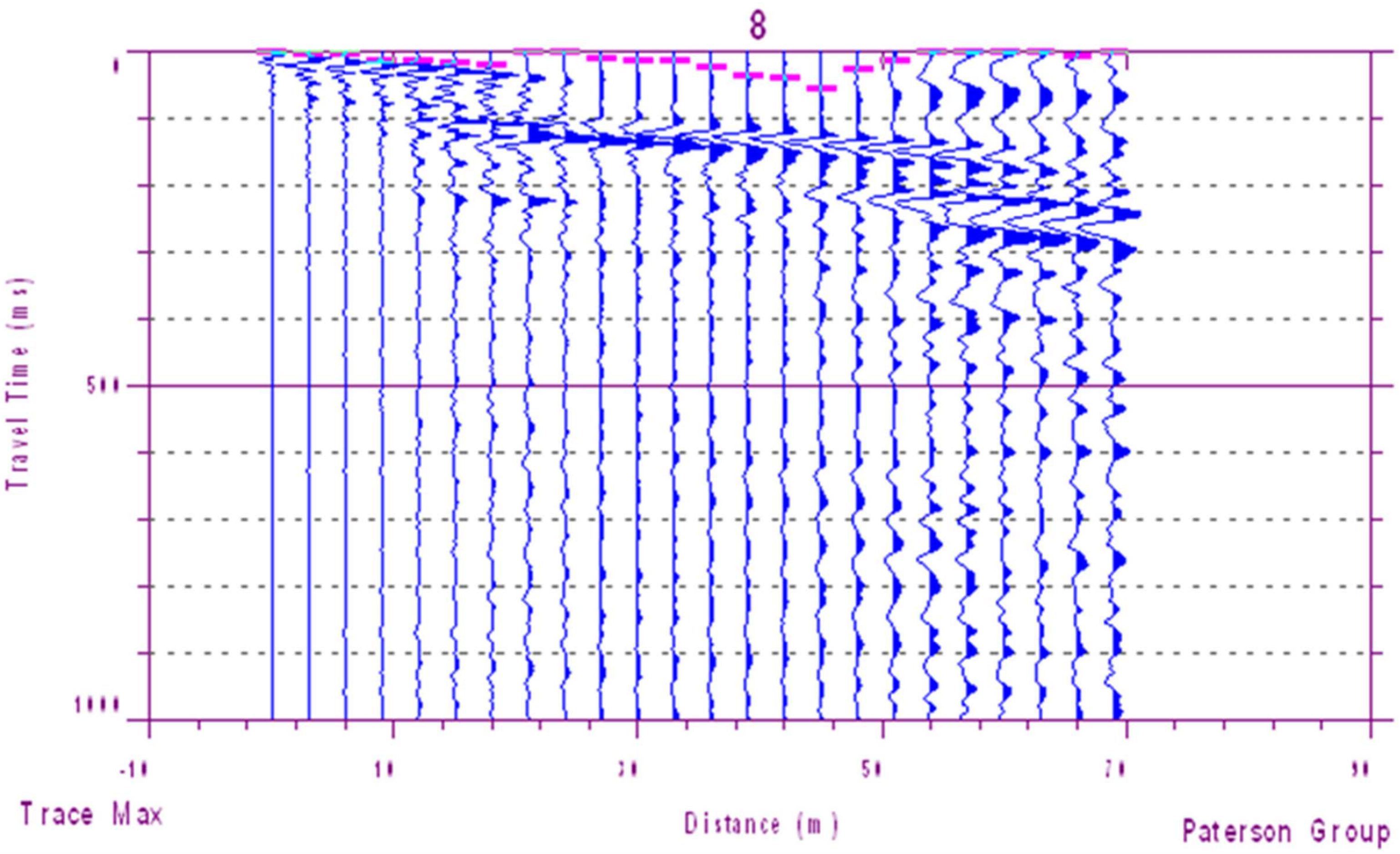


Figure 2 – Shear Wave Velocity Profile at Shot Location -3.0 m

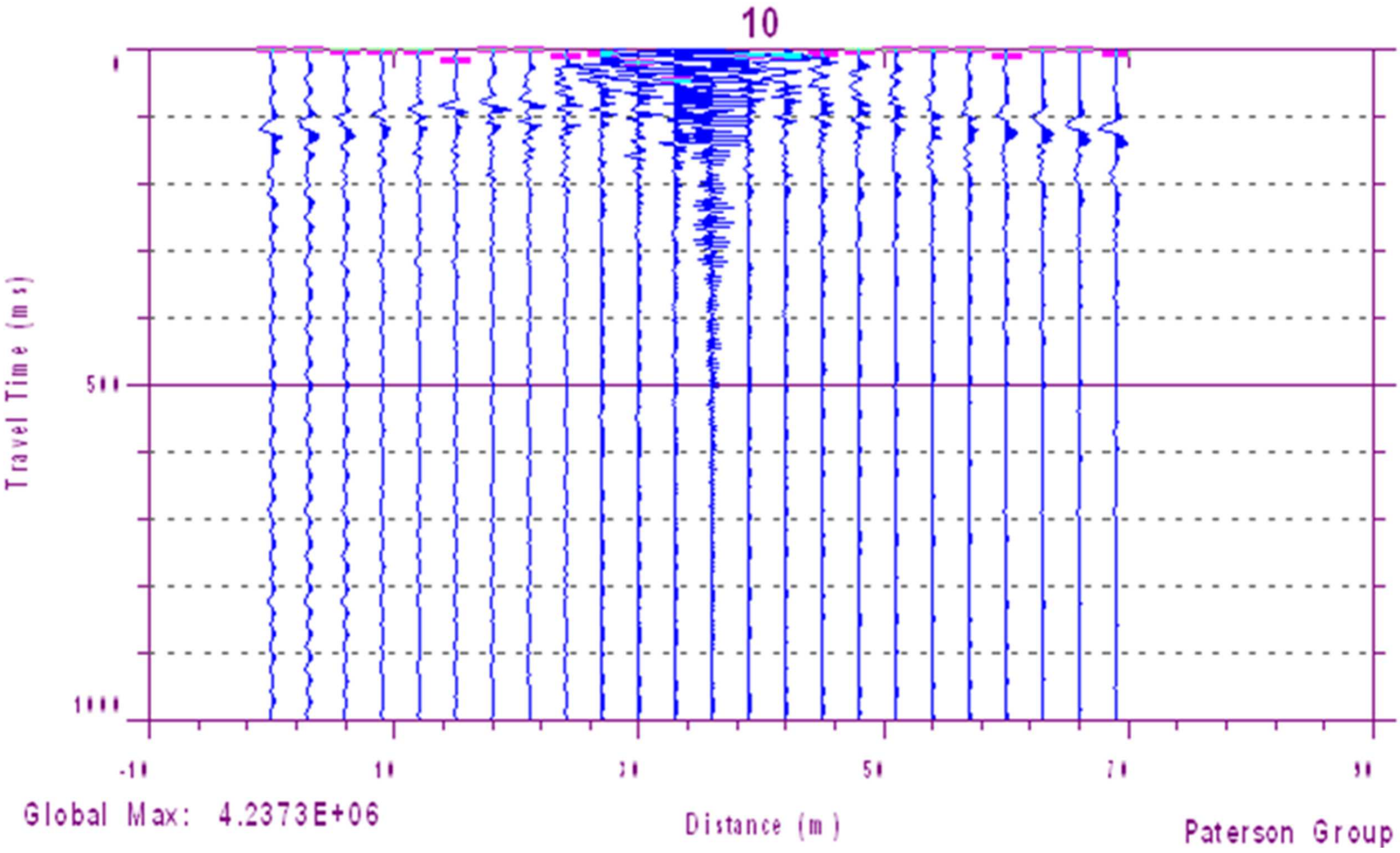
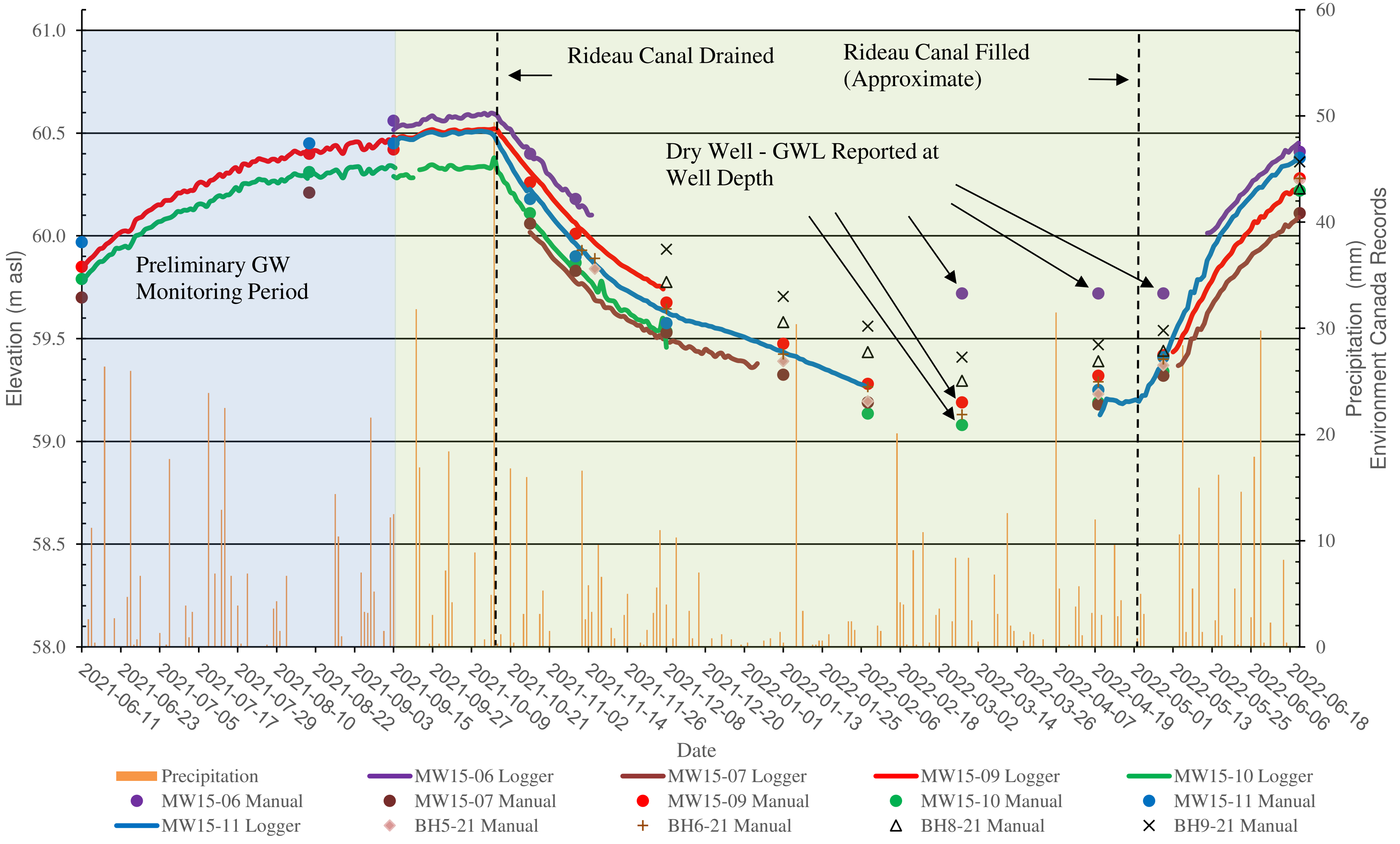
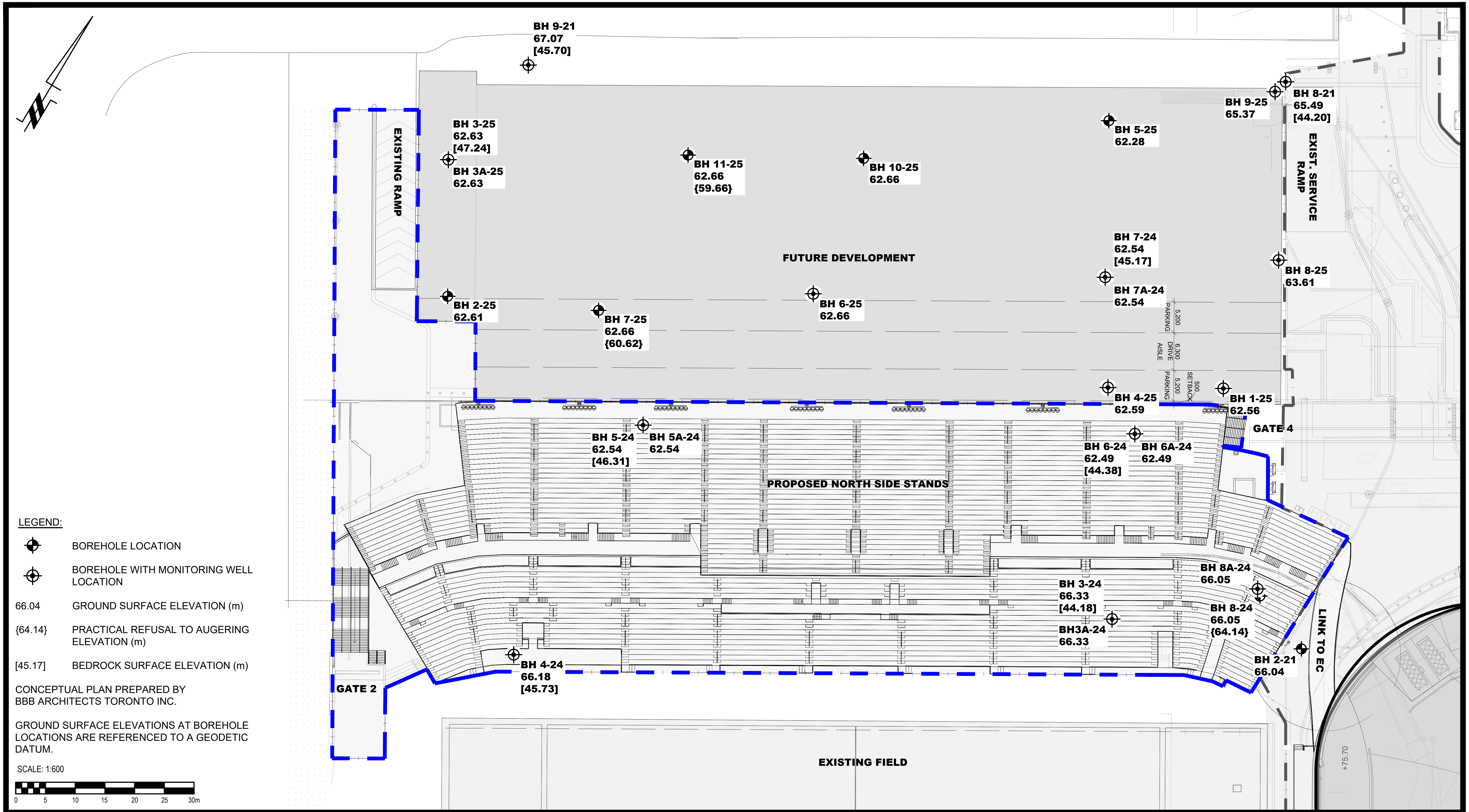


Figure 3 – Shear Wave Velocity Profile at Shot Location 34.5 m

Figure 4 : Groundwater Elevation Monitoring - Program Update



NOTE: THIS FIGURE HAS BEEN APPENDED TO PATERSON REPORT PG6655-2 DATED NOVEMBER 22, 2024, HOWEVER, THIS IS AN EXCERPT FROM A HYDROGEOLOGICAL REPORT PREPARED BY PATERSON AND SHOULD BE REFERENCED UNDER THE SEPERATE COVER.



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
66.04	GROUND SURFACE ELEVATION (m)
{64.14}	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
[45.17]	BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PREPARED BY
BBB ARCHITECTS TORONTO INC.

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

SCALE: 1:600

0 5 10 15 20 25 30m

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

NO.	REVISIONS	DATE	INITIAL
2	ADDED 2025 BOREHOLE LOCATIONS	18/03/2025	FC
1	UPDATED CONCEPTUAL PLAN AND SITE BOUNDARY	17/12/2024	FC

CITY OF OTTAWA
GEOTECHNICAL INVESTIGATION
LANDSDOWNE PARK REDEVELOPMENT
PROPOSED NORTH SIDE STANDS

OTTAWA, ONTARIO

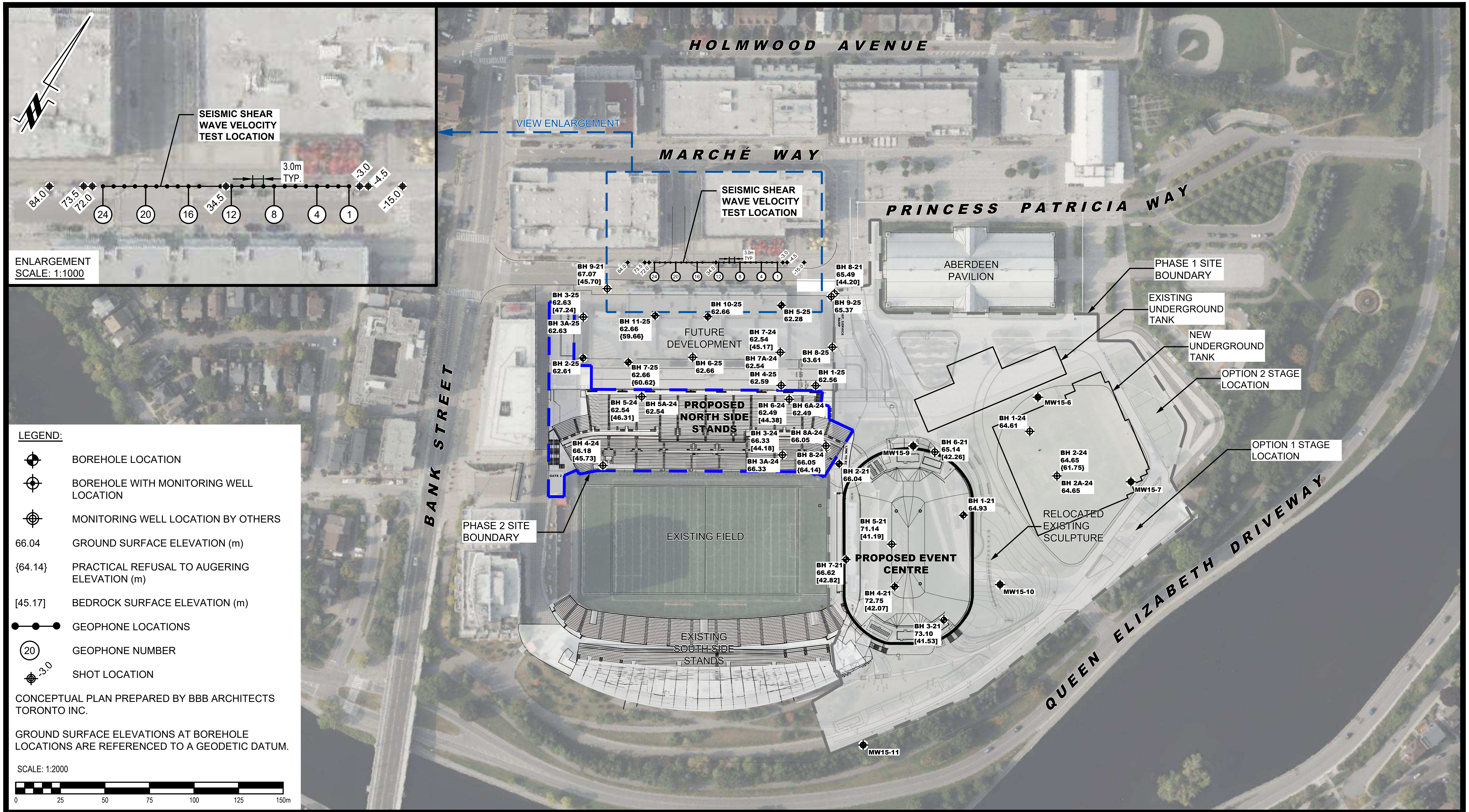
Title:

TEST HOLE LOCATION PLAN

Scale: 1:600
Date: 11/2024
Drawn by: ZS
Report No.: PG6655-1
Checked by: FC
Dwg. No.: PG6655-2
Approved by: DP
Revision No.: 2

p:\autocad drawings\geotechnical\pg66xx\pg6655\pg6655-2-test hole location plan (rev.02).dwg

11x17



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	MONITORING WELL LOCATION BY OTHERS
66.04	GROUND SURFACE ELEVATION (m)
{64.14}	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
[45.17]	BEDROCK SURFACE ELEVATION (m)
●—●—●	GEOPHONE LOCATIONS
⑳	GEOPHONE NUMBER
⊕ -3.0	SHOT LOCATION

CONCEPTUAL PLAN PREPARED BY BBB ARCHITECTS TORONTO INC.

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
2	ADDED 2025 BOREHOLE LOCATIONS, UPDATED PHASE I CONCEPTUAL PLAN AND UPDATED SITE BOUNDARY	18/03/2025	FC
1	UPDATED CONCEPTUAL PLAN AND SITE BOUNDARY	17/12/2024	FC

CITY OF OTTAWA
GEOTECHNICAL INVESTIGATION
LANDSDOWNE PARK REDEVELOPMENT
PROPOSED NORTH SIDE STANDS

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2000
Date: 11/2024
Drawn by: ZS
Report No.: PG6655-1
Checked by: FC
Dwg. No.: PG6655-2A
Approved by: DP
Revision No.: 2

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