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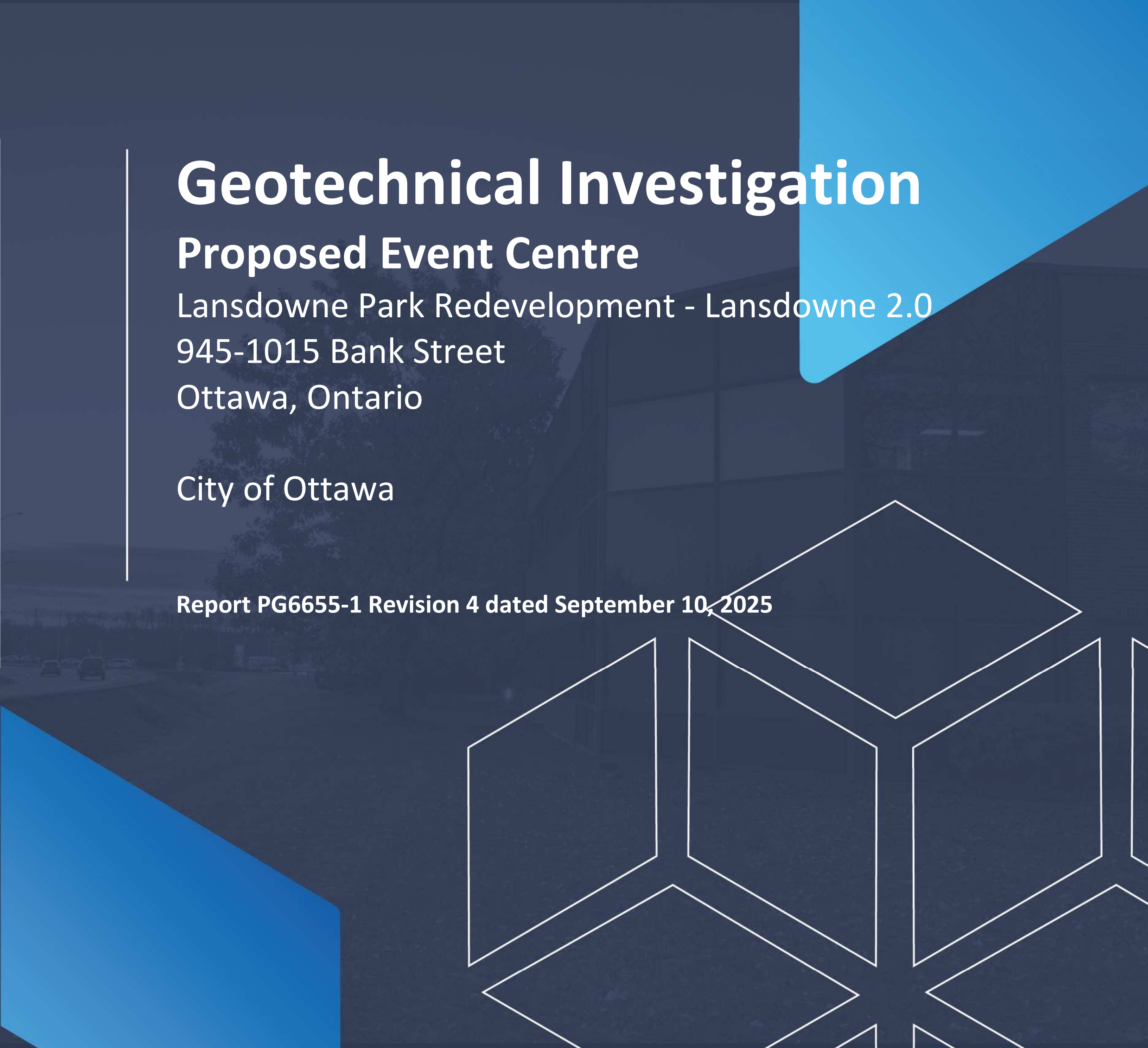

Geotechnical Investigation

Proposed Event Centre

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

City of Ottawa

Report PG6655-1 Revision 4 dated September 10, 2025

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1.0 Introduction

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

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of boreholes.

- Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

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

2.0 Proposed Development

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

The exterior of the arena facility will be surrounded by patios, terraced landscaped areas and hardscaping (i.e., paver and/or paved pathways), terraced seating and public art features. It is also understood that an open “Great Lawn” landscaped area will be located to the east of the arena and separated from the arena by an approximately 5.5 m high berm along within the landscaped area. Further, an underground prefabricated watertight stormwater tank system will be included as part of the proposed project.

It is expected that the proposed building will be municipally serviced. Further, existing infrastructure will be demolished in support of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

A field investigation program was completed at the subject site by Paterson from October 25 to November 17, 2021. At that time, a total of eight (8) boreholes were advanced to a maximum depth of 33.4 m below existing grade. A supplemental field investigation was completed by this firm on October 9, 2024 and consisted of advancing two (2) boreholes to a maximum depth of 6.7 m below the existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features.

A previous geotechnical investigation was completed by others on October 21 and October 22, 2015. At the time, the investigation consisted of advancing a total of five (5) boreholes to a maximum depth of 6.1 m below ground surface. The borehole locations of the current and previous investigations are shown on Drawing PG6655-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

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

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

Diamond drilling was completed at boreholes BH 3-21, BH 4-21, BH 5-21, BH 6-21, BH 7-21 and BH 8-21 to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage.

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

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

Groundwater

Boreholes BH 1-24, BH 2A-24, BH 5-21, BH 6-21 and BH 8-21 were fitted with PVC groundwater monitoring wells. The remaining boreholes were fitted with flexible polyethylene standpipes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- Slotted PVC screen at the base of each borehole.
- 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 for the current investigation were selected by Paterson personnel in a manner to provide general coverage of the proposed development, taking into consideration existing site features. The borehole locations and ground surface elevations were referenced to a geodetic datum. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG6655-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

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

3.4 Analytical Testing

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

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 located southeast of TD Place stadium and south of the Aberdeen Pavilion within Lansdowne Park Development. Currently, the location of the proposed event centre is landscaped, and grass covered. There is an approximately 9 m high grass covered berm throughout the southwestern portion of the subject site and throughout the footprint of the proposed arena footprint. The remainder of the subject site is relatively flat and either grass-covered or landscaped with pavers and associated hardscaped access lanes and walking paths.

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

4.2 Subsurface Profile

Overburden

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

The fill material was observed to generally consist of brown silty sand to sandy silt with varying amounts of crushed stone, gravel, cobble, boulders, clay and topsoil. Trace amounts of asphaltic concrete were also observed at the location of BH 3-21, BH 4-21 and BH 5-21. Based on the encountered fill thicknesses, the native, in-situ, undisturbed soils were encountered at approximate geodetic elevations between 60.1 to 64.7m.

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

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

Bedrock

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

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

4.3 Groundwater

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 1 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.

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.

Table 1 – Groundwater Elevation Summary

Test Hole	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
MW 15-6	64.90	4.16	60.74	August 16, 2022
		Dry	NA	March 9, 2022 April 30, 2022 May 10, 2022
MW 15-7	64.51	4.09	60.42	August 18, 2022
		5.33	59.18	April 30, 2022
MW 15-9	65.25	4.65	60.60	August 16, 2022
		Dry	NA	March 9, 2022
MW 15-10	64.91	4.37	60.57	August 17, 2022
		Dry	NA	March 9, 2022
MW 15-11	64.57	3.90	60.67	Sept. 22, 2022
		5.45	59.12	Nov. 20, 2021
BH 1-21	64.93	5.09	59.84	Nov. 12, 2021
BH 3-21	73.10	13.46	59.64	Nov. 16, 2021
BH 4-21	72.75	10.51	62.24	Nov. 16, 2021
BH 5-21	71.14	11.30	59.84	Nov. 16, 2021
BH 6-21	65.14	5.25	59.89	Nov. 16, 2021
BH 7-21	66.62	Dry	NA	Nov. 16, 2021
BH 8-21	65.45	4.85	60.60	Sept. 26, 2022

Note: The ground surface elevation at each borehole location was surveyed by Paterson using a handheld GPS and was referenced to a geodetic datum

4.4 Hydraulic Conductivity Testing Results

Hydraulic conductivity (slug) tests were conducted at three (3) monitoring well locations on November 16 and December 8, 2021, to evaluate the hydraulic properties of the overburden material at the test locations.

The measured hydraulic conductivity (K) values ranged between approximately 7.75 x 10^{-5} to 2.31 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 satisfactory for the construction of the proposed Event Centre. 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 (such as logs, stumps, peat and other highly organic material), should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

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

Reference should be made to memo report PG6655-MEMO.08 Revision 4 dated July 28, 2025, included in Appendix 3 for detail recommendations in relation to the fill material encountered at the subgrade depth for the proposed stormwater tank system.

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. Paterson personnel should review and approve all bearing surfaces prior to backfilling.

Fill Placement

Building Area

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Compaction of these materials should be complete using a suitably sized sheepsfoot roller making a suitable number of passes and under the supervision of Paterson field personnel. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as CCW MiraDRAIN 2000 or Delta-Teraxx, connected to a perimeter drainage system.

Landscape Berm

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

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

Underground Stormwater Tank System

A summary of the fill structure to be used at the proposed stormwater tank system is provided in memo report PG6655-MEMO.08 Revision 4 dated July 28, 2025, included in Appendix 3 of this report.

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.

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, compact silty sand and/or very dense to compact 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.

The proposed underground stormwater tank system is expected to be founded on the existing compact fill material consisting of silty sand with trace amounts of gravel. Based on the above, a bearing resistance value for the proposed structure may be considered to be **120 kPa** (SLS) and a factored bearing resistance value at ULS of **180 kPa** may be considered for the system and associated infrastructure/structures.

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**.

Where footings are 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**.

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.

Settlement

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

Lateral Support

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

Proof Rolling and Subgrade Improvement

Loose Sand Below Footings

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

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

Depending on the looseness and degree of saturation of loose sandy soils at the time of construction, other measures (additional compaction, sub-excavation and reinstatement of crushed stone fill, mud slab) may be recommended to accommodate site conditions at the time of construction.

However, these considerations would be evaluated at the time of design by Paterson on a footing-specific basis

Existing Fill below Stormwater Tank System

It is recommended that the existing fill encountered below the underground stormwater tank system be proof-rolled under dry conditions and above freezing temperatures by an adequately sized sheepsfoot roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by Paterson personnel at the time of construction.

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

Reference should be made to memo report PG6655-MEMO.08 Revision 4 dated July 28, 2025, included in Appendix 3 for detail recommendations in relation to the existing fill encountered below the stormwater tank system.

Deep Foundations – Drilled Shaft Caissons and End-Bearing Piles

Where required, drilled shafts and 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 2 below presents the estimated capacity for different typical caisson sizes founded within and upon an in-situ, dense glacial till bearing surface.

Table 2 – 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.

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.

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

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

Table 3 - End Bearing Pile Foundation 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 4 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 4 - 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

Seismic Shear Wave Velocity Testing

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed structures as per the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave interpretation are presented in Figure 2 and Figure 3 in Appendix 2 of the present report.

Field Program

The shear wave testing was located along Exhibit Way, as presented in Drawing PG6655-1A - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly an east-west orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio.

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the centre of the geophone array and 1.6, 3.1 and 9 m away from the first and last geophone.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods.

The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile.

The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The depth to bedrock is known to vary across the site, therefore a conservative estimate of 22 m below ground surface was used for calculation of the V_{s30}.

Overall, the average shear wave velocity through the overburden materials was interpreted to be **387 m/s**. Under normal circumstances, the bedrock velocity is interpreted using the main refractor wave velocity, however, this particular test did not provide sufficiently accurate readings to determine a bedrock velocity. In its place, Paterson has assumed a conservative bedrock velocity of **1,500 m/s**.

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

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

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

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

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

5.5 Basement Slab Construction

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 at 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 buildings sump pit, or 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.

Rink Slab Recommendations

Paterson understands the rink slab design will be undertaken by a specialized design contractor. At this time, Paterson anticipates the slab will be underlain by a layer of engineered fill, such as OPSS Granular A, and a thermal break layer to protect the underlying subsoils from freeze-thaw action.

The engineered fill layer is recommended to be a minimum of 450 mm thick, placed in 300 mm maximum thick loose lifts, compacted to a minimum of 99% of the materials SPMDD, be please upon native, in-situ, undisturbed dense silty sand to glacial till soil surfaces and be reviewed and approved by Paterson personnel at the time of construction.

A preliminary subgrade modulus of **20 MPa/m** may be considered by the specialized design contractor. However, it is recommended that the design be reviewed and coordinated with Paterson during the design phase once detailed design information is known and may be adapted to the project-specific design that may be undertaken at that time.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 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

Flexible Pavement Design – At-Grade Areas

The flexible pavement structure presented in Table 5 and Table 6 should be used for at grade access lanes and heavy loading parking areas.

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).

Table 5 – 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 6 – 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.	

Where pavement structures overlie the underground stormwater storage system, it is recommended that Paterson review the associated tie-ins and details for constructing the pavement structure over the system from a geotechnical perspective.

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

Paterson understands a shoring system consisting of a combination of secant pile walls and secant piles with timber lagging are being considered for construction at the subject site. Design parameters associated with the proposed system have been provided in Section 6.3 for use by the shoring design engineer. Where lateral resistance is insufficient for the proposed system, rock or soil anchors can be utilized to provide additional lateral resistance to the proposed shoring system.

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 cementitious 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 7 – 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 8.

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 8 – 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 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.

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 of 2.1 m thick soil cover (or insulation equivalent).

Unheated structures 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.

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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 is anticipated to be implemented to support the overburden soil to complete the required excavations for site servicing and foundation construction works.

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 9 – 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

If the excavation for the proposed building is to extend within the lateral support zone of adjacent building foundations, underpinning of these structures would be required. The depth of the underpinning, if required, would be dependent on the depth of the neighbouring foundations relative to the founding depth of the proposed building at the subject site.

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. 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 of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil.

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 an aggressive to very aggressive corrosive environment.

6.8 Slope Stability Assessment

Slope Conditions

Based on the available plans and drawings, it is understood that a berm and associated slope has been proposed as part of the landscaping at the subject site.

The berm and slope are understood to be located east of the Event Centre and west of the Great Lawn also proposed throughout the subject site, and as indicated on Drawing PG6655-1 - Test Hole Location Plan, included in Appendix 2 of the present geotechnical report.

As part of the current investigation, Paterson completed a slope stability analysis of the proposed conditions to evaluate the stability of the slope taking into consideration existing and proposed features, and as described in the following sections. One (1) cross-section was studied as the worst-case scenario (i.e., steepest topographic relief and steepest slope inclination).

The location of the cross-section is presented on Drawing PG6655-1 - Test Hole Location Plan, included in Appendix 2 of the present geotechnical report.

Slope Stability Analysis

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

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

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16 g was considered for the cross-section for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading. One (1) slope cross-section was analyzed based on proposed conditions under static and seismic loading. Subsoil conditions at the cross-section were inferred based on the findings of the geotechnical investigation and borehole information. The cross-section location is presented on Drawing PG6655-1 - Test Hole Location Plan in Appendix 2.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation. The effective strength soil parameters used for static analysis are presented in Table 10.

Table 10 – Effective Soil and Material Parameters (Static Analysis)

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

The total strength parameters for seismic analysis were chosen based on the subsurface conditions observed in the test holes, and our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 11.

Table 11 – Total Stress Soil and Material Parameters (Seismic Analysis)

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

Static Loading Analysis

The results for the static analysis under proposed conditions are presented in Figure 5A included in Appendix 2. The results indicate that the slope stability factor of safety was found to be greater than 1.5 for slope section A. Therefore, the proposed slope is considered stable under static loading.

Seismic Loading Analysis

The results of the analyses considering seismic loading are presented in Figure 5B in Appendix 2. The slope stability factor of safety was found to be greater than 1.1 for slope section A. Based on these results, the proposed slope is considered stable under seismic loading. Therefore, a stable slope allowance setback is not required from a geotechnical perspective.

Conclusion

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

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

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

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

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

6.9 Landscaping Considerations

Retaining Walls

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

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

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

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

The soil parameters presented in Table 12 can be used in the design of the retaining walls.

Table 12 – Soil parameters for global stability analysis

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

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

Backfill Materials

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

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

Geotechnical parameters of the proposed free-draining backfill material to be used at the subject site are provided in Table 13 for design purposes.

Table 13 – Geotechnical parameters for backfill material

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

Notes:

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

Lateral Earth Pressure

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

Tree Planting Considerations

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

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

7.0 Recommendations

It is recommended for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant:

- 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.
- Inspection of the installation of the geotextile liners, Stormtech tanks and associated fill layers.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Review of the earthworks program associated with the proposed berm.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than City of Ottawa or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Fernanda Carrozzi, PhD. Geoph.

Drew Petahtegoose, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Sept 10, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

PHOTOGRAPHS OF ROCK CORE

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368851.35 **NORTHING:** 5029165.32 **ELEVATION:** 64.61

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

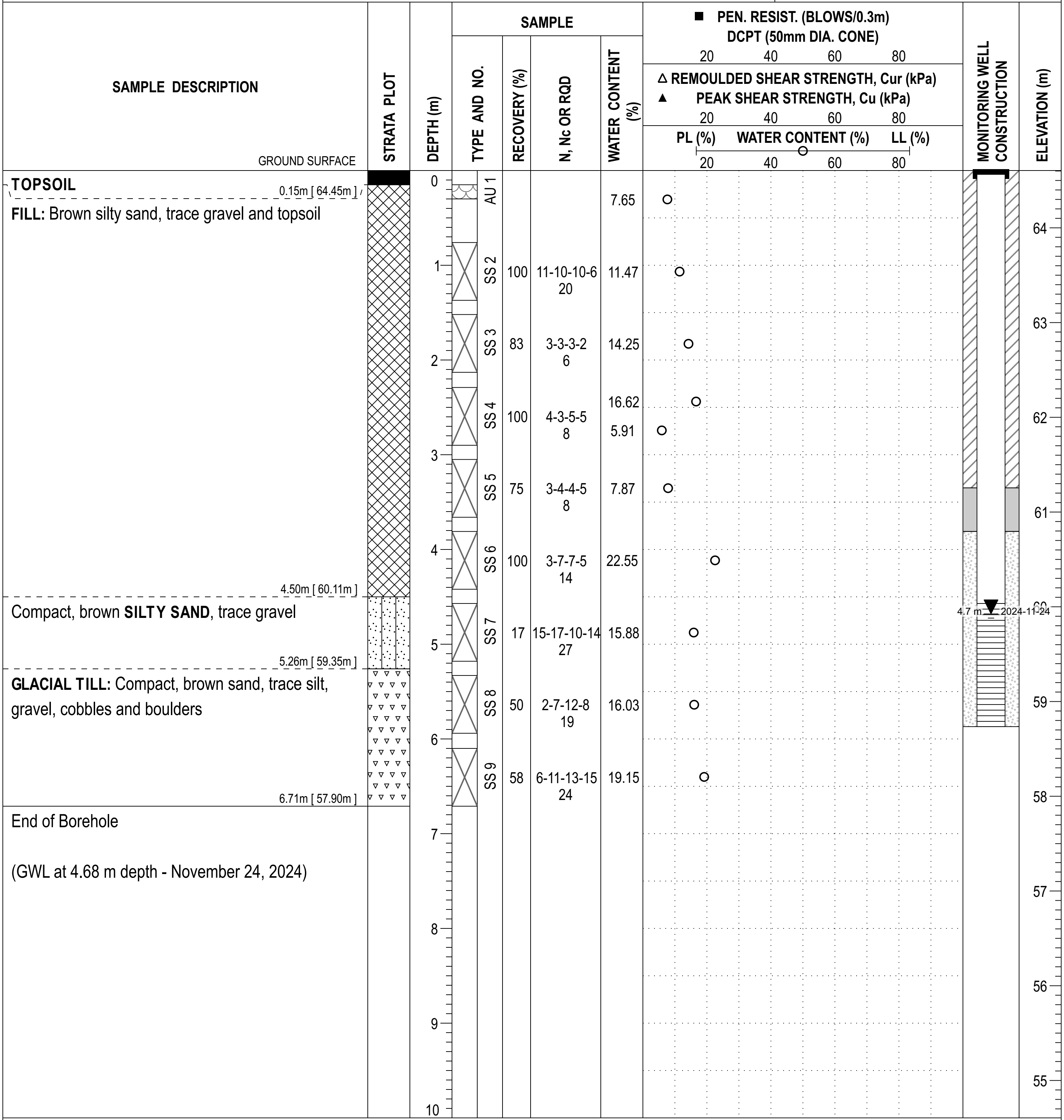
DATE: October 09, 2024

FILE NO. : PG6655

HOLE NO. : BH 1-24

SAMPLE DESCRIPTION	Bottom depth [elevation]
GROUND SURFACE	
TOPSOIL	0.15m [64.45m]
FILL: Brown silty sand, trace gravel and topsoil	4.50m [60.11m]
Compact, brown **SILTY SAND**, trace gravel	5.26m [59.35m]
GLACIAL TILL: Compact, brown sand, trace silt, gravel, cobbles and boulders	6.71m [57.90m]
End of Borehole (GWL at 4.68 m depth - November 24, 2024)	

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			7.65
SS 2	100	11-10-10-6 20	11.47
SS 3	83	3-3-3-2 6	14.25
SS 4	100	4-3-5-5 8	16.62 5.91
SS 5	75	3-4-4-5 8	7.87
SS 6	100	3-7-7-5 14	22.55
SS 7	17	15-17-10-14 27	15.88
SS 8	50	2-7-12-8 19	16.03
SS 9	58	6-11-13-15 24	19.15



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368877.23 **NORTHING:** 5029151.57 **ELEVATION:** 64.65

PROJECT: Proposed North Stands

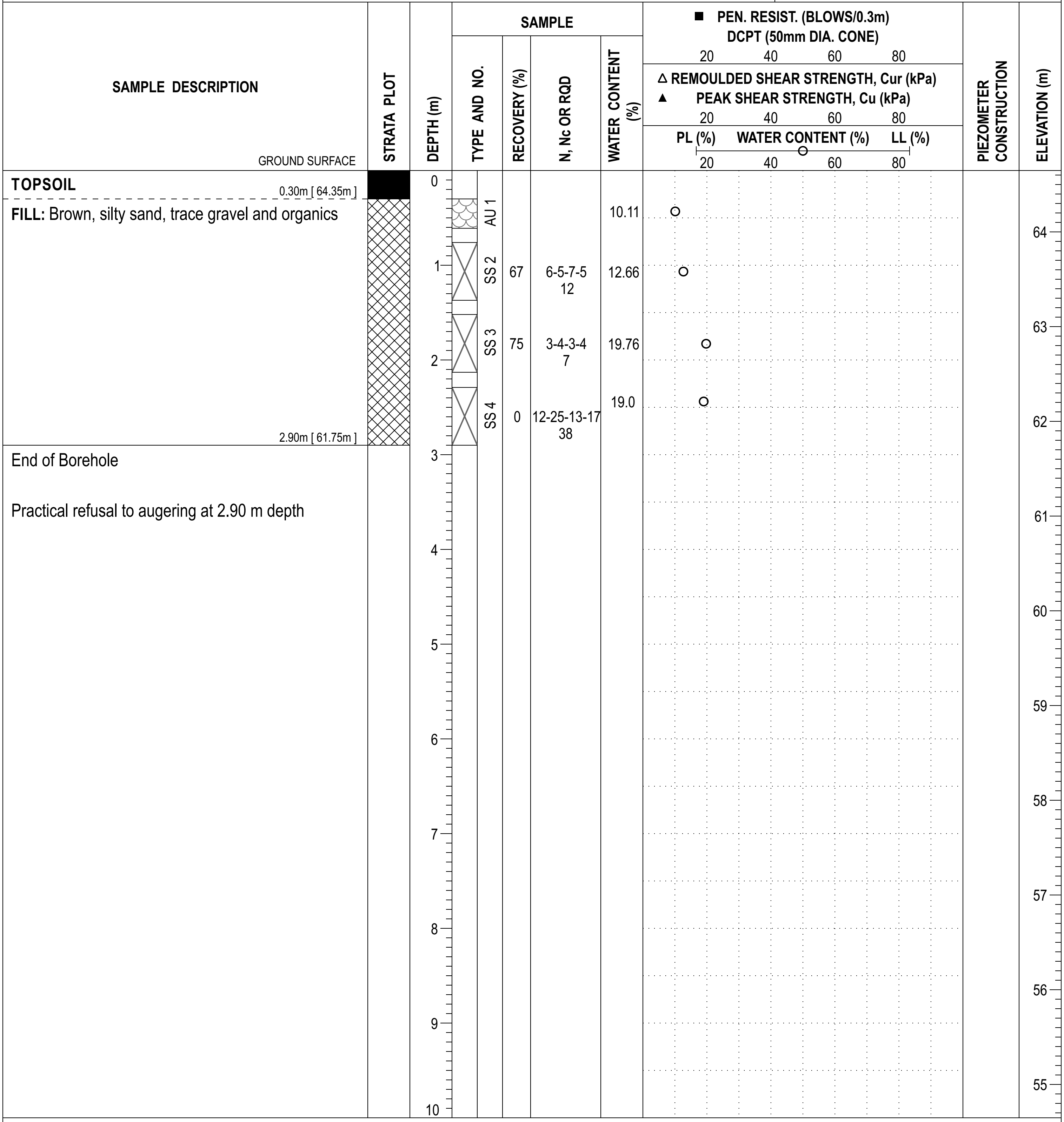
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: October 09, 2024

FILE NO. : PG6655

HOLE NO. : BH 2-24



TOPSOIL 0.30m [64.35m]

FILL: Brown, silty sand, trace gravel and organics 2.90m [61.75m]

End of Borehole

Practical refusal to augering at 2.90 m depth

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			10.11
SS 2	67	6-5-7-5 12	12.66
SS 3	75	3-4-3-4 7	19.76
SS 4	0	12-25-13-17 38	19.0

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368879.56 **NORTHING:** 5029150.20 **ELEVATION:** 64.65

PROJECT: Proposed North Stands

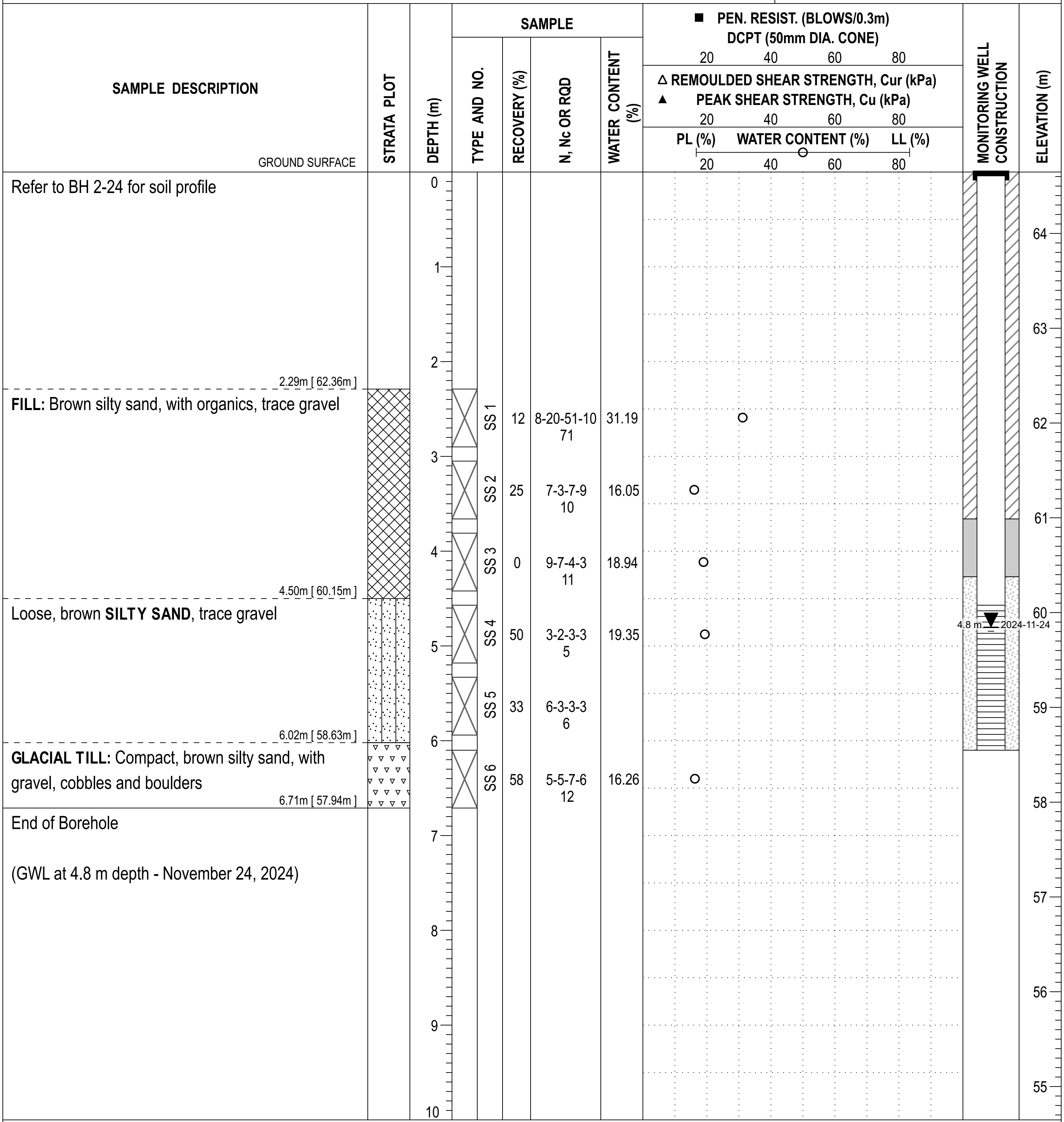
BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: October 09, 2024

FILE NO. : PG6655

HOLE NO. : BH 2A-24



SAMPLE DESCRIPTION	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE				
Refer to BH 2-24 for soil profile				
2.29m [62.36m]				
FILL: Brown silty sand, with organics, trace gravel	SS 1	12	8-20-51-10 71	31.19
	SS 2	25	7-3-7-9 10	16.05
	SS 3	0	9-7-4-3 11	18.94
4.50m [60.15m]				
Loose, brown **SILTY SAND**, trace gravel	SS 4	50	3-2-3-3 5	19.35
	SS 5	33	6-3-3-3 6	
6.02m [58.63m]				
GLACIAL TILL: Compact, brown silty sand, with gravel, cobbles and boulders	SS 6	58	5-5-7-6 12	16.26
6.71m [57.94m]				
End of Borehole				
(GWL at 4.8 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:** 368758.14 **NORTHING:** 5029099.16 **ELEVATION:** 66.05

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

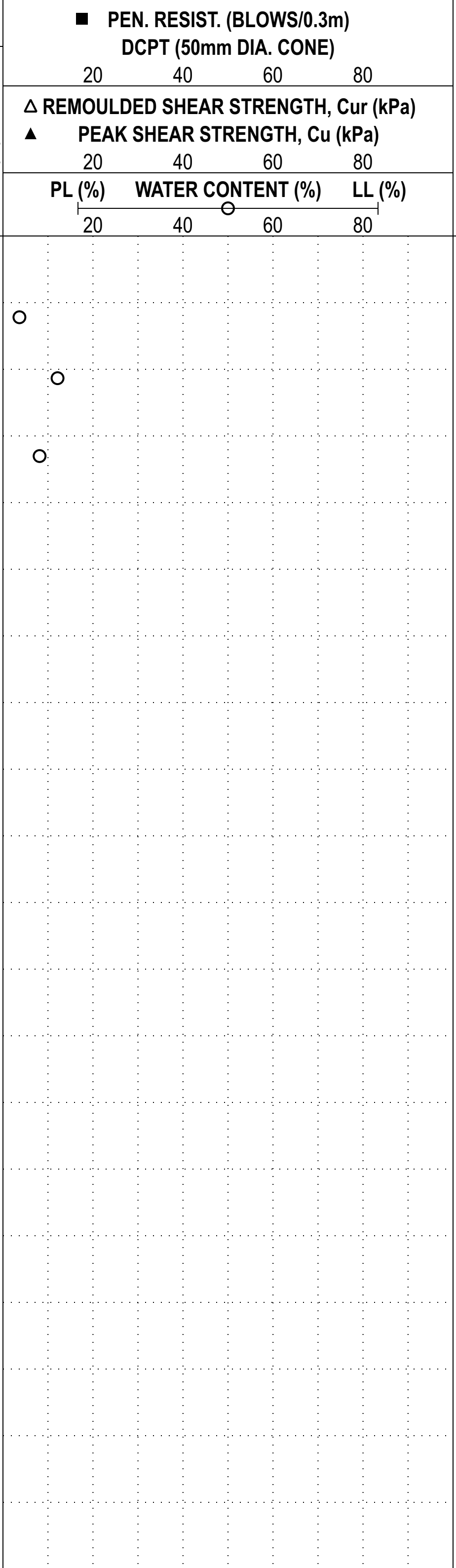
REMARKS: Borehole Drilled Indoors

DATE: November 01, 2024

FILE NO. : PG6655

HOLE NO. : BH 8-24

SAMPLE DESCRIPTION GROUND SURFACE	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	PIEZOMETER CONSTRUCTION	ELEVATION (m)
ASPHALT 0.06m [65.99m]		0						66
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		2						64
Practical refusal to augering at 1.91 m depth								
		3						63
		4						62
		5						61
		6						60
		7						59
		8						58
		9						57
		10						



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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

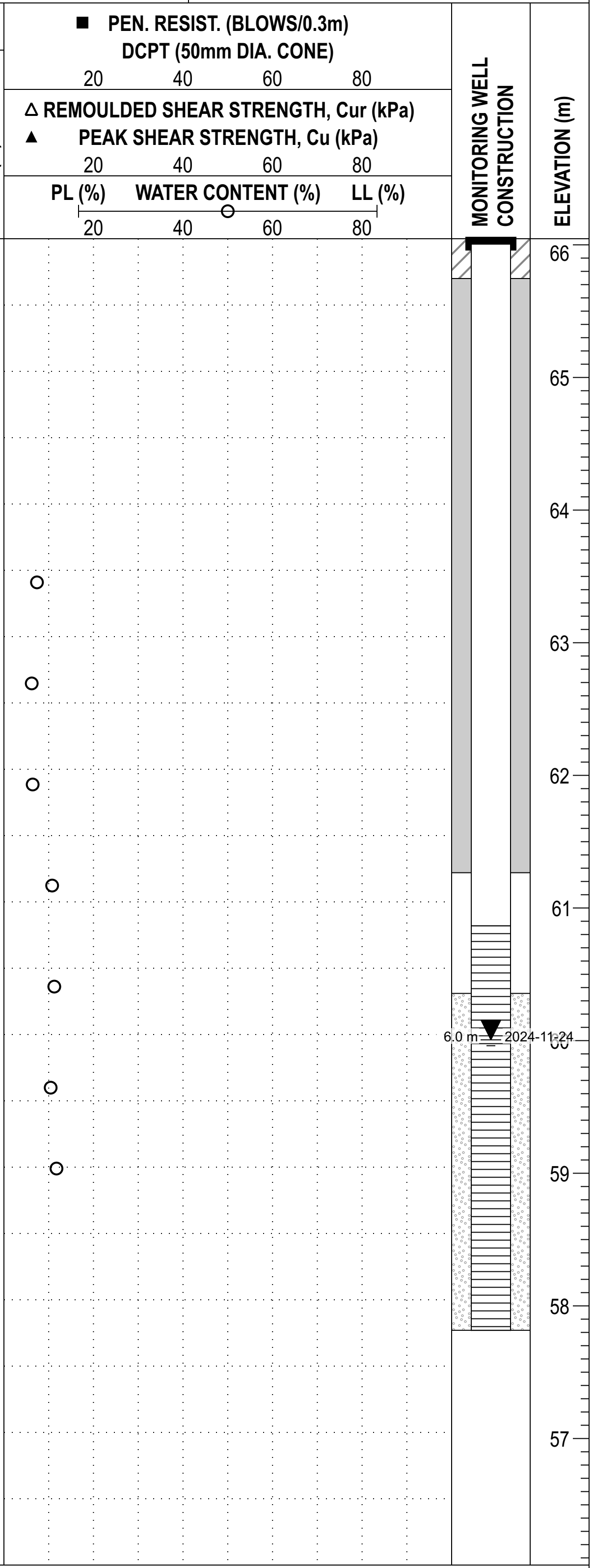
REMARKS: Borehole Drilled Indoors

DATE: November 01, 2024

FILE NO. : **PG6655**

HOLE NO. : **BH8A-24**

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE	0				
Refer to BH 8-24 for soil profile					
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

REMARKS

BORINGS BY CME-55 Low Clearance Drill

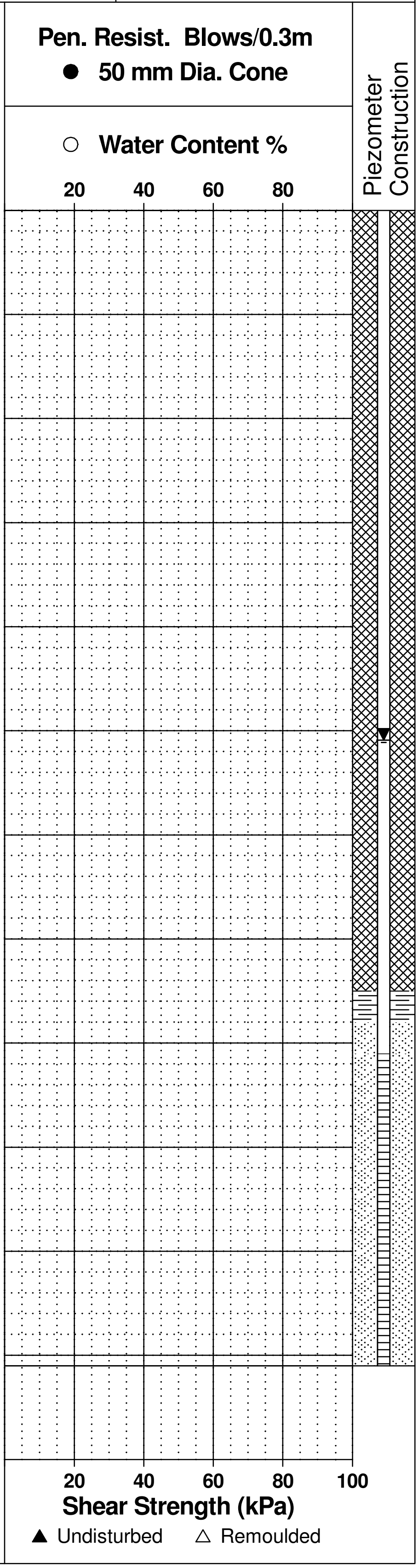
DATE October 25, 2021

FILE NO. PG5792

HOLE NO. BH 1-21

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

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



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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 25, 2021

FILE NO. PG5792

HOLE NO. BH 2-21

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

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

○ **Water Content %**

20 40 60 80

Piezometer Construction

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

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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE October 27, 2021

FILE NO. PG5792

HOLE NO. BH 3-21

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

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

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

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

○ Water Content %
20 40 60 80

Piezometer Construction

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

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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

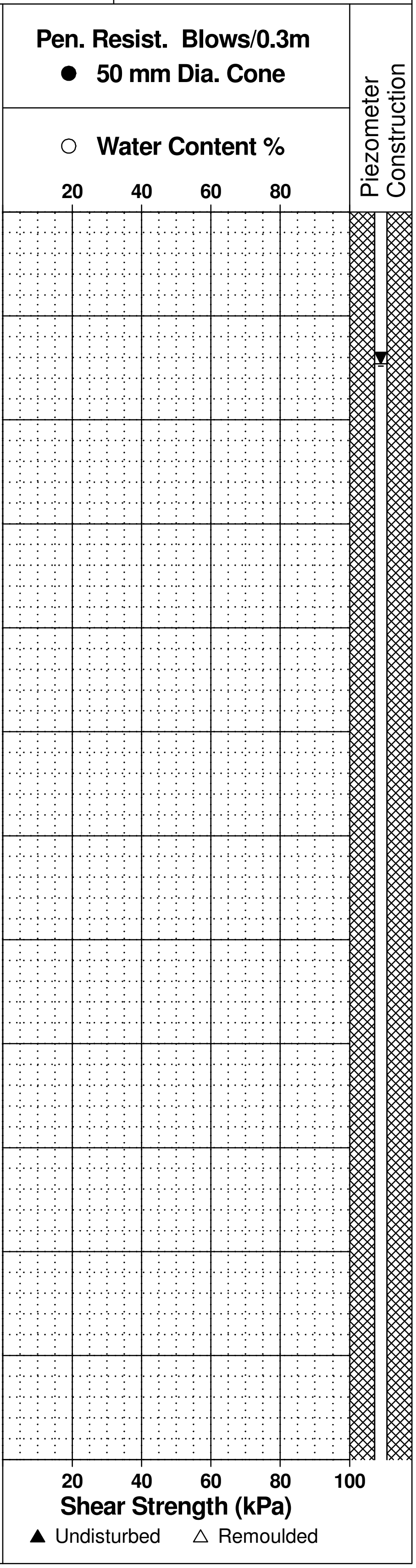
BORINGS BY CME-55 Low Clearance Drill

DATE October 27, 2021

FILE NO. PG5792

HOLE NO. BH 3-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
Compact, brown **SILTY SAND,** some gravel					12	61.10
	SS	17	33	19		
					13	60.10
	SS	18	25	18		
	SS	19	4	12		
					14	59.10
	SS	20	4	21		
					15	58.10
15.54						
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders - grey by 20.2m depth - compact by 21.3m depth	SS	21	50	36		
					16	57.10
	SS	22	67	60		
	SS	23	33	50+		
	RC	2	70			
					17	56.10
	SS	24	4	50+		
					18	55.10
					19	54.10
	RC	3	64			
					20	53.10
	RC	4	52			
					21	52.10
	RC	5	30			
					22	51.10
					23	50.10
	RC	6	13			
					24	49.10



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

FILE NO. PG5792

REMARKS

HOLE NO. BH 3-21

BORINGS BY CME-55 Low Clearance Drill

DATE October 27, 2021

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders - cobbles and boulders content decreasing with depth	RC	7	8	
	RC	8	0	
	RC	9	0	
	RC	10	0	
31.57	RC	11	100	71
BEDROCK: Good to excellent quality, grey limestone with occasional shale partings	RC	12	100	98
33.45				

End of Borehole

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

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

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

○ **Water Content %**

20 40 60 80

Piezometer Construction

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

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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

FILE NO. PG5792

REMARKS

HOLE NO. BH 4-21

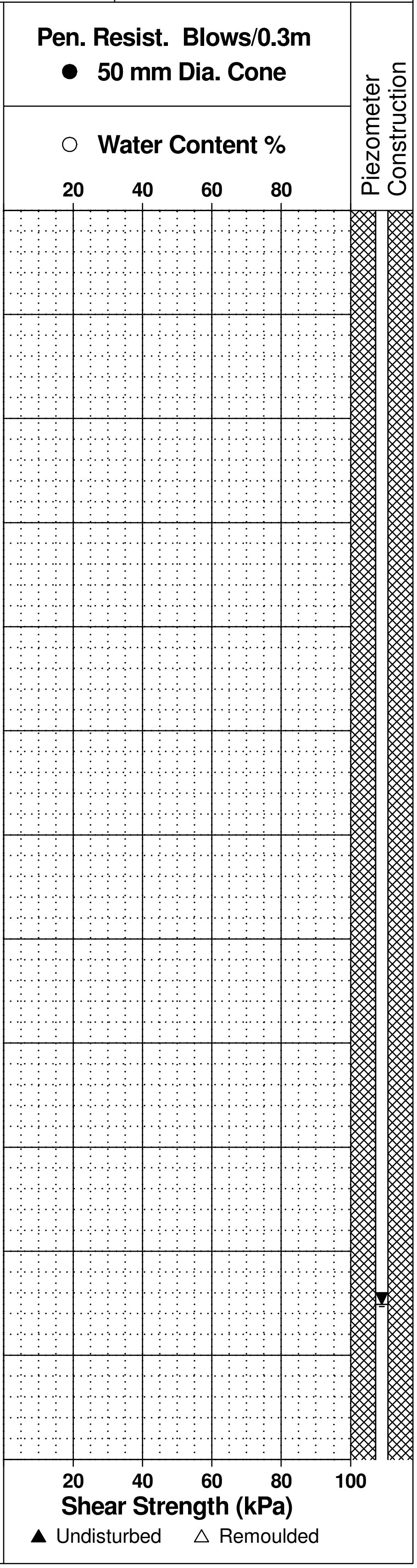
BORINGS BY CME-55 Low Clearance Drill

DATE November 5, 2021

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.30
FILL: Brown silty sand iwth gravel and cobbles, occasional boulders, trace clay - some topsoil from 5.3 to 5.9m depth - some asphaltic concrete from 7.6 to 8.2m depth	8.53
Compact, brown **SILTY SAND to SANDY SILT**	11.25
GLACIAL TILL: Very dense to dense, silty sand with gravel, cobbles and boulders	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	33	5
SS	3	58	49
SS	4	50	10
SS	5	50	8
SS	6	50	8
SS	7	42	46
SS	8	33	28
SS	9	50	19
SS	10	18	9
SS	11		50+
SS	12	58	13
SS	13		14
SS	14	42	19
SS	15	50	18
SS	16	33	59

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



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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

FILE NO. PG5792

REMARKS

HOLE NO. BH 4-21

BORINGS BY CME-55 Low Clearance Drill

DATE November 5, 2021

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
GLACIAL TILL: Very dense to dense, silty sand with gravel, cobbles and boulders	SS	17	60	50+
	RC	1	33	
	RC	2	41	
	SS	18	75	50+
	RC	3	34	
	RC	4	24	
	SS	19	0	50+
	RC	5	7	
- grey by 20.8m depth	SS	20	42	15
	RC	6	0	
	SS	21	0	50+
	RC	7	20	

DEPTH (m)	ELEV. (m)
12	60.75
13	59.75
14	58.75
15	57.75
16	56.75
17	55.75
18	54.75
19	53.75
20	52.75
21	51.75
22	50.75
23	49.75
24	48.75

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

FILE NO. PG5792

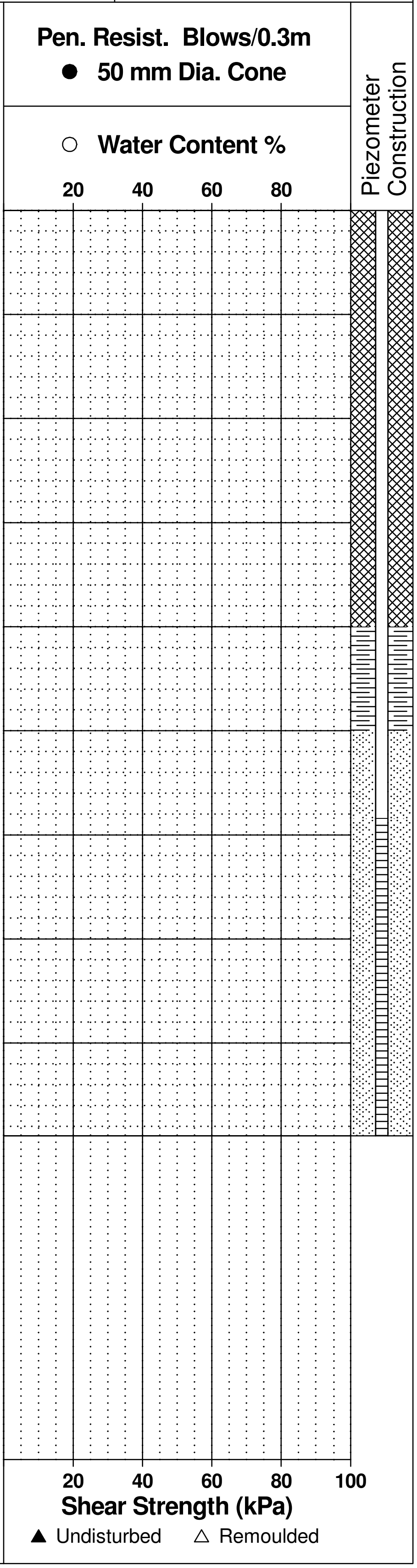
REMARKS

HOLE NO. BH 4-21

BORINGS BY CME-55 Low Clearance Drill

DATE November 5, 2021

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



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Redevelopment
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DATUM Geodetic

FILE NO. **PG5792**

REMARKS

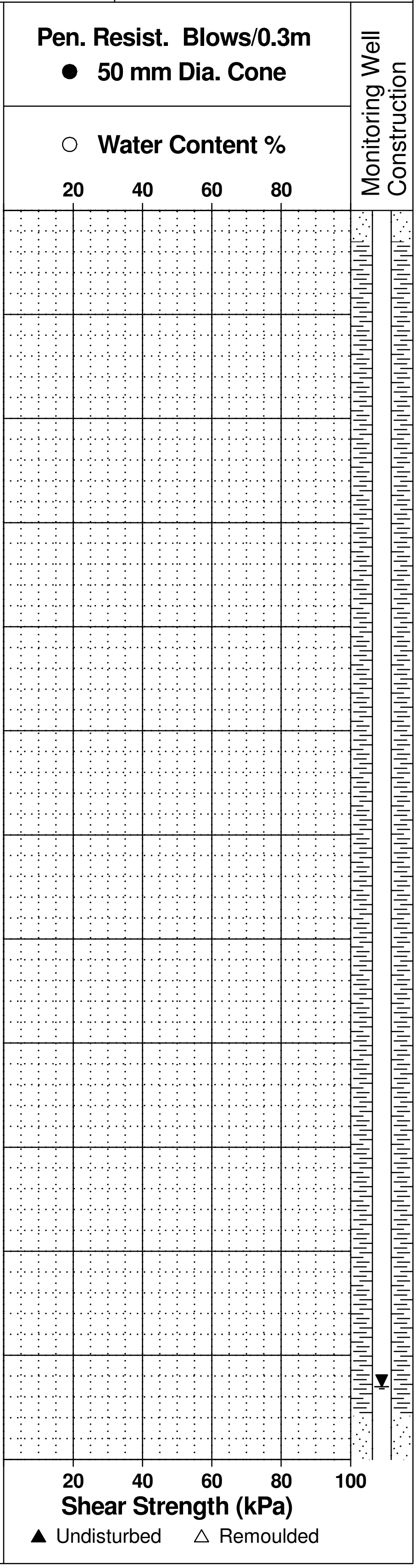
HOLE NO. **BH 5-21**

BORINGS BY CME 55 Power Auger

DATE November 9, 2021

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.36	AU	1		
FILL: Brown silty sand with gravel, occasional cobbles - trace topsoil and concrete from 2.3 to 2.9m depth - with asphaltic concrete by 6.1m depth	6.70	SS	2	63	50+
		SS	3	50	19
		SS	4	50	15
		SS	5	0	14
		SS	6	25	13
		SS	7	0	50+
		SS	8	58	43
		SS	9	67	15
Compact to dense, brown **SILTY SAND** - some gravel by 8.5m depth		SS	10	50	14
		SS	11	42	17
		SS	12	50	34
		SS	13	42	47
		SS	14	50	48
		SS	15	88	50+
		SS	16	50	35

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



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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

FILE NO. PG5792

REMARKS

HOLE NO. BH 5-21

BORINGS BY CME 55 Power Auger

DATE November 9, 2021

SOIL DESCRIPTION

GROUND SURFACE

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

14.20

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

- grey by 18.2m depth

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	17	21	9
SS	18	50	23
SS	19	50	28
SS	20	55	50+
RC	1	60	
SS	21	42	71
RC	2	22	
SS	22	64	38
RC	3	15	
SS	23	100	50+
RC	4	15	
SS	24	0	50+
RC	5	19	

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

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

○ Water Content %

20 40 60 80

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

Monitoring Well Construction

SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE November 9, 2021

FILE NO. PG5792

HOLE NO. BH 5-21

SOIL DESCRIPTION

GROUND SURFACE

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

29.95

BEDROCK: Excellent quality, grey limestone with occasional shale partings

31.55

End of Borehole

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

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	25	80	50+
RC	6	0	
SS	26	0	50+
RC	7	0	
SS	27	86	50+
RC	8	37	
SS	28	0	10
RC	9	100	100
RC	10	100	93

DEPTH (m)	ELEV. (m)
24	47.14
25	46.14
26	45.14
27	44.14
28	43.14
29	42.14
30	41.14
31	40.14

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

○ Water Content %

20 40 60 80

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

Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

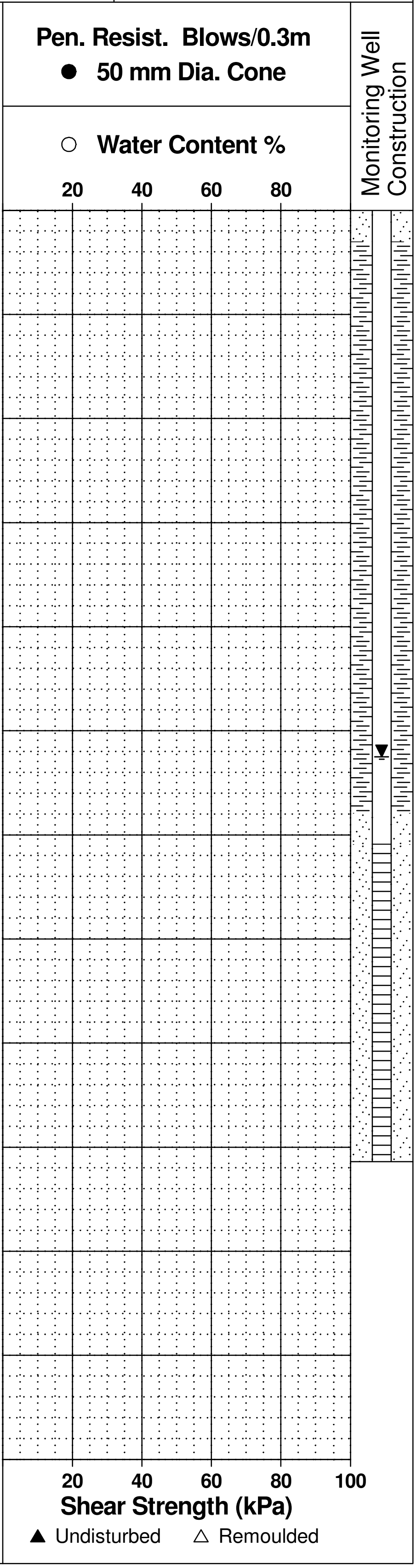
Geotechnical Investigation
Lansdowne Park Redevelopment
Prop. Multi-Storey Buildings & Rink Structure, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE November 11, 2021
FILE NO. PG5792
HOLE NO. BH 6-21

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

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

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



SOIL PROFILE AND TEST DATA

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

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE November 11, 2021
FILE NO. PG5792
HOLE NO. BH 6-21

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
GLACIAL TILL: Dense, grey silty sand with gravel, cobbles and boulders - some clay by 16.8m depth		SS	17		50+	12	53.14		
						13	52.14		
		RC	3	34					
						14	51.14		
		SS	18	52	41				
		RC	4	19					
						15	50.14		
		SS	19	86	50+				
						16	49.14		
		RC	5	0					
						17	48.14		
		SS	20	50	28				
						18	47.14		
		RC	6	11					
						19	46.14		
						20	45.14		
		SS	21	0	50+				
		RC	7	14					
						21	44.14		
		SS	22	0	50+				
						22	43.14		
		RC	8	35					
22.88									
						23	42.14		
BEDROCK: Good to excellent quality, grey limestone with occasional shale partings		RC	9	100	85				
						24	41.14		

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

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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 11, 2021

FILE NO. PG5792

HOLE NO. BH 6-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Monitoring Well Construction
GROUND SURFACE						24	41.14		
BEDROCK: Good to excellent quality, grey limestone with occasional shale partings		RC	10	100	98	25	40.14		
25.73									
End of Borehole (GWL @ 5.25m - Nov. 16, 2021)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

FILE NO. PG5792

REMARKS

HOLE NO. BH 7-21

BORINGS BY CME-55 Low Clearance Drill

DATE November 15, 2021

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

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

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

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

○ **Water Content %**

20 40 60 80

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

Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Redevelopment
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REMARKS

BORINGS BY CME-55 Low Clearance Drill

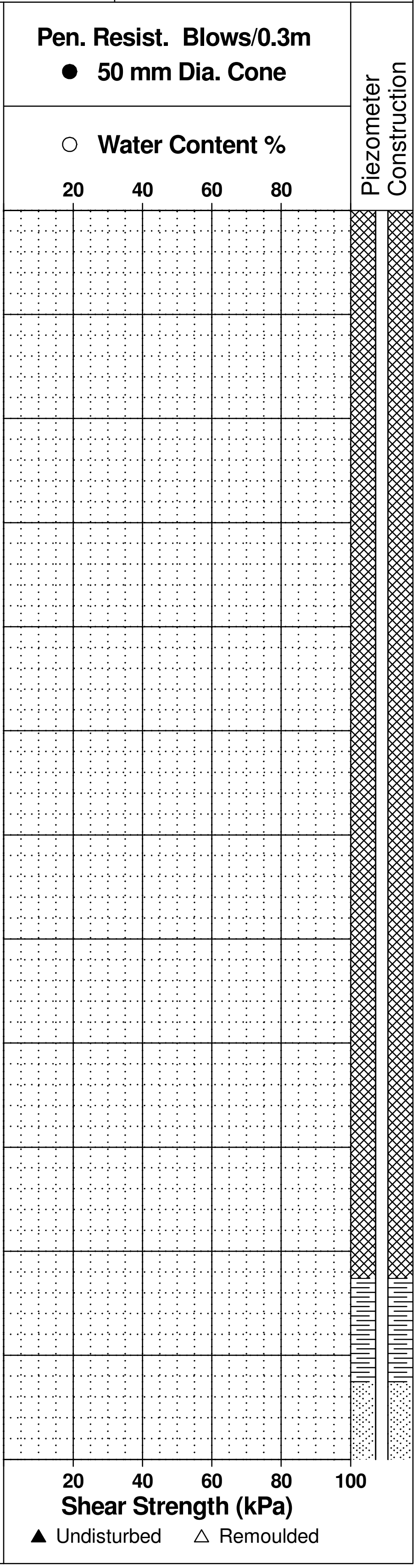
DATE November 15, 2021

FILE NO. PG5792

HOLE NO. BH 7-21

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders - grey by 13.7m depth	RC	4	48	
	SS	8	33	48
	RC	5	47	
	SS	9	33	50+
	RC	6	0	
	SS	10	0	50+
	RC	7	30	
	SS	11	73	50+
	RC	8	12	
	SS	12	77	50+
	RC	9	18	
	SS	13	0	50+
23.80	RC	10	100	100

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

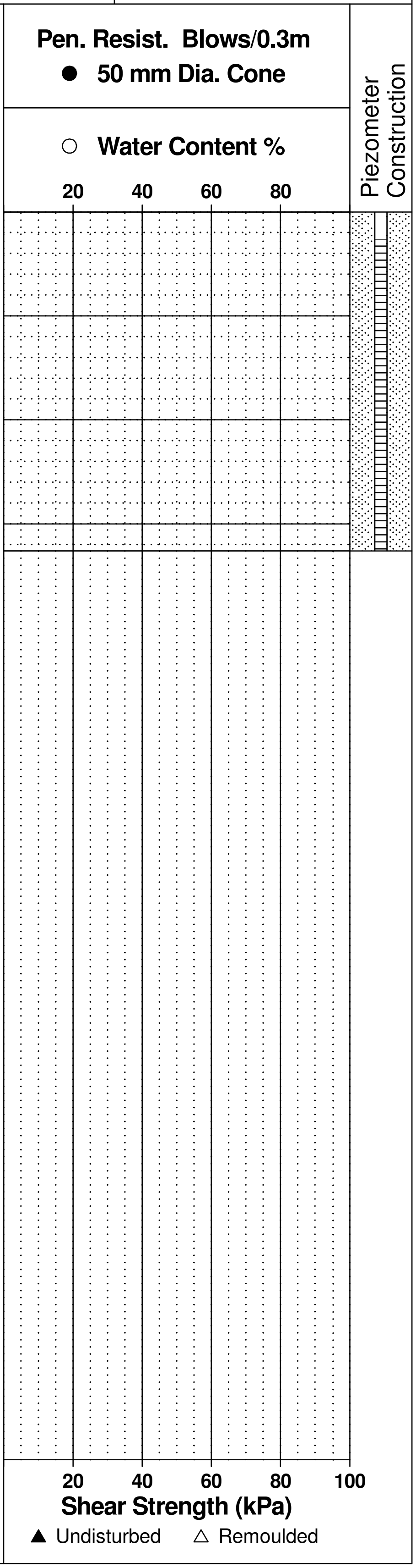


SOIL PROFILE AND TEST DATA

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

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE November 15, 2021
FILE NO. PG5792
HOLE NO. BH 7-21

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



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

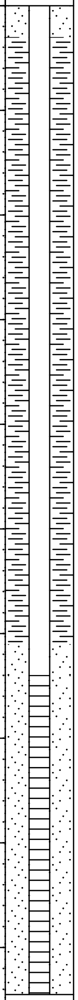
SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	20
SS	3	0	15
SS	4	0	8
SS	5	17	37
SS	6	42	41
SS	7	50	57
SS	8	42	36
SS	9	50	40
SS	10	50	36
SS	11	58	47
SS	12	50	41
SS	13	67	36
SS	14		45
SS	15	67	69
SS	16	67	43
SS	17	50	14

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

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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REMARKS

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DATE November 17, 2021

FILE NO. PG5792

HOLE NO. BH 8-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

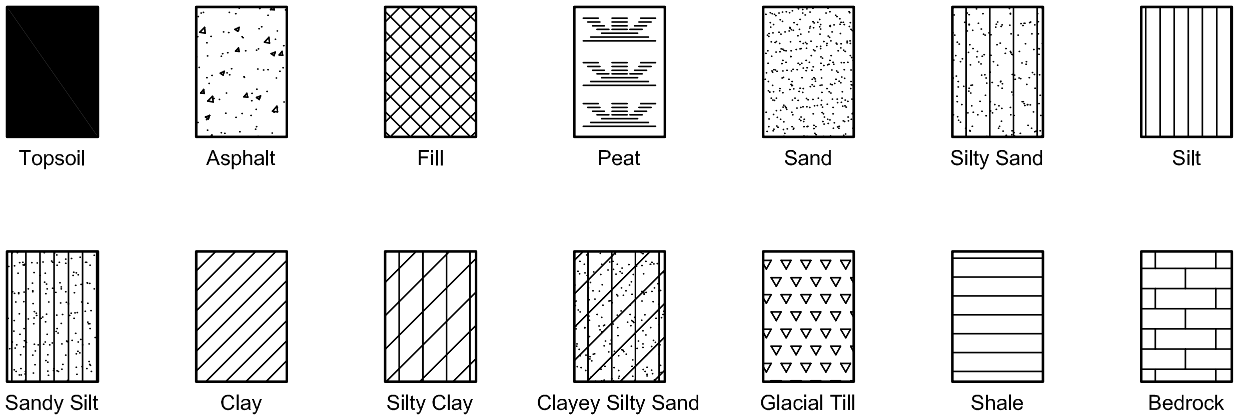
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

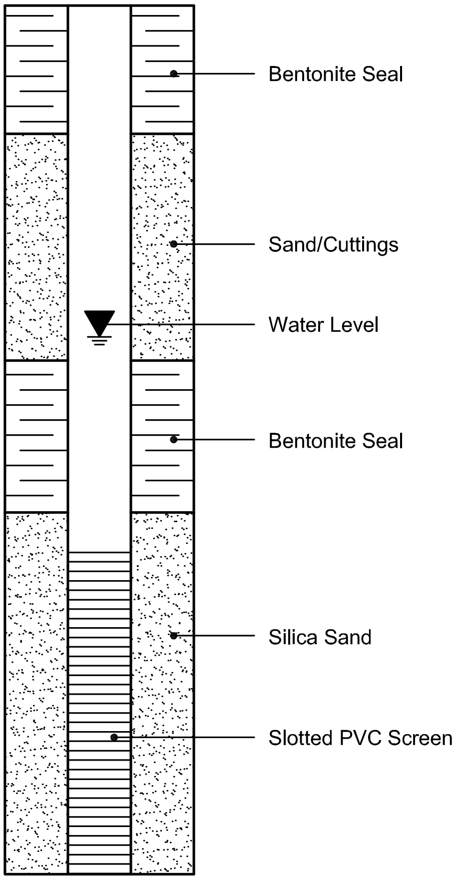
SYMBOLS AND TERMS (continued)

STRATA PLOT

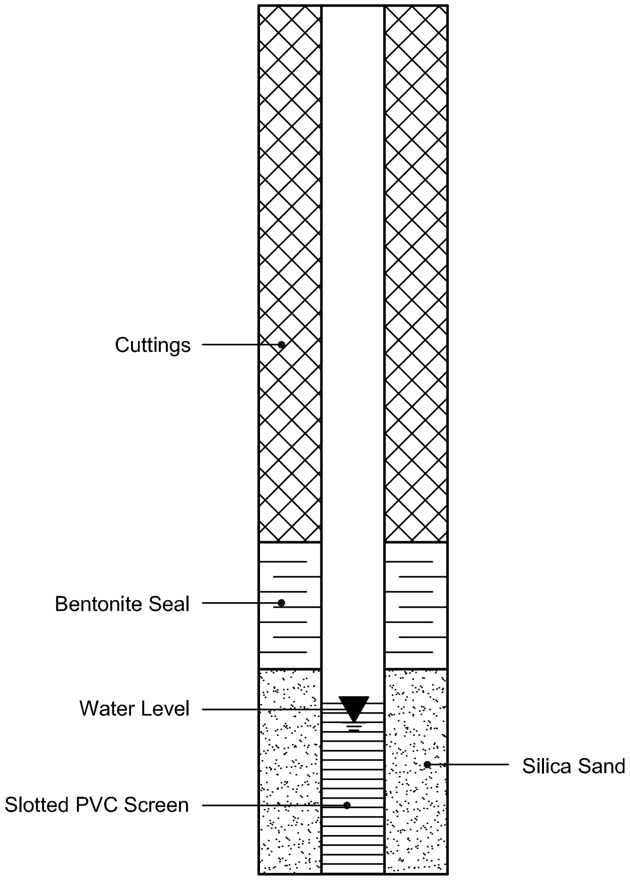


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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

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

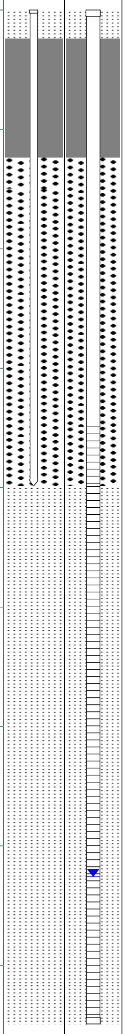
Project Name: CPU Ground Water Monitoring Program
Client: City of Ottawa
Entered By: KYLT
Drill Method: Direct Push
Drilled By: Strata Drilling Group

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

SUBSURFACE PROFILE		SAMPLE DATA				
Description	Elevation (m) / Depth (m)	Type	Number	Sample	N or RQD	Recovery (%)
Ground Surface	64.9					
TOPSOIL	0.0	SS				
FILL Fine grained loamy sand, trace gravel, dark brown	64.5 0.4	SS	1			45
Very fine grained sandy loam, dark brown, moist		SS	2			65
Brownish grey, wet						
Fine to medium grained sand, grey		SS	3			43
Trace gravel						
Fine to medium grained sandy loam and gravel						
SAND Fine to coarse grained sand, trace gravel	60.2 4.7					
END OF BOREHOLE	59.7 5.2					

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

WELLS



Remarks

Elevation: 64.924 masl
Easting: 368843.807
Northing: 5029183.520

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

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

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

Stratigraphic and Instrumentation Log: MW15-7

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

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

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

SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
	Ground Surface	64.51
0.00	**TOPSOIL**	
0.40	**FILL** Gravel and sand, grey Fine loamy sand, greyish brown Wet Fine to medium grained sand, brown Fine grained sandy loam	64.12
3.71	**SAND** Fine to coarse grained sand, trace gravel, brown, wet Trace silt Slightly grey	60.80
6.10	**END OF BOREHOLE**	58.42

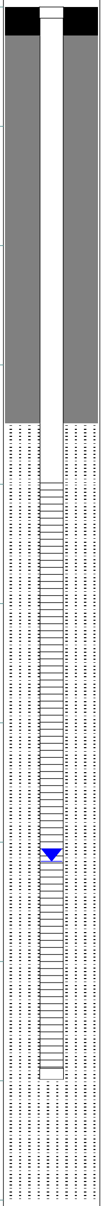
SAMPLE DATA

Type	Number	Sample	N or RQD	Recovery (%)
SS	1			68
SS	2			70
SS	3			65
SS	4			55

Combustible Vapour ○ (ppm) 250 750 1250

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

Monitoring Well Details



Remarks

Depth scale: ft 0–23; m 0–7

Elevation: 64.513 masl
Easting: 368911.901
Northing: 5029169.410

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

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

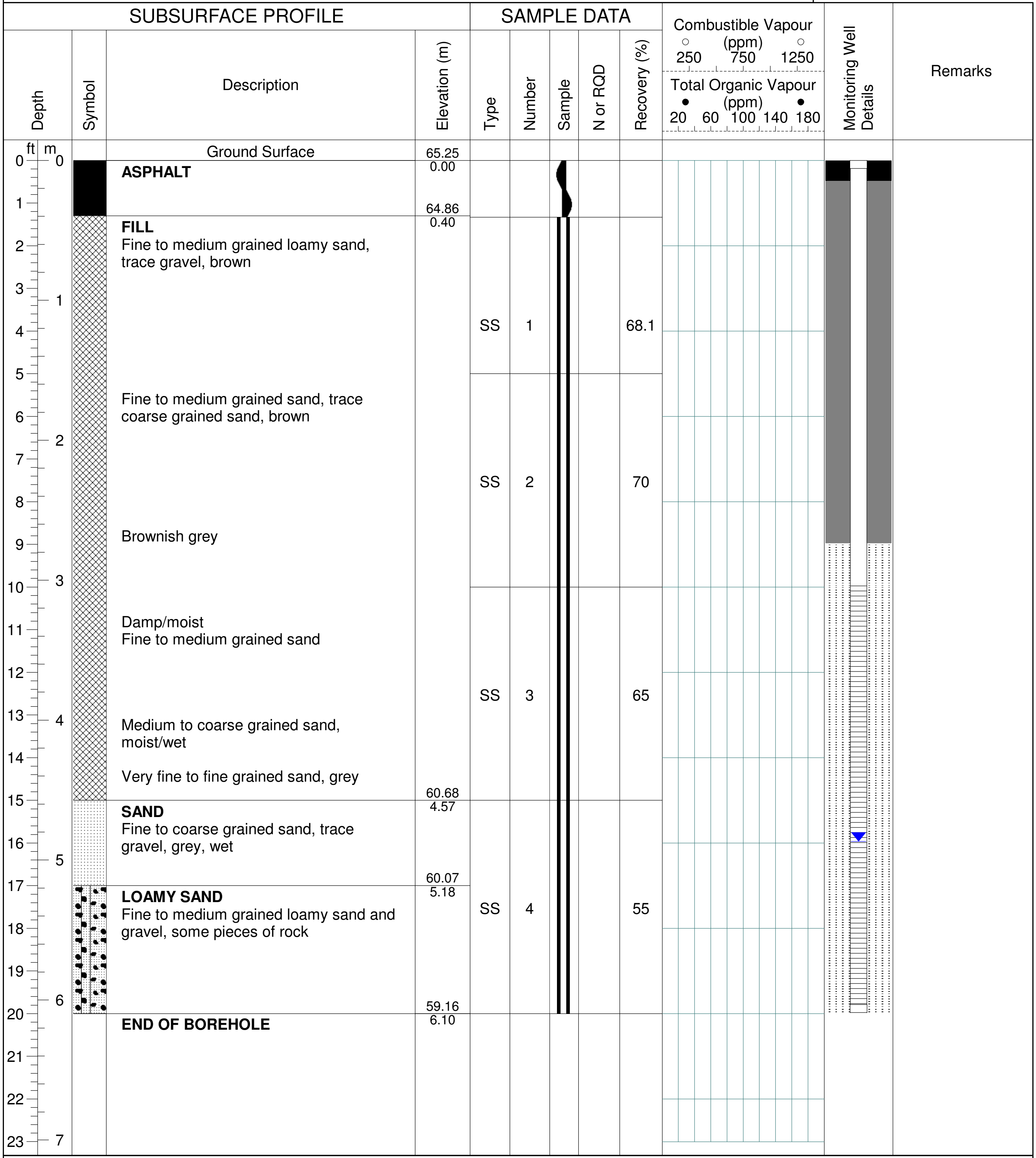
Datum: Geodetic
Checked by: KDH
Sheet: 1 of 1

Stratigraphic and Instrumentation Log: MW15-9

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

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

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



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

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

Elevation: 65.253 masl
Easting: 368798.392
Northing: 5029125.377

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

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

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

Stratigraphic and Instrumentation Log: MW15-10

amec foster wheeler

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

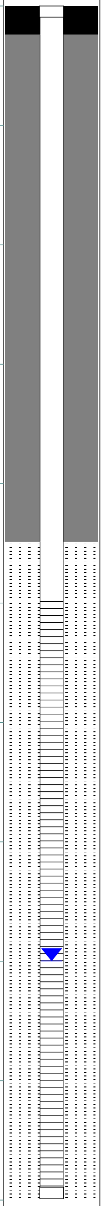
Project No: TZ10100106
Location: 945 Bank Street, Ottawa
Logged By: JFT
Drill Date: October 22, 2015
Hole Size: 127 mm

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

SUBSURFACE PROFILE | SAMPLE DATA

Combustible Vapour (ppm) ○ 250 750 1250 ○
Total Organic Vapour (ppm) ● 20 60 100 140 180 ●

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



Elevation: 64.043 masl
Easting: 368878.435
Northing: 5029083.949

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

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

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

Stratigraphic and Instrumentation Log: MW15-11

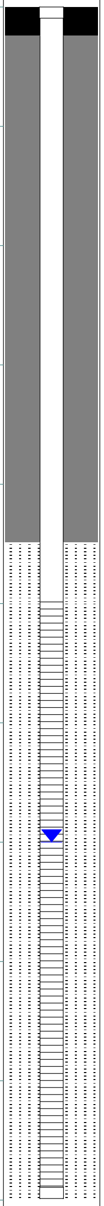
Project No: TZ10100106
Location: 945 Bank Street, Ottawa
Logged By: JFT
Drill Date: October 22, 2015
Hole Size: 127 mm

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

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

SUBSURFACE PROFILE			SAMPLE DATA							
Depth (ft / m)	Symbol	Description	Elevation (m)	Type	Number	Sample	N or RQD	Recovery (%)	Combustible Vapour (ppm) ○ 250 750 1250 Total Organic Vapour (ppm) ● 20 60 100 140 180	Remarks
0 ft / 0 m		Ground Surface	64.57							
0		**TOPSOIL**	0.00							
1.3		**FILL** Very fine to fine grained sand, trace silt, grey/brown	64.17 0.40	SS	1			66		
		Very fine to medium grained sand, brown/grey		SS	2			58		
		Fine to medium grained loamy sand and gravel, moist								
		Gravelly loamy sand, some pieces of rock		SS	3			52		
		Wet								
15		**SAND** Fine to medium and trace grained sand, some gravel	60.00 4.57	SS	4			33		
		Coarse sand and gravel								
20		**END OF BOREHOLE**	58.47 6.10							

Monitoring Well Details



Elevation: 64.571 masl
Easting: 368858.743
Northing: 5028968.821

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

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

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

Photograph 1: BH 3-21 RC 11 and RC12

Photograph 2: BH 4-21 RC10

Photograph 3: BH 4-21 RC11.

Photograph 4: BH 5-21 RC10

Photograph 5: BH 6-21 RC9.

Photograph 6: BH 8-21 RC7.

Photograph 7: BH 8-21 RC8

Certificate of Analysis

Report Date: 25-Nov-2024

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

Order Date: 19-Nov-2024

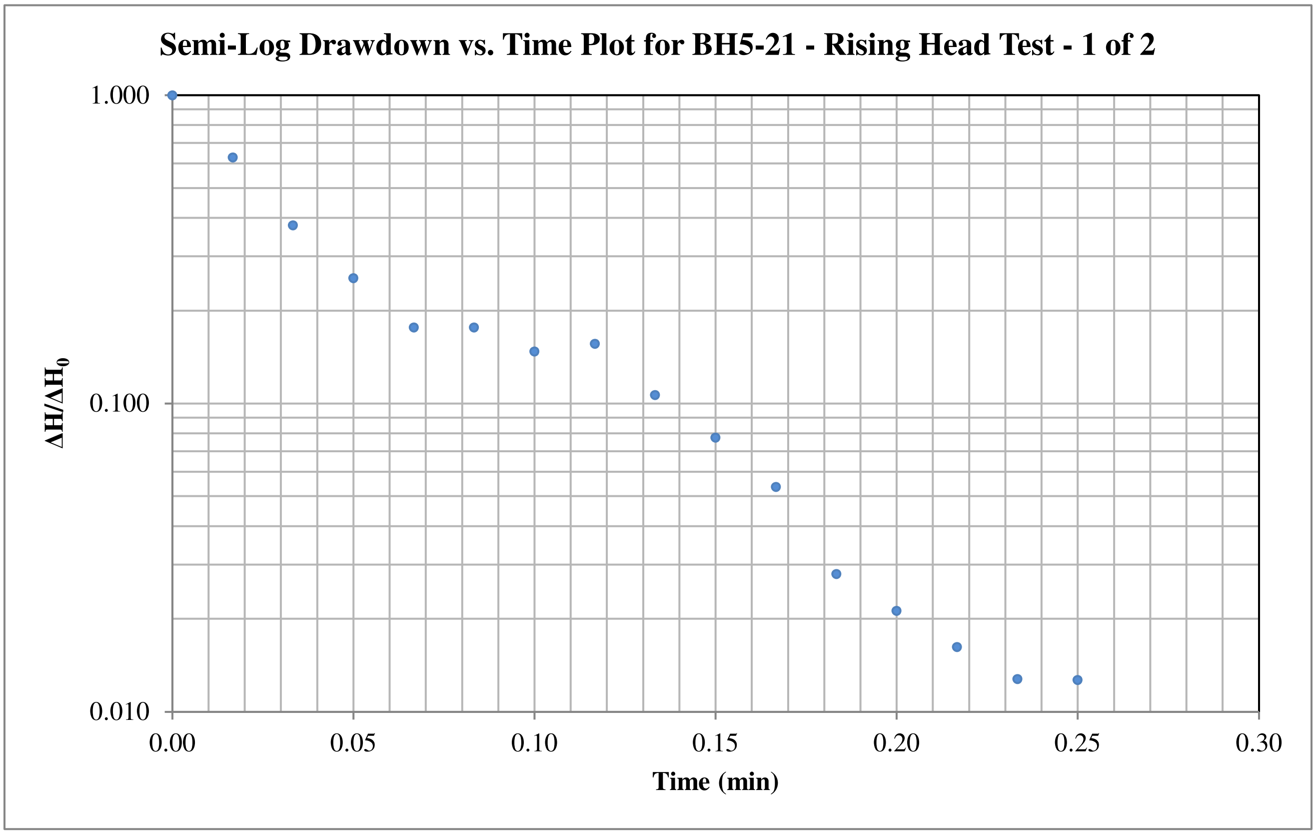
Client PO: 61790

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: BH5-21
Test: Rising Head - 1 of 2
Date: November 16, 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

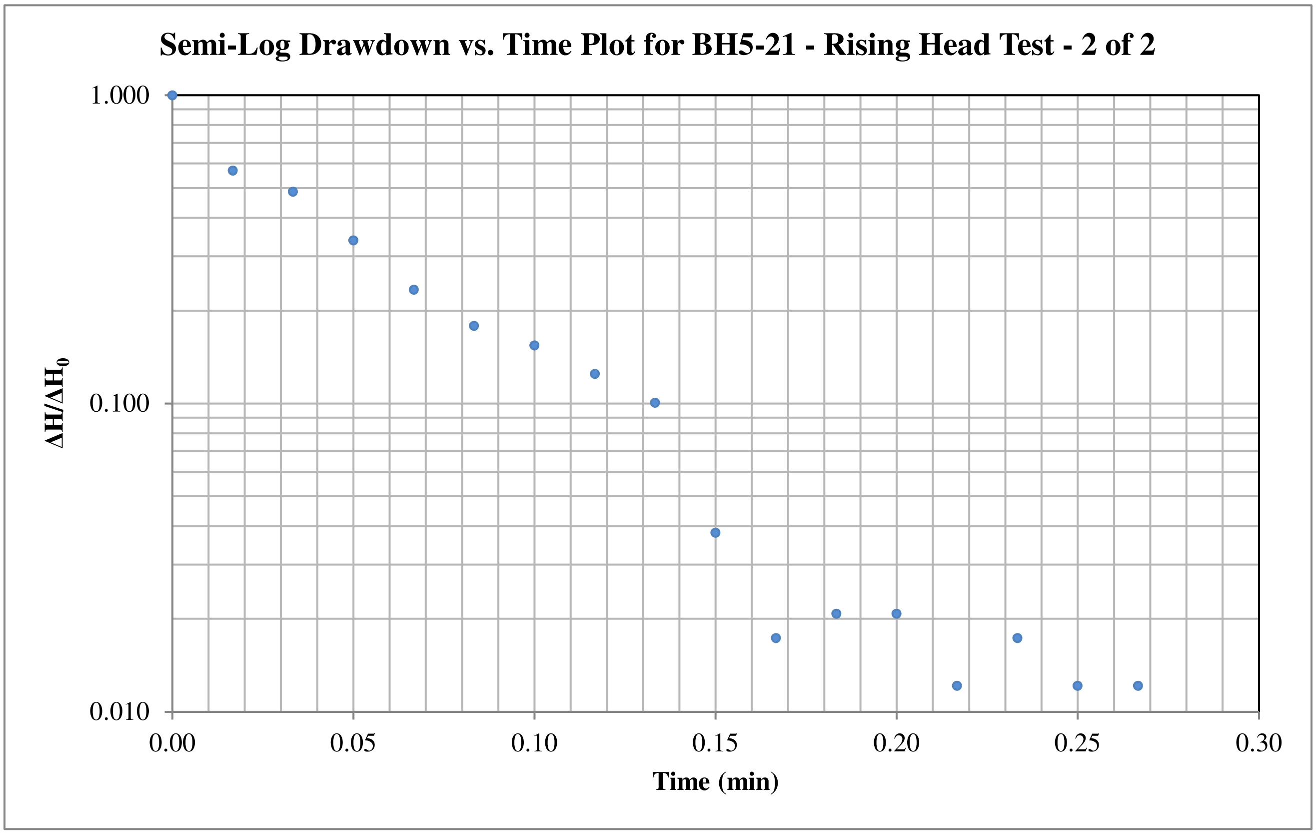
Data Points (from plot):

t*: 0.034 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.06E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Lansdowne - Trinity
Test Location: BH5-21
Test: Rising Head - 2 of 2
Date: November 16, 2021



Semi-Log Drawdown vs. Time Plot for BH5-21 - 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: 3.59613

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):

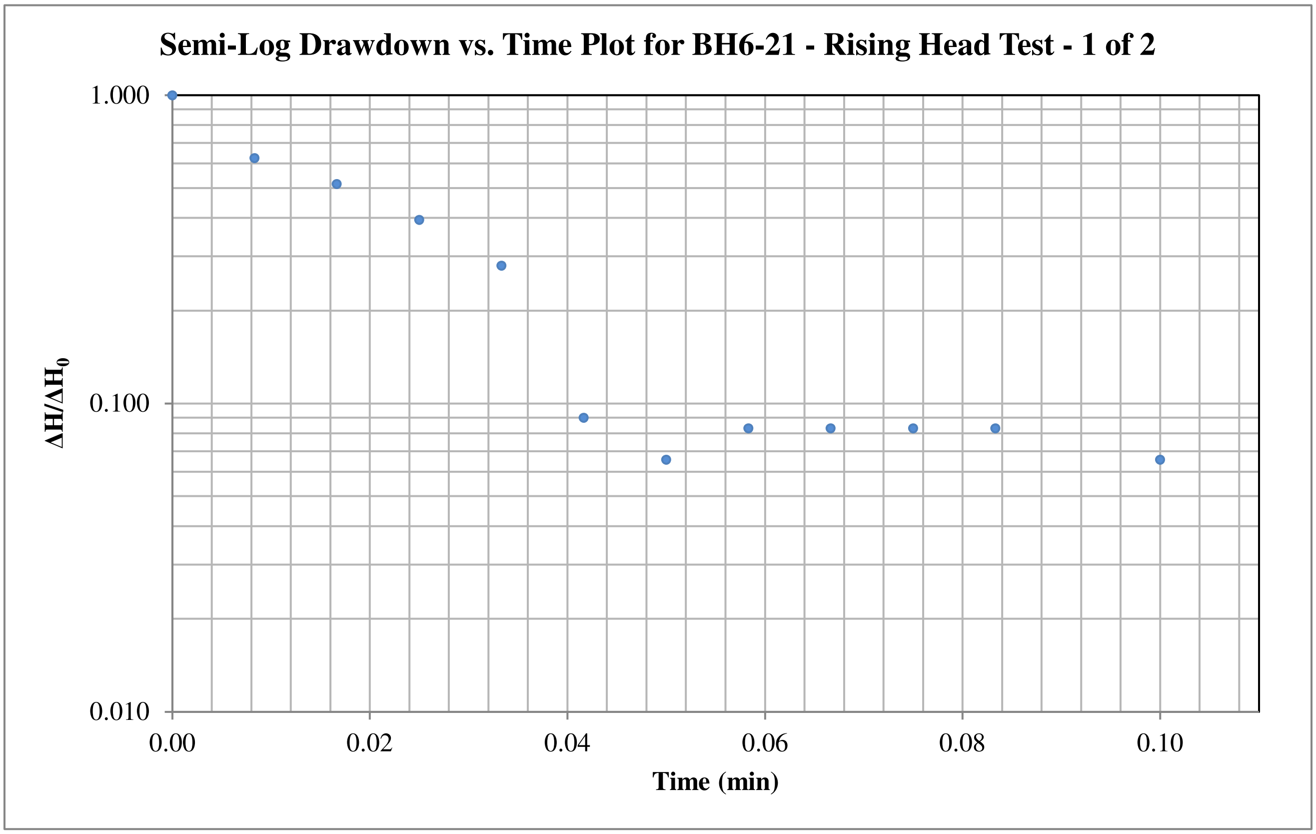
t*: 0.047 minutes

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

Horizontal Hydraulic Conductivity
K = 7.75E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Lansdowne - Trinity
Test Location: BH6-21
Test: Rising Head - 1 of 2
Date: November 16, 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

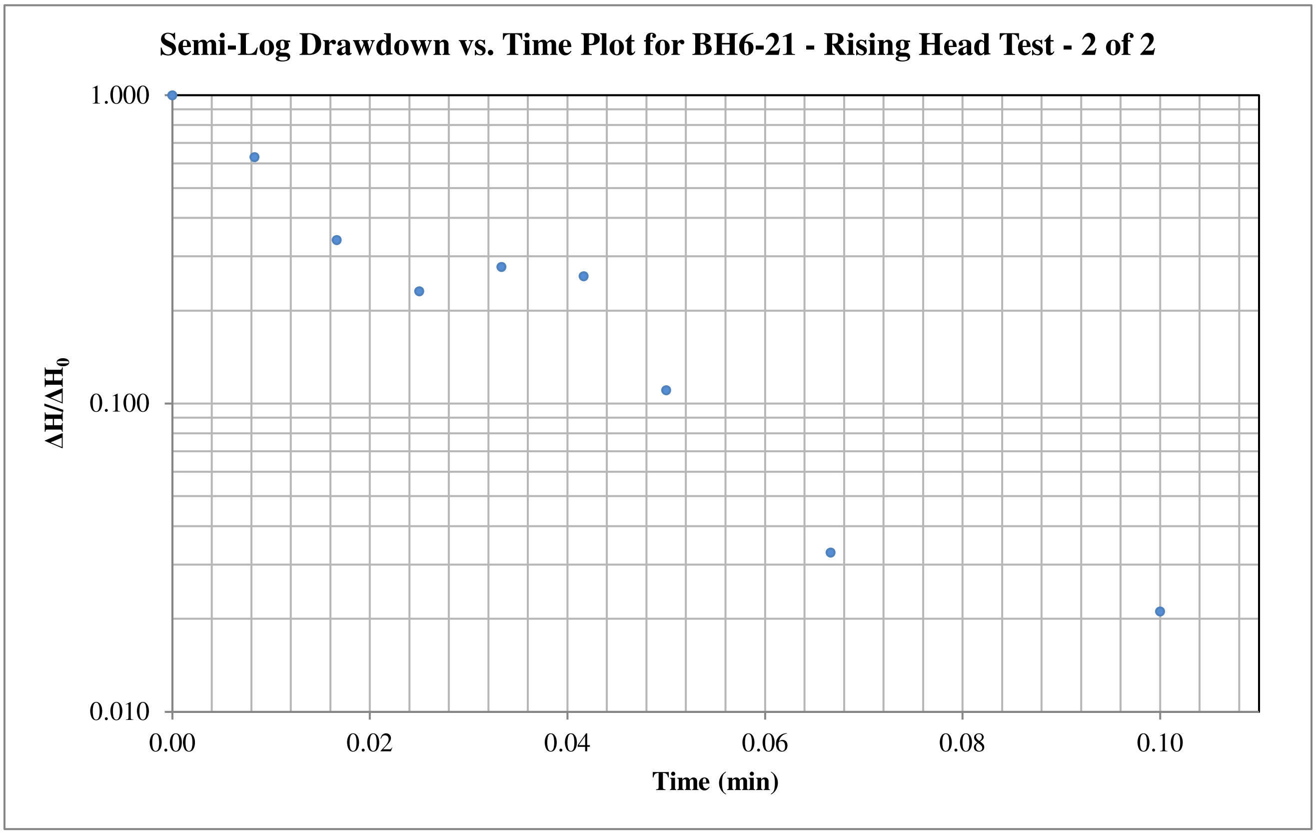
Data Points (from plot):

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

Horizontal Hydraulic Conductivity
K = 1.36E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Lansdowne - Trinity
Test Location: BH6-21
Test: Rising Head - 2 of 2
Date: November 16, 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

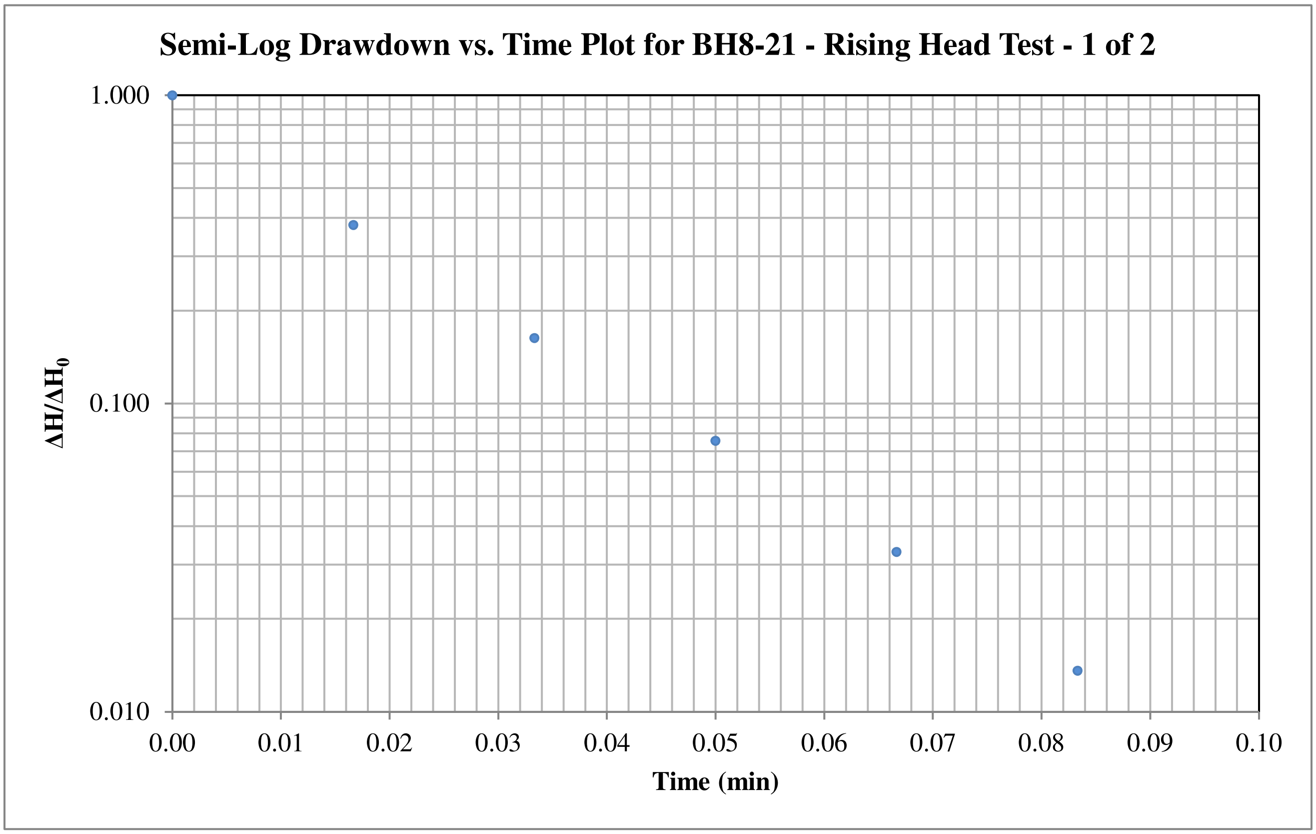
Data Points (from plot):

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

Horizontal Hydraulic Conductivity
K = 2.31E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Lansdowne - Trinity
Test Location: BH8-21
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):

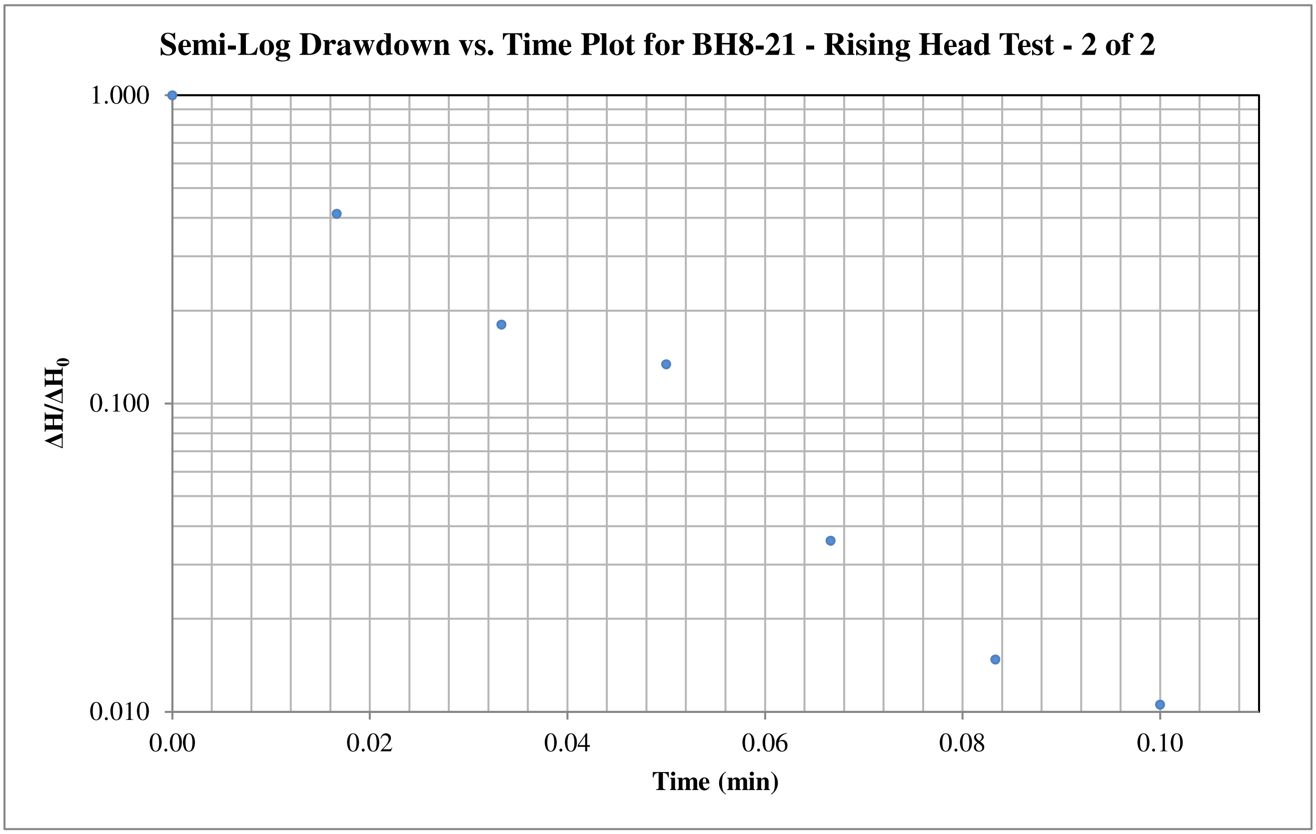
t*: 0.017 minutes

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

Horizontal Hydraulic Conductivity
K = 2.11E-04 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Lansdowne - Trinity
Test Location: BH8-21
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*: 0.019 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.92E-04 m/sec

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 - GROUNDWATER ELEVATION MONITORING - PROGRAM UPDATE

FIGURE 5 - SLOPE STABILITY ANALYSIS CROSS-SECTIONS

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

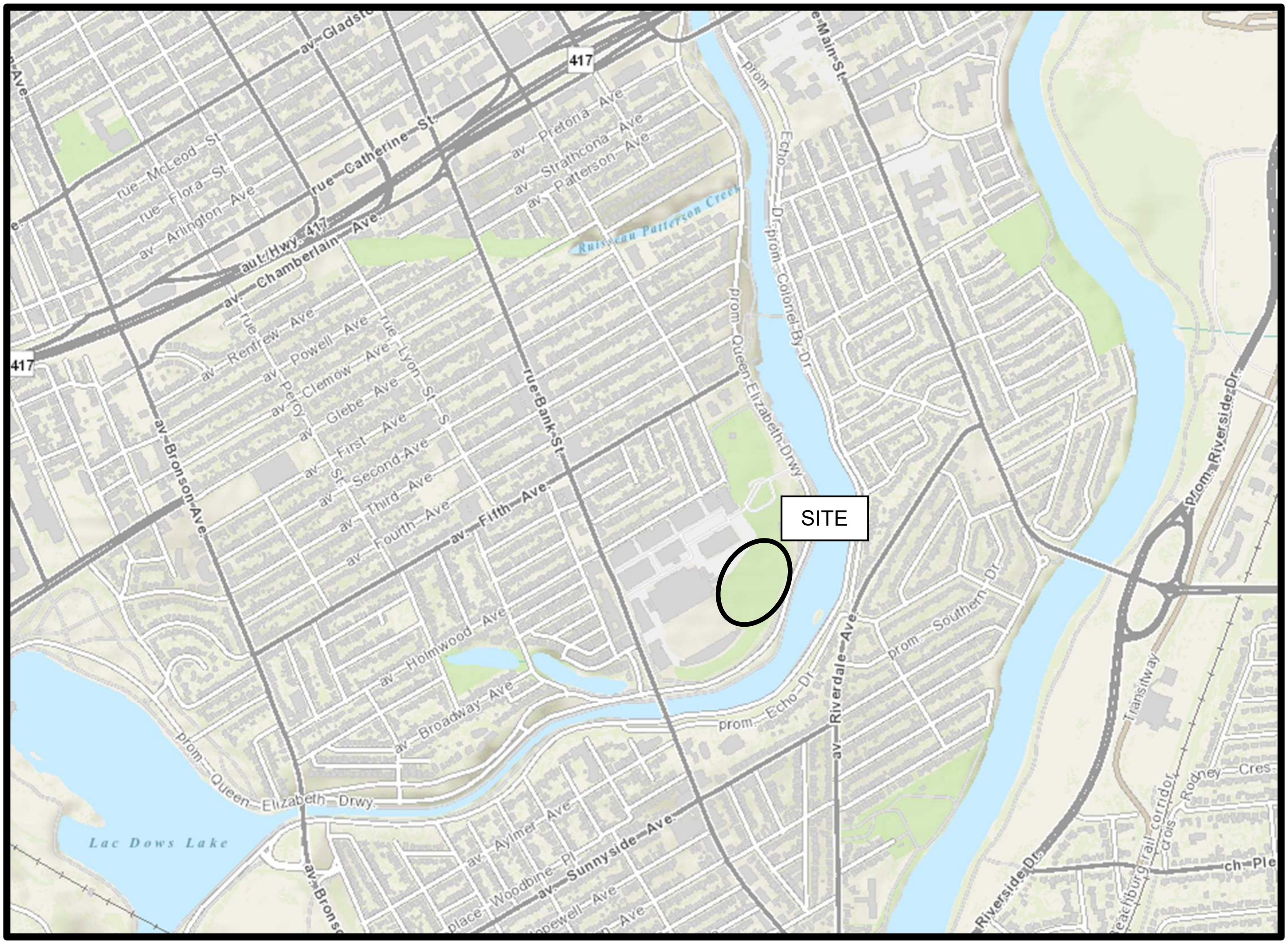


FIGURE 1

KEY PLAN

PATERSON GROUP

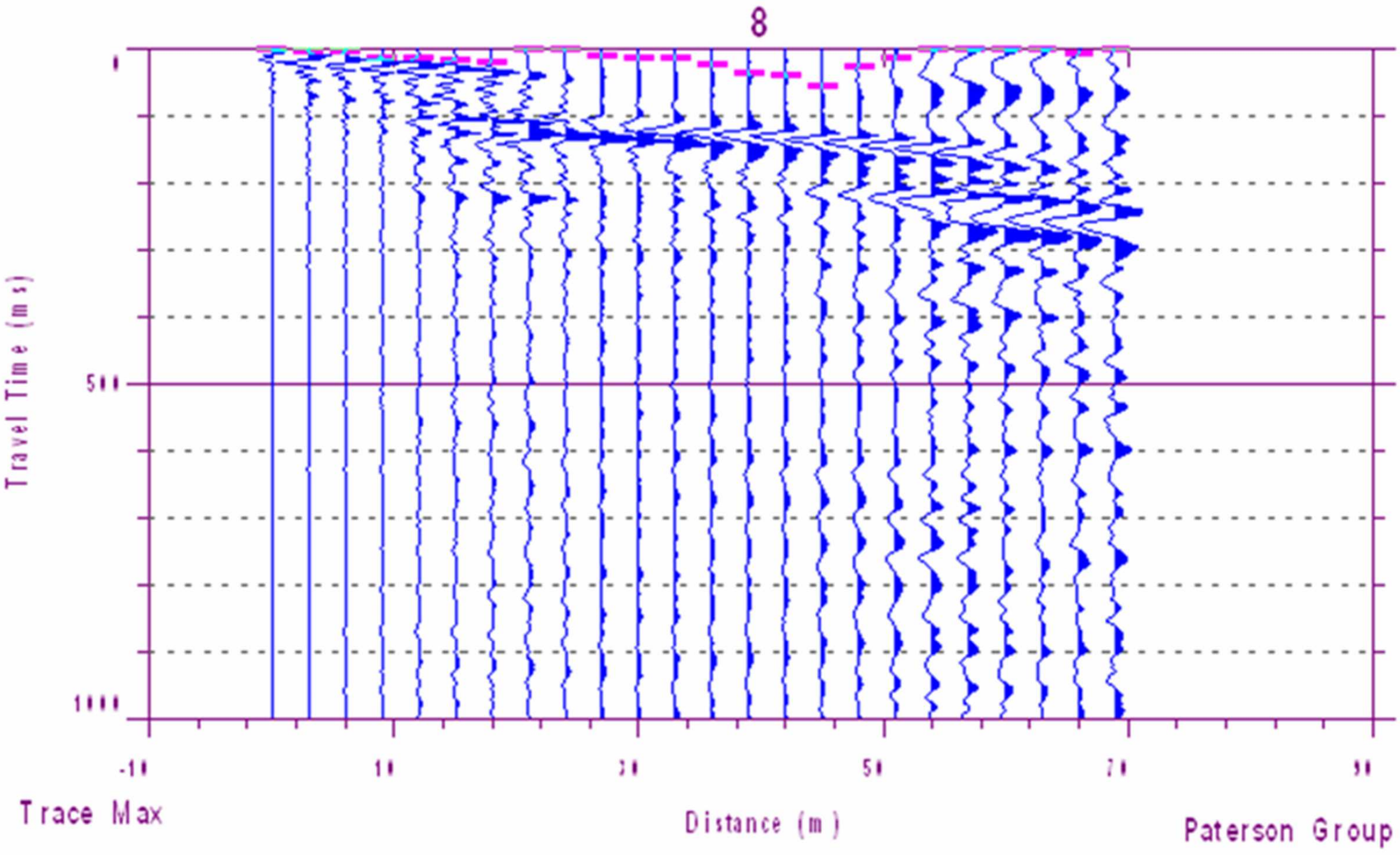


Figure 2 – Shear Wave Velocity Profile at Shot Location -3.0 m

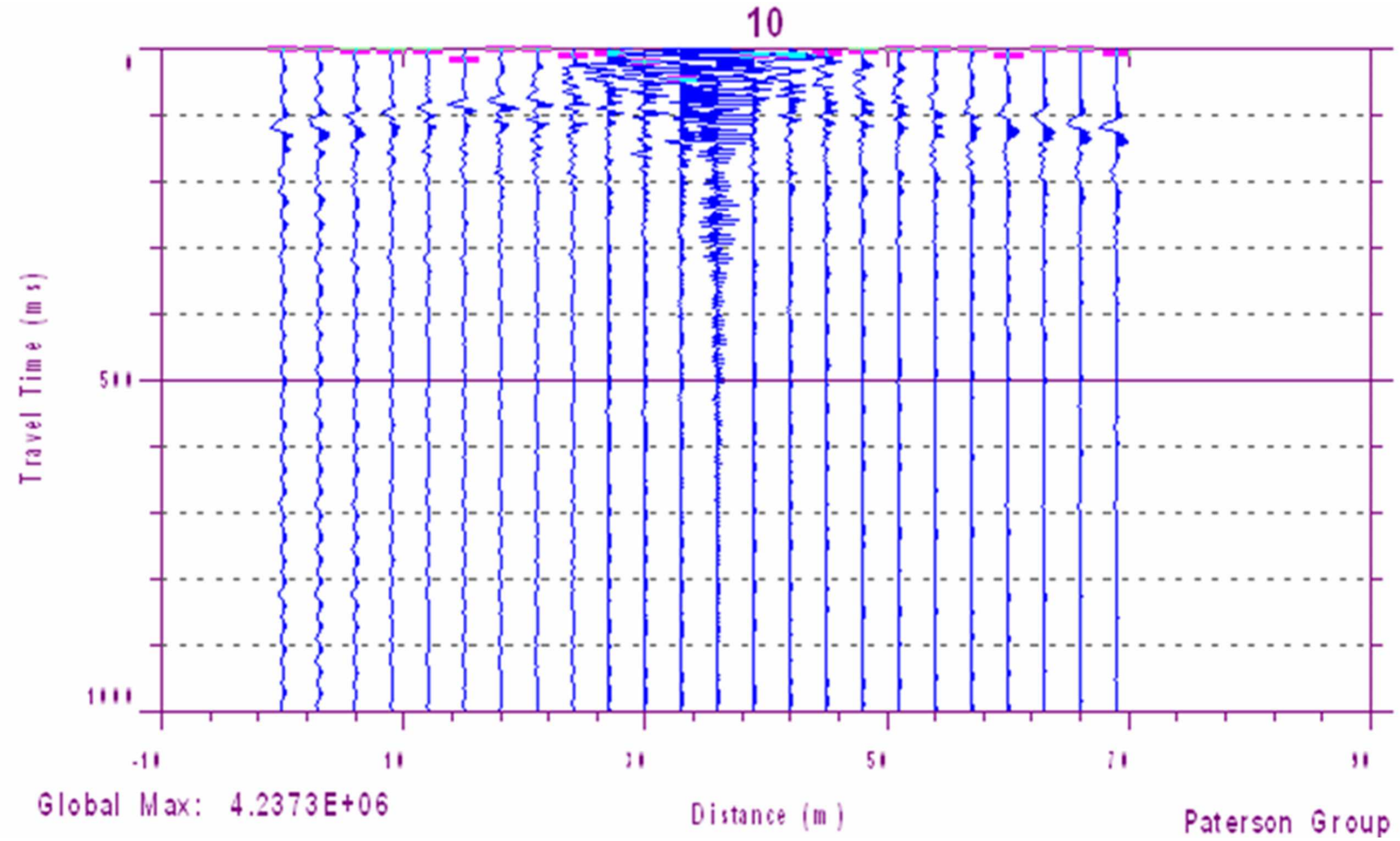
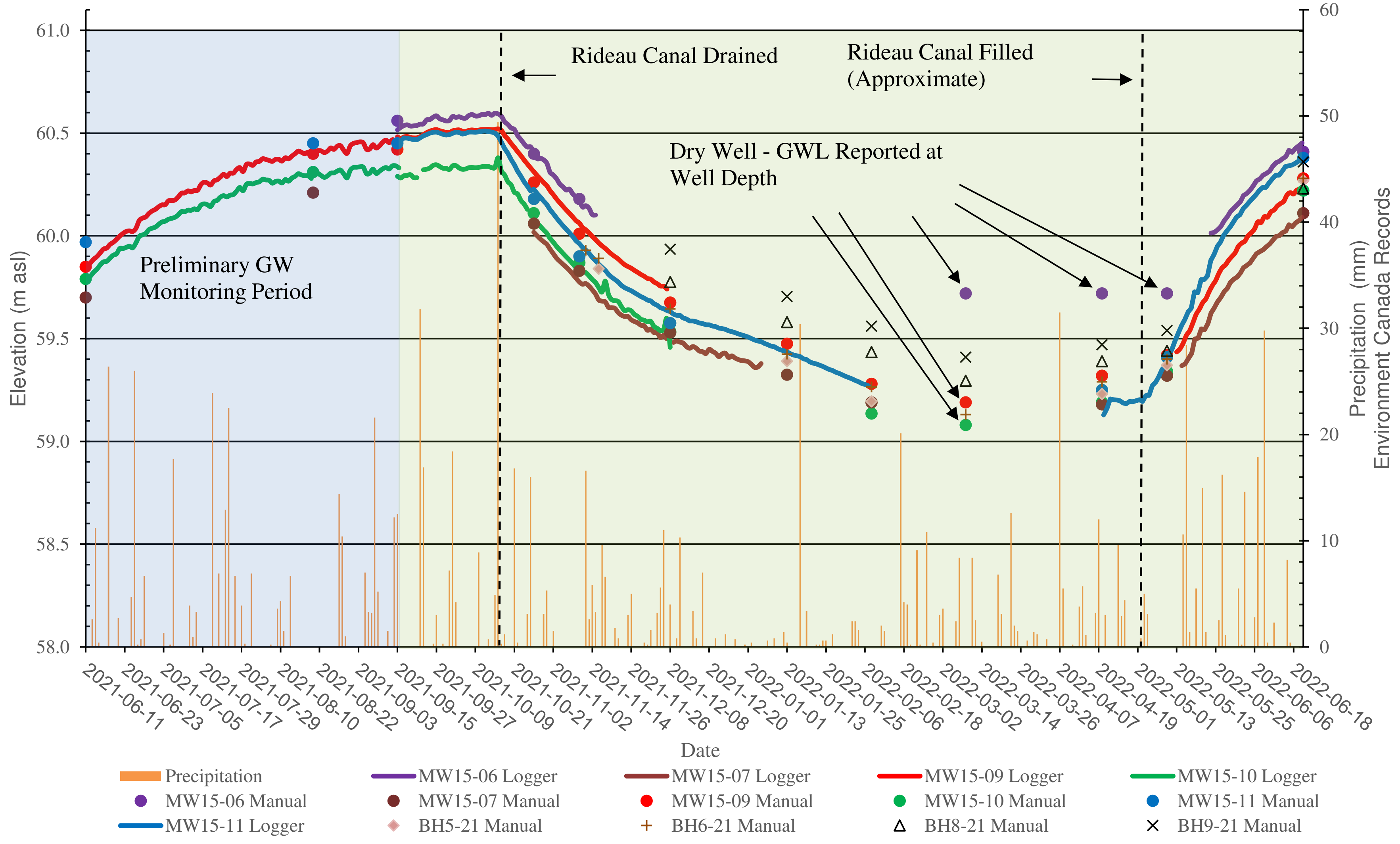


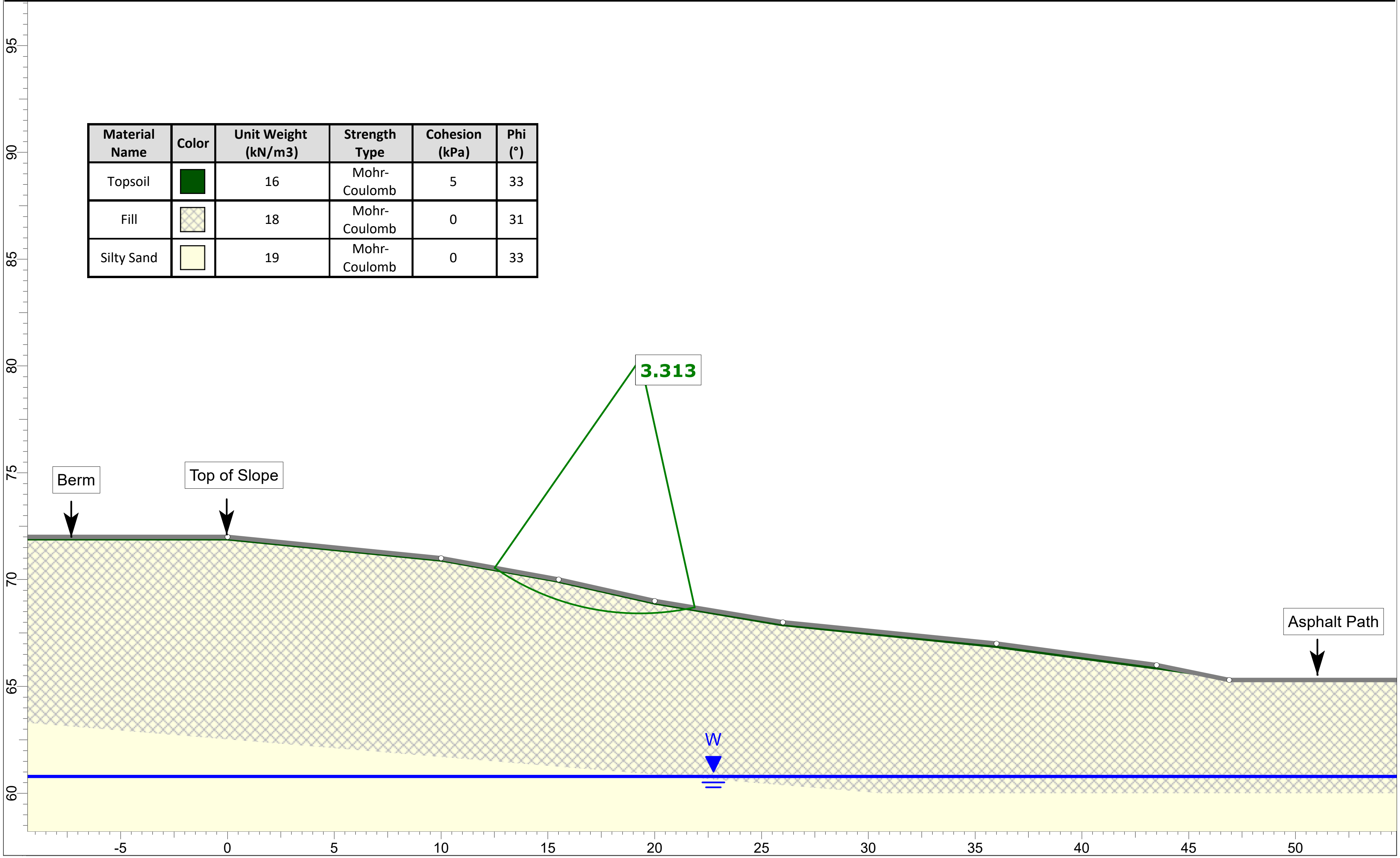
Figure 3 – Shear Wave Velocity Profile at Shot Location 34.5 m

Figure 4 : Groundwater Elevation Monitoring - Program Update



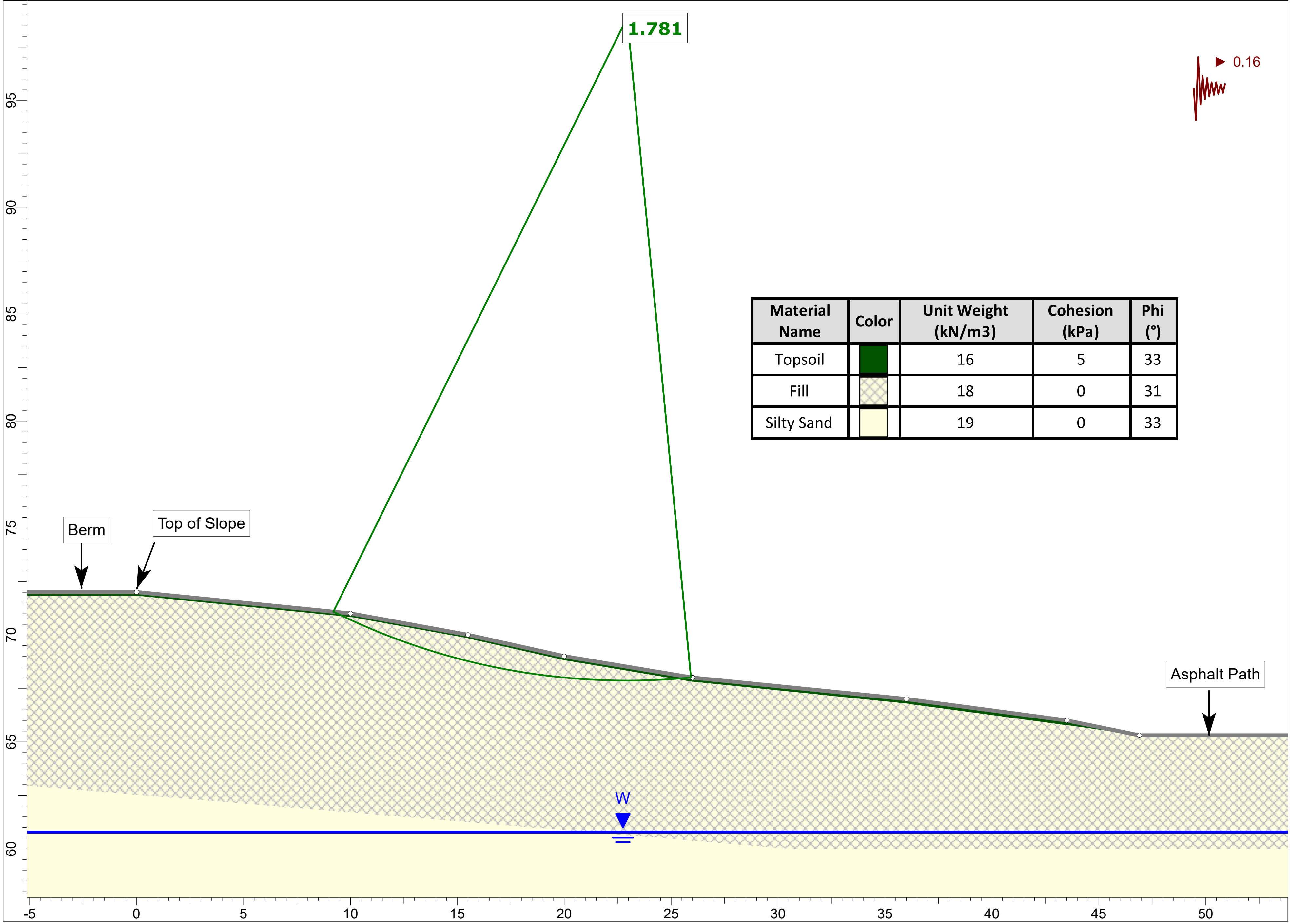
NOTE: THIS FIGURE HAS BEEN APPENDED TO PATERSON REPORT PG6655-2 DATED NOVEMBER 22, 2024, HOWEVER, THIS IS AN EXCERPT FROM A HYDROGEOLOGICAL REPORT PREPARED BY PATERSON AND SHOULD BE REFERENCED UNDER THE SEPERATE COVER.

FIGURE 5A - SLOPE SECTION A - PROPOSED CONDITIONS - STATIC LOADING

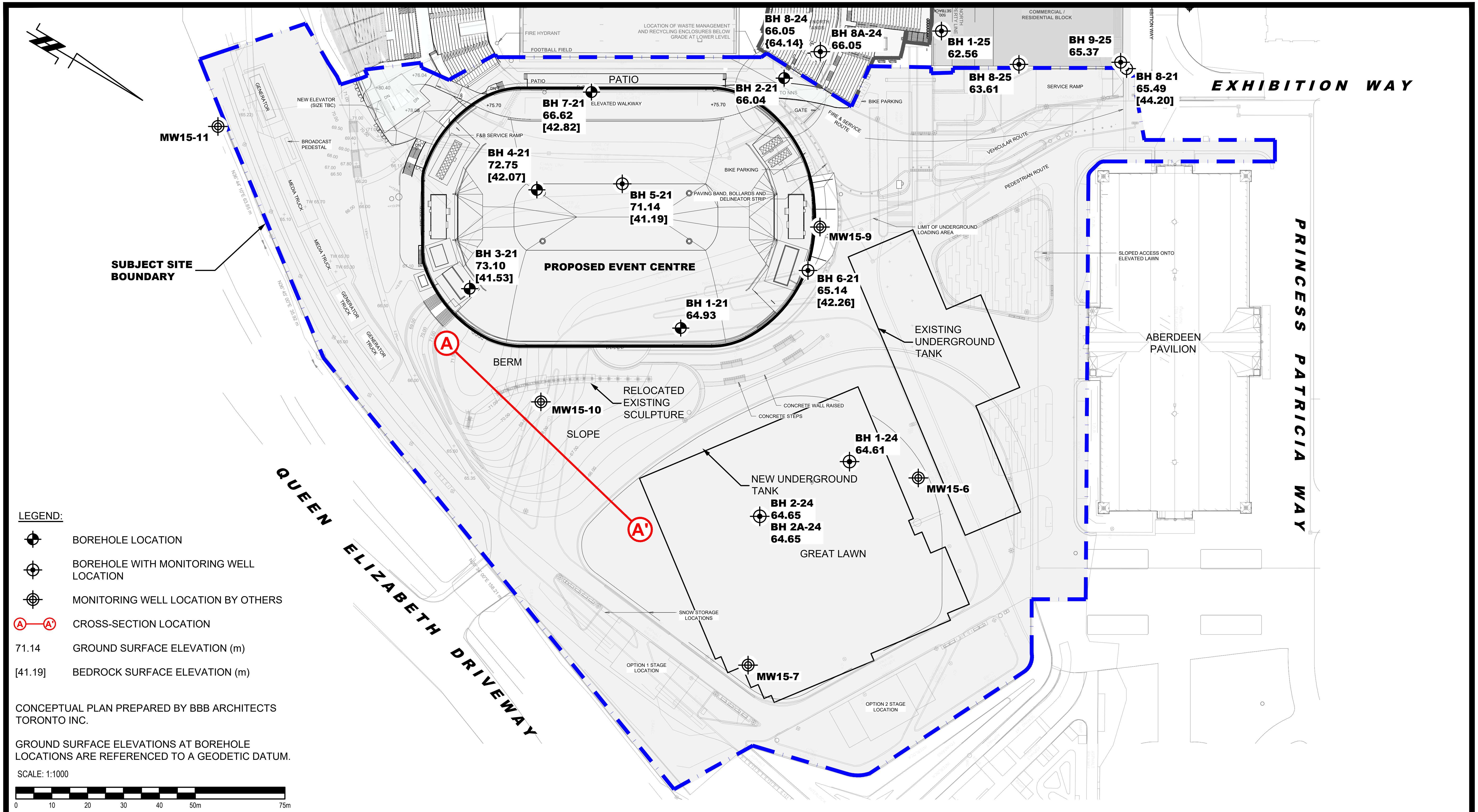


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Topsoil	🟩	16	Mohr-Coulomb	5	33
Fill	▨	18	Mohr-Coulomb	0	31
Silty Sand	□	19	Mohr-Coulomb	0	33

FIGURE 5B - SLOPE SECTION A - PROPOSED CONDITIONS - SEISMIC LOADING



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



LEGEND:

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

CONCEPTUAL PLAN PREPARED BY BBB ARCHITECTS TORONTO INC.

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

SCALE: 1:1000

0 10 20 30 40 50m 75m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
5	ADDED 2025 BOREHOLE LOCATIONS	18/03/2025	FC
4	UPDATED PHASE 1 CONCEPTUAL PLAN AND SITE BOUNDARY	15/01/2025	FC
3	UPDATED PHASE 2 CONCEPTUAL PLAN AND SITE BOUNDARY	17/12/2024	FC
2	ADDED 2024 BOREHOLE LOCATIONS, PHASE 2 CONCEPTUAL PLAN AND SITE BOUNDARY	27/11/2024	FC
1	UPDATED SITE BOUNDARY	06/08/2024	FC

CITY OF OTTAWA
GEOTECHNICAL INVESTIGATION
LANDSDOWNE PARK REDEVELOPMENT
PROPOSED EVENT CENTRE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000
Date: 05/2024
Drawn by: ZS
Report No.: PG6655-1
Checked by: FC
Dwg. No.: PG6655-1
Approved by: DP
Revision No.: 5

11x17

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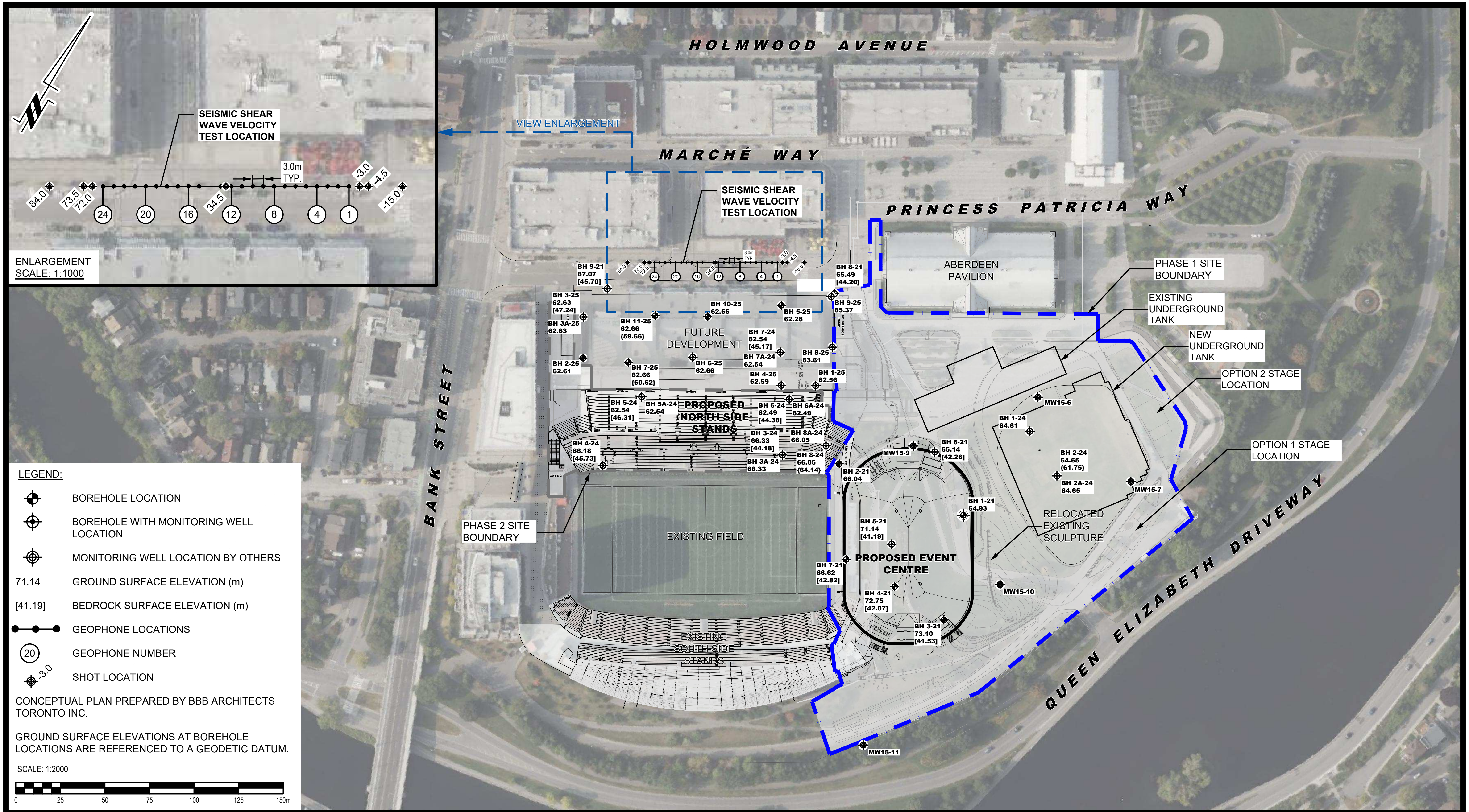

LEGEND:

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

CONCEPTUAL PLAN 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
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NO.	REVISIONS	DATE	INITIAL
5	ADDED 2025 BOREHOLE LOCATIONS	18/03/2025	FC
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1	UPDATED SITE BOUNDARY	06/08/2024	FC

CITY OF OTTAWA
GEOTECHNICAL INVESTIGATION
LANDSDOWNE PARK REDEVELOPMENT
PROPOSED EVENT CENTRE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2000
Date: 05/2024
Drawn by: ZS
Report No.: PG6655-1
Checked by: FC
Dwg. No.: PG6655-1A
Approved by: DP
Revision No.: 5

11x17

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APPENDIX 3

MEMORANDUM REPORT PG6655-MEMO.08 REVISION 4

memorandum

re: **Geotechnical Review and Recommendations - Underground Stormwater Tank System**
Proposed Lansdowne Development - Proposed Event Centre
Lansdowne Park - 945-1015 Bank Street - Ottawa
to: City of Ottawa - **Sean Moore** - sean.moore@ottawa.ca
date: July 28, 2025
file: PG6655-MEMO.08 Revision 4

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide geotechnical review and recommendations regarding the proposed underground stormwater tank system designed within the proposed Lansdowne Development located within the subject site. This memorandum supplements and supersedes the recommendations provided in *Subsection 5.8 - Underground Stormwater Tank System* of Paterson Group Report PG6655-1 Revision 3 dated April 3, 2025.

1.0 Background Information

It is understood that an underground prefabricated watertight stormwater tank system has been proposed as part of the development. It is further understood that the proposed tank currently consists of an MC-3500 Stormtech Chamber system with an approximate footprint of 5,000 m^2 which will be located within the Great Lawn area and east of the proposed Event Centre. The proposed system will be connected to the existing underground stormwater tank system located to the northwest of the proposed structure.

Paterson reviewed the following drawings and specifications regarding the aforementioned system:

- ❒ Lansdowne 2.0 - Project # S4 26399 - Sheet No. 1 to 6 – Revision 2 dated December 13, 2024, prepared by Advanced Drainage Systems, Inc (ADS).
- ❒ Technical Note TN 6.50 – Thermoplastic Liners for Detention Systems, prepared by Advanced Drainage Systems, Inc (ADS).
- ❒ Lansdowne 2.0 – Risk Management Plan – Dwg. RM01 to RM07 – Revision 4 dated June 20, 2025, prepared by WSP
- ❒ Grading Plan - Lansdowne Event Centre 945 & 1015 Bank Street – Project No. CA0033920.1056 – Drawing C04 – Revision 12 dated June 20, 2025, prepared by WSP.
- ❒ Servicing Plan - Lansdowne Event Centre 945 & 1015 Bank Street – Project No. CA0033920.1056 – Drawing C05B – Revision 12 dated June 20, 2025, prepared by WSP.

Structural analysis regarding the proposed Stormwater Tank System to be installed at the subject site were also provided by ADS and are attached to this memo report. The results of the analysis indicated that an additional pressure up to 4,769 kg/m^2 (46.7 kPa) may be supported by the system until failure provided that a minimum 731 mm layer of fill material is placed between the proposed finished grade and the top of the MC-3500 chamber.

Field Observations

Multiple geotechnical field investigations have been conducted within the subject area. Field investigation programs were completed by Paterson in 2024, 2021, 2013, 2010, 2003 and 1998 and consisted of a total of thirteen (13) boreholes to a maximum depth of 25.7 m below existing ground surface. Supplemental investigations were completed by others in 2015 and 2010 and consisted of a total of eight (8) boreholes to a maximum depth of 7.6 m below existing ground surface. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG6655-3 – Test Hole Location Plan attached to the present memorandum.

Generally, the subsurface profile encountered at the test hole locations consists of topsoil and fill underlain by a deposit of silty sand which is further underlain by a glacial till deposit. The fill material was generally observed to consist of silty sand with trace amounts of gravel and organics.

A historical landfill area has previously been identified throughout the northeast portion of the subject area. The fill material throughout this area was generally observed to consist of silty sand with gravel, organics and waste (i.e., wood, concrete, glass, metal, ash, ceramic). Based on aerial photos and available reports, the disposal program associated to the landfill area is estimated to have been completed before 1928 which is considered to be the earliest public aerial image for the site.

Based on the existing borehole information, the native, in-situ, undisturbed soils were encountered at approximate geodetic elevation between 60.1 to 64.7 m throughout the subject area. Reference should be made to the Soil Profile and Test Data sheets and the Borehole Logs by Others attached to the present memo report for details of the soil profile encountered at each borehole location.

2.0 Geotechnical Review

Based on our review of the above noted drawings, the subsurface profile and soil conditions within the area of the proposed underground storage tank, it is understood that the tanks will be placed at an elevation of 63.026 m and within the existing fill material.

Based on the results of our geotechnical investigation and associated groundwater monitoring program, the current design groundwater table elevation may be considered at a geodetic elevation of 60.78 m. Therefore, the underside of the proposed underground storage tank will be founded over 1 m above the seasonal high groundwater table level.

It is further understood that the stormwater tank system will be constructed as a watertight system and provided with a thermoplastic liner around all vertical excavation walls and 229 mm below the bottom of the chamber. Consideration should be given to using a 40mil linear low-density polyethylene (LLDPE) as thermoplastic liner with welded joints reviewed and approved by Paterson. Reference should be made to Technical Note TN 6.50 Thermoplastic Liners for Detention Systems attached to the present memo report. Equivalent membranes that meet the same technical requirements as the above noted, reviewed and approved by Paterson, could also be considered.

It is recommended that heat welded pipe “boots” are used to seal pipe penetrations through the liner, i.e., at the connection between the inlet and the tanks. Pipe boots should be further sealed with liquid waterproofing membrane. Further, where the liner abuts against sewer infrastructure such as manholes and catch-basins, all portions of the area of contact between the liner and the infrastructure should be sealed with a liquid waterproofing membrane such as Soprema LM Barr, Henry BlueSkin and/or equivalent other reviewed and approved by the manufacturer and Paterson. The installation of all types of waterproofing membranes, liners and geotextiles should be reviewed and approved by Paterson field personnel.

Based on Paterson review, the proposed stormwater tank system will be watertight and not contribute to groundwater level fluctuations by infiltration.

Stripping Depth

Topsoil and deleterious fill, such as those containing significant amounts of organic materials, should be stripped from under the proposed storage system. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

The existing fill, where free of organics and deleterious materials, can be left in place below the proposed system. It is recommended that the existing fill be proof-rolled under dry conditions and above freezing temperatures by an adequately sized sheepsfoot roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by Paterson personnel at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and reinstated with an approved engineered fill, such as OPSS Granular B Type II.

It is expected the northeastern portion of the proposed tank footprint will be located throughout a historical landfill footprint, and as depicted on Drawing PG6655-3 – Test Hole Location Plan. Where significant amounts of inorganic waste (i.e., concrete, glass, metal, ash, ceramic) are encountered at the founding depth of the proposed stormwater tank system over the historical landfill footprint, fill identified as unsuitable by Paterson personnel should be locally sub-excavated below the founding elevation and replaced with engineered fill, such as OPSS Granular B Type II or suitable site-generated fill material resulting from the excavation of the tank and expected to consist of silty sand. The fill material should be compacted to a minimum 95% of the material's SPMDD. The remaining material encountered at the subgrade level will be considered suitable to be left in place for proof-rolling.

The excavation, backfill, and compaction program should be reviewed and approved by Paterson personnel at the time of construction. All portions of the tank footprint located over the landfill footprint will also require to be proof-rolled as indicated herein to improve the compactness of the in-situ soils that are anticipated to be in a relatively loose to compact state upon sub-excavation. Undertaking the subgrade improvement efforts as noted herein will provide an adequate bearing medium for the proposed tank system and maintain total and differential settlements within the tolerances advised within the *Bearing Resistance Values* portion of this memorandum.

It is further understood that the existing non-woven geotextile and soft soil or hard cap encountered within the existing layout of the proposed underground tank will be removed. It is recommended that Paterson review the associated tie-ins and details for construction of the non-woven geotextile and soft soil or hard cap in relation to the existing system located northwest of the proposed new tank.

Existing foundation walls and other construction debris beyond the historical landfill footprint that might be encountered within the area of the excavation should be entirely removed from within the system perimeter. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Paterson personnel should review and approve all bearing surfaces prior to backfilling.

Fill Placement

A summary of the fill structure to be used at the proposed stormwater tank system is provided below, in *Table 1 – Fill Material Summary* and in *Risk Management Plan Section B & C – DWG No. RM04* prepared by WSP and attached to the present memorandum. Reference should also be made to the drawings prepared by ADS and attached to the current memo report.

Layer “A”

Fill placed for grading beneath the stormwater system area, or “Layer A”, should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular B Type I or II and in accordance with the above-mentioned drawings and *Table 1- Fill Material Summary*. The fill should be placed in maximum 150 mm thick loose lifts and compacted by suitable vibratory compaction equipment.

Fill placed beneath the structure should be compacted to a minimum of 99% of the standard Proctor maximum dry density (SPMDD) and using several passes of the compaction equipment.

Layer “B”

Fill placed for embedment of the tanks, or “Layer B”, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular B Type I or II, and in accordance with the above-mentioned drawings and *Table 1- Fill Material Summary*. The fill should be placed in maximum 200 mm thick loose lifts and compacted by suitable compaction equipment to achieve a minimum compaction of 99% of the materials SPMDD. Compaction between the chambers using a vibratory diesel plate.

Embedment fill material should be placed from outside the excavation footprint using an excavator with a long boom reach or stone-slinger and the equipment should not be situated over the chambers. Use of a dozer to push embedment stone between the rows of chambers may cause damage to the chamber and is not permitted unless considered suitable by the manufacturer.

A 40mil LLDPE thermoplastic liner, reviewed and approved by the manufacturer, will be provided along the walls of the excavation and 229 mm below the bottom of the chamber. The thermoplastic liner should be wrapped with 12-ounce non-woven geotextile, such as Terrafix 1200R or equivalent other reviewed and approved by Paterson, on both sides of the thermoplastic liner and above the topmost layer of the embedment fill, as per the manufacturer recommendations.

Layer “C”

According to the structural analysis provided, the system is to be provided with a minimum 450 mm fill material overlaying the embedment fill, or “Layer B”, and as indicated in the drawings attached to this document and *Table 1 - Fill Material Summary*.

It is recommended that Layer “C” consists of a minimum 300 mm thick layer of OPSS Granular A or Granular B Type I. The fill material is recommended to be placed in maximum 300 mm thick loose lifts and compacted using a suitably sized vibratory smooth drum roller to a minimum of 99% of the standard Proctor maximum dry density (SPMDD) and using several passes of the compaction equipment.

Layer “D”

The material used to backfill up to the proposed finished grade, “Layer D”, should consist of site-generated and sub-excavated sand fill consisting of clean silty sand in landscaped areas.

These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Compaction of these materials should be complete using a suitably sized smooth drum roller to a minimum of 95% of the standard Proctor maximum dry density (SPMDD) making a suitable number of passes and under the supervision of Paterson field personnel.

The fill material is recommended to be placed in dry and above-freezing conditions. Frozen fill material that is placed during winter months will thaw and settle more than is expected to be considered throughout the finished surface. Preparation and placement of the fill material is recommended to be verified and approved by Paterson field personnel at the time of construction.

OPSS Granular A crushed stone should be used to build up the base course below asphalt in paved areas. This fill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 99% of the materials SPMDD.

Table 1 - Fill Material Summary

Layer	Elevation (m)	Minimum Thickness (mm)	Material Description
D	64.92-Finished Grade	As Required to Meet Finished Grading	**Site Generated Sand Fill -** Placed in maximum 300 mm and compacted to 95% SPMDD.
C	64.47-64.92	450	**OPSS Granular A or Granular B Type I-** Placed in maximum 300 mm thick and compacted to 99% SPMDD.
Non-Woven Geotextile – According to manufacturer specifications			
B	63.03-64.47	1,400	**OPSS Granular B Type I or II** - Placed in maximum 200 mm thick and compacted to 99% SPMDD with a vibratory plate.
Woven Geotextile – According to manufacturer specifications			
A	62.12-63.03	900	**OPSS Granular B Type I or II** – Placed in maximum 150 mm thick and compacted to 99% SPMDD. **LINER–** 40mil LLDPE thermoplastic liner (using materials identified herein and with heat welded joints) along walls and 229 mm below bottom of chamber, wrapped with non-woven geotextile on both sides of the thermoplastic liner (Terrafix 1200R or equivalent other reviewed by Paterson), and according to manufacturer specifications.
SUBGRADE - Either approved fill, in-situ, or OPSS Granular B Type II material placed on in-situ soil or fill.			

Bearing Resistance Values

The proposed underground stormwater tank system is expected to be founded on the existing compact fill material consisting of silty sand with trace amounts of gravel. Based on the above, a bearing resistance value for the proposed structure may be considered to be **120 kPa** (SLS) and a factored bearing resistance value at ULS of **180 kPa** may be considered for the system and associated infrastructure/structures.

It is recommended that the existing fill be proof-rolled under dry conditions and above freezing temperatures by an adequately sized sheepsfoot roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by Paterson personnel at the time of construction.

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

Structures bearing on a subgrade medium prepared as indicated 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.

The bearing medium is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the existing fill 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.

Pavement Design

If required, the flexible pavement structure presented in Table 2 and Table 3 should be used for at grade access lanes and car-only parking areas. Any landscaped and hardscaped areas intended for pedestrian traffic are recommended to be reviewed by Paterson from a geotechnical perspective to ensure adequate drainage and support is provided by the proposed fill layers.

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

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

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

Table 3 - Recommended Asphalt Pavement Structure - Access Lanes

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.	

Excavation Side Slope

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.

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Groundwater Control

Based on Paterson review, it is expected that the excavation will be completed through fill material or silty sand soils and above the groundwater table elevation. Therefore, it is expected that water takings for excavations undertaken throughout the fill material and silty sand will mostly consist of surface water resulting from precipitation and snowmelt. These water takings should be manageable using open sumps and are not expected to result in dewatering that would impact neighbouring structure and infrastructure from a geotechnical perspective.

Any temporary dewatering during excavation and construction of the proposed underground tank system will take place within a limited range of the excavation area and is not expected to negatively impact the neighbouring structures.

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 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.

Precautions must be taken where excavations are carried out in proximity of existing structures which may be adversely affected due to the freezing conditions. 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.

3.0 Conclusion and Recommendations

From a geotechnical perspective, the proposed underground prefabricated watertight stormwater tank system is considered acceptable from a geotechnical perspective. It is recommended that Paterson field personnel complete inspection of the following items at the time of construction:

- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Periodic observation of the condition of unsupported excavation side slope in excess of 3 m in height, if applicate.
- Review confirmation of assumptions of the founding conditions for existing adjacent structures prior to construction.
- Inspection of the installation of the geotextile liners, Stormtech tanks and associated fill layers.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
July 28, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Fernanda Carozzi, PhD. Geoph.

Drew Petahtegoose, P.Eng.

Attachments:

- Soil Profile and Test Data sheets
- Symbols and Terms
- Borehole logs by Others
- Underground Stormwater Tank System Design Drawings prepared by ADS.
- Technical Note TN6.50 Thermoplastic Liners for Detention Systems
- Terrafix Geomembrane 40mil LLDPE Smooth – Technical Data Sheet
- Structural Analysis Results for StormTech MC-3500 System – Lansdowne 2.0 prepared by ADS
- Lansdowne 2.0 – Risk Management Plan Section B & C – DWG No. RM04 Revision 4 dated June 20, 2025, prepared by WSP
- Drawing PG6655-3 Test Hole Location Plan

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List of Services
Geotechnical Engineering ◊ Environmental Engineering ◊ Hydrogeology
Materials Testing ◊ Retaining Wall Design ◊ Rural Development Design
Temporary Shoring Design ◊ Building Science ◊ Noise and Vibration Studies

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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368851.35 **NORTHING:** 5029165.32 **ELEVATION:** 64.61

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: October 09, 2024

FILE NO. : PG6655

HOLE NO. : BH 1-24

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL 0.15m [64.45m]

FILL: Brown silty sand, trace gravel and topsoil 4.50m [60.11m]

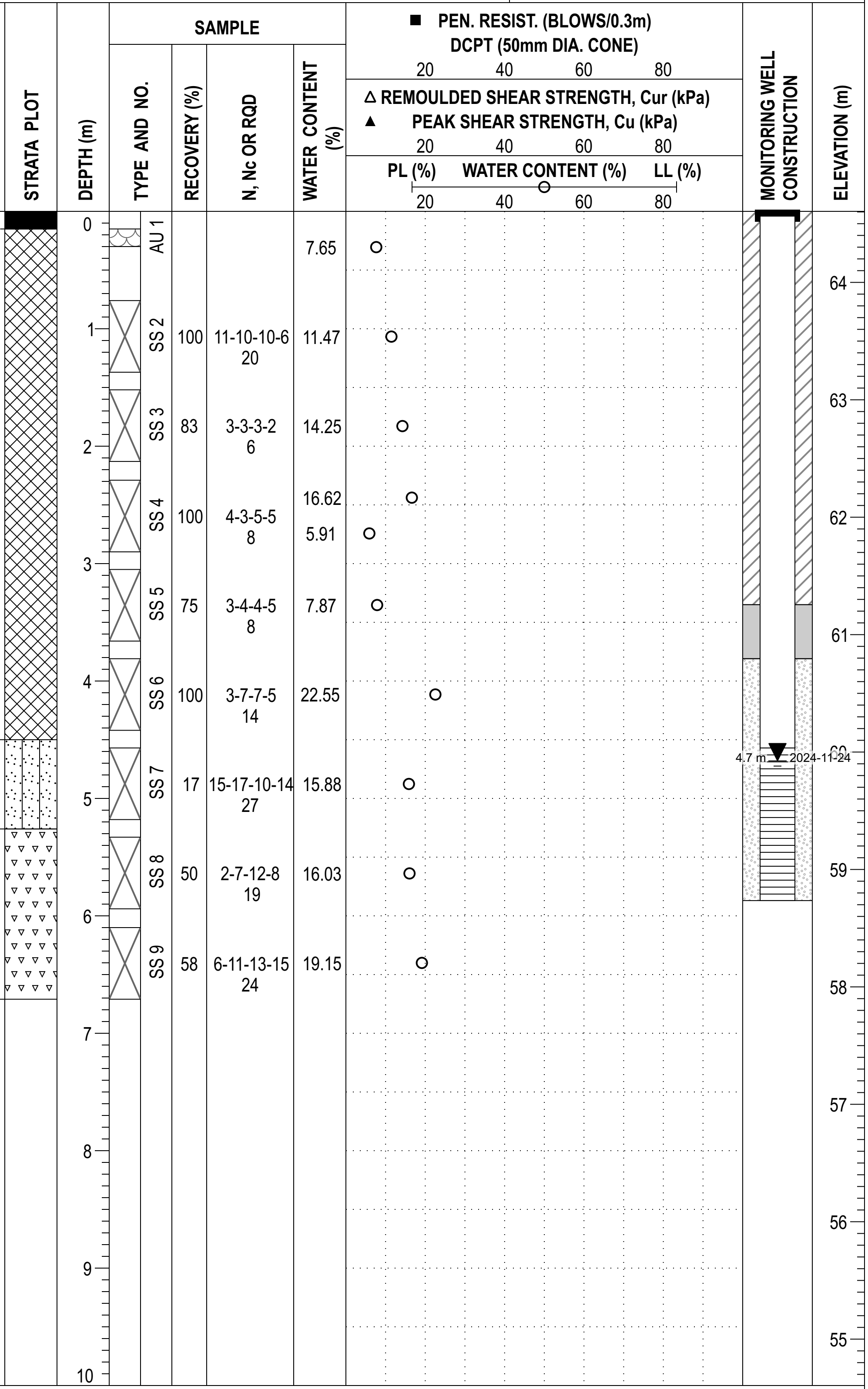
Compact, brown **SILTY SAND**, trace gravel 5.26m [59.35m]

GLACIAL TILL: Compact, brown sand, trace silt, gravel, cobbles and boulders 6.71m [57.90m]

End of Borehole

(GWL at 4.68 m depth - November 24, 2024)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			7.65
SS 2	100	11-10-10-6 20	11.47
SS 3	83	3-3-3-2 6	14.25
SS 4	100	4-3-5-5 8	16.62 5.91
SS 5	75	3-4-4-5 8	7.87
SS 6	100	3-7-7-5 14	22.55
SS 7	17	15-17-10-14 27	15.88
SS 8	50	2-7-12-8 19	16.03
SS 9	58	6-11-13-15 24	19.15



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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368877.23 **NORTHING:** 5029151.57 **ELEVATION:** 64.65

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

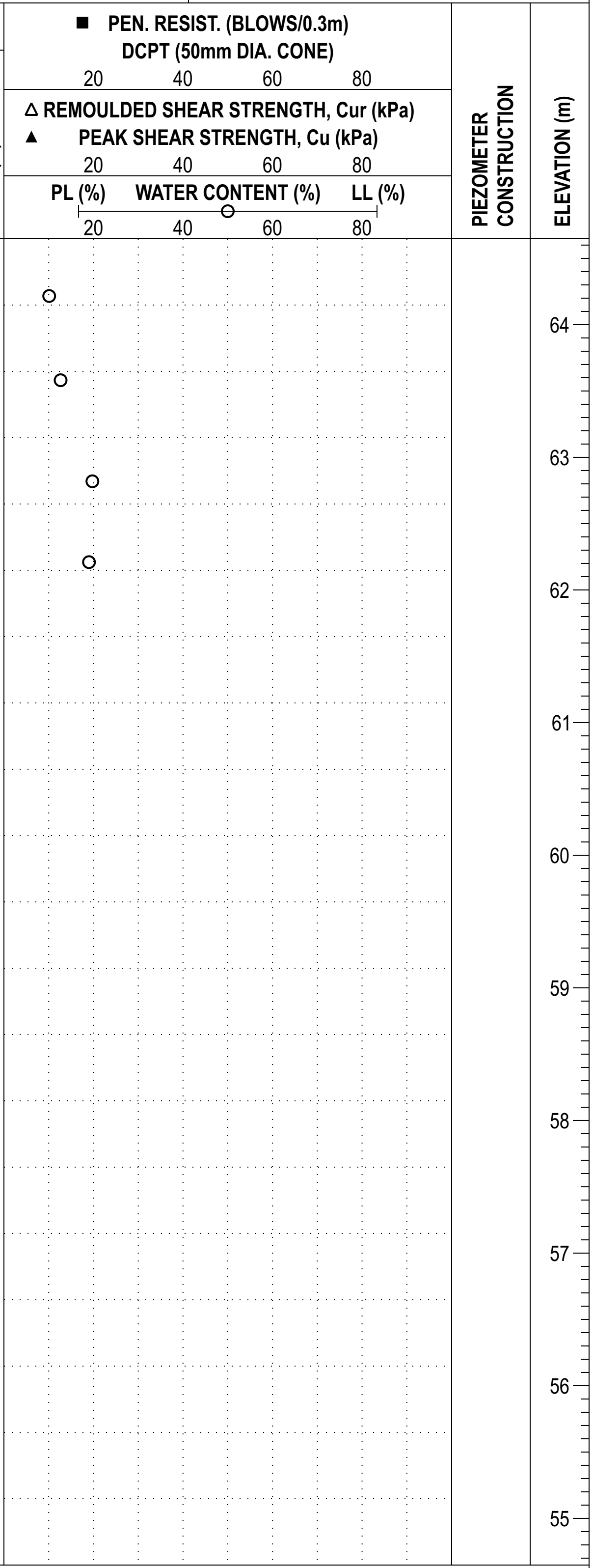
REMARKS:

DATE: October 09, 2024

FILE NO. : PG6655

HOLE NO. : BH 2-24

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE	0				
TOPSOIL 0.30m [64.35m]					
FILL: Brown, silty sand, trace gravel and organics		AU 1			10.11
	1	SS 2	67	6-5-7-5 12	12.66
	2	SS 3	75	3-4-3-4 7	19.76
2.90m [61.75m]		SS 4	0	12-25-13-17 38	19.0
End of Borehole Practical refusal to augering at 2.90 m depth	3				



SOIL PROFILE AND TEST DATA
Geotechnical Investigation
Lansdowne Park Redevelopment

COORD. SYS.: MTM ZONE 9 **EASTING:** 368879.56 **NORTHING:** 5029150.20 **ELEVATION:** 64.65

PROJECT: Proposed North Stands

BORINGS BY: CME-55 Low Clearance Drill

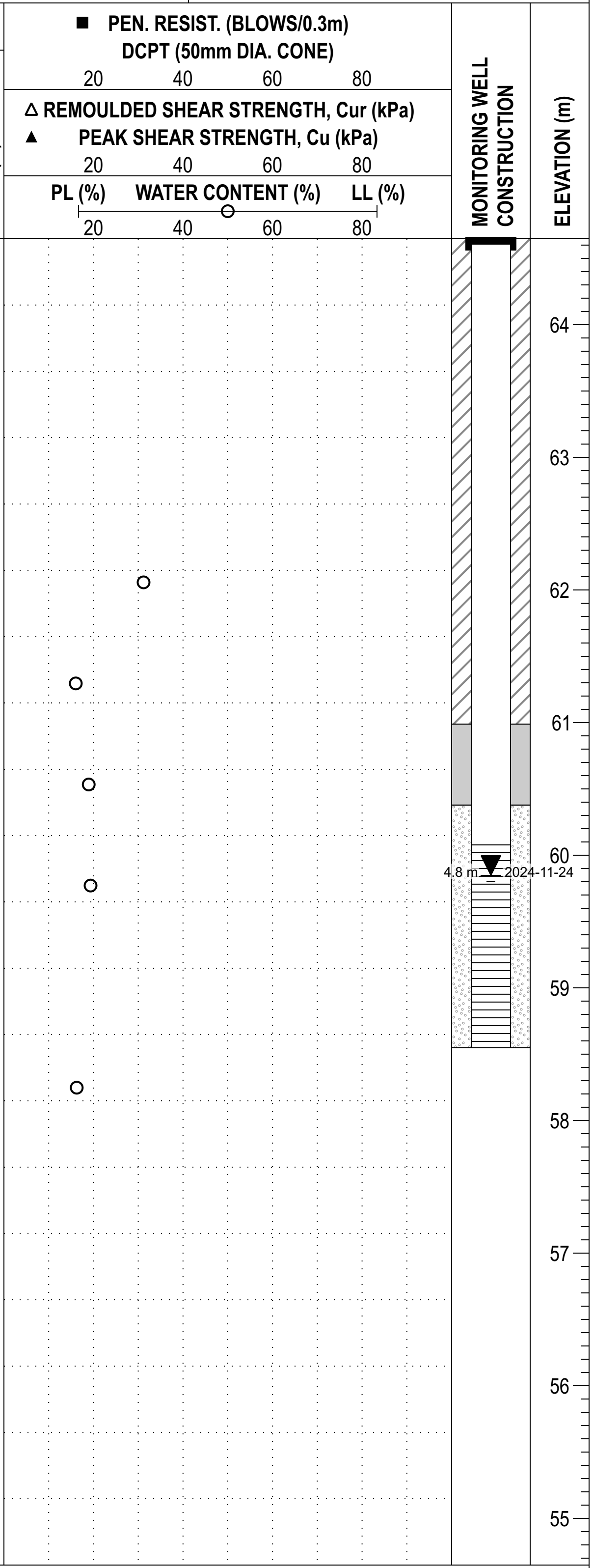
REMARKS:

DATE: October 09, 2024

FILE NO. : PG6655

HOLE NO. : BH 2A-24

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE	0				
Refer to BH 2-24 for soil profile					
2.29m [62.36m]	2.29				
FILL: Brown silty sand, with organics, trace gravel		SS 1	12	8-20-51-10 71	31.19
		SS 2	25	7-3-7-9 10	16.05
		SS 3	0	9-7-4-3 11	18.94
4.50m [60.15m]	4.50				
Loose, brown **SILTY SAND**, trace gravel		SS 4	50	3-2-3-3 5	19.35
		SS 5	33	6-3-3-3 6	
6.02m [58.63m]	6.02				
GLACIAL TILL: Compact, brown silty sand, with gravel, cobbles and boulders		SS 6	58	5-5-7-6 12	16.26
6.71m [57.94m]	6.71				
End of Borehole (GWL at 4.8 m depth - November 24, 2024)					



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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

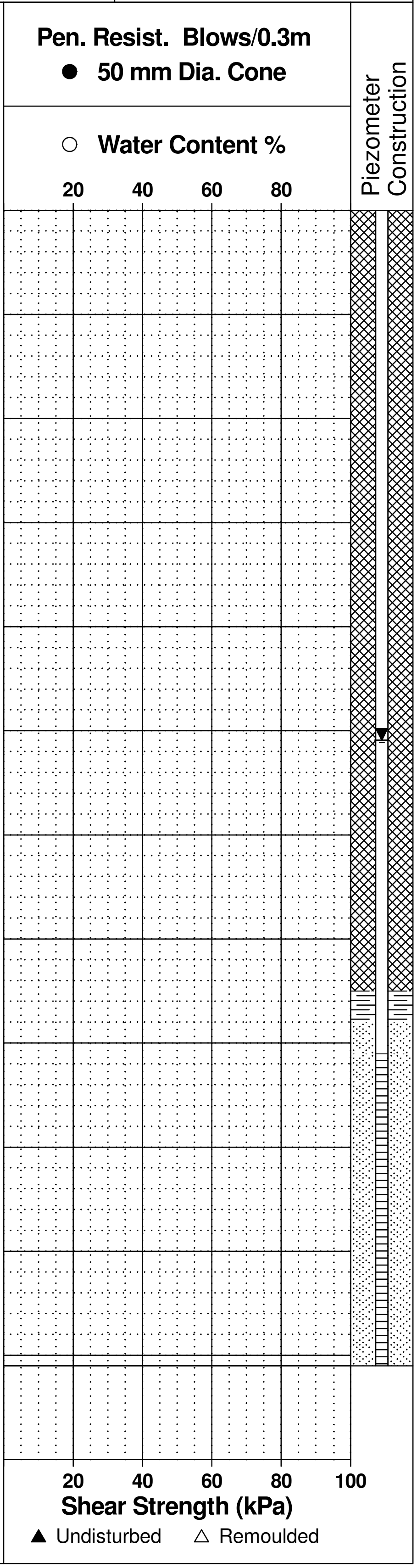
BORINGS BY CME-55 Low Clearance Drill

DATE October 25, 2021

FILE NO. PG5792

HOLE NO. BH 1-21

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



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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 11, 2021

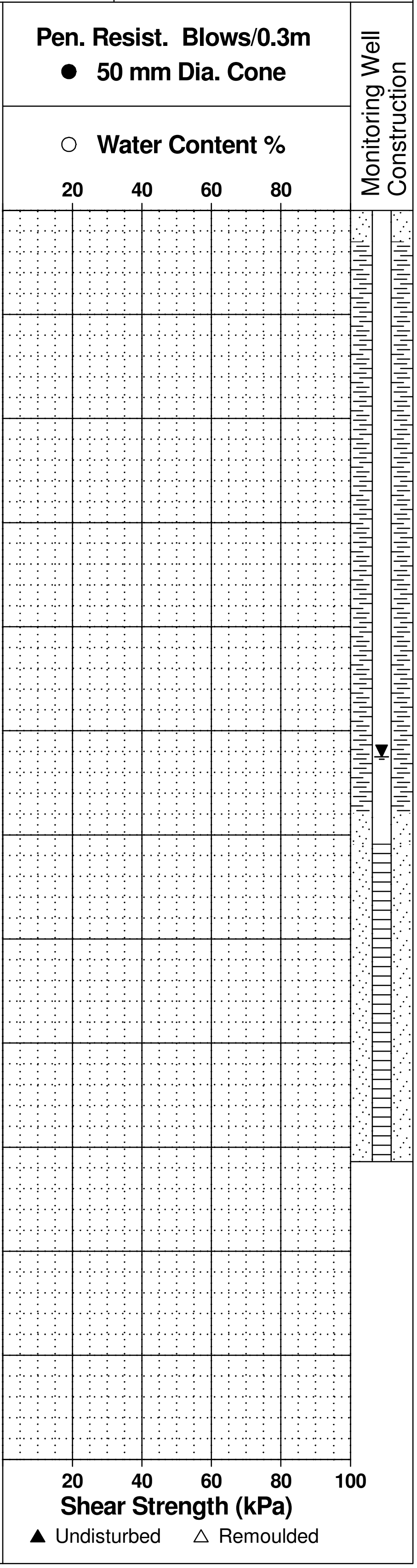
FILE NO. PG5792

HOLE NO. BH 6-21

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

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

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



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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE November 11, 2021

FILE NO. PG5792

HOLE NO. BH 6-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
	SS	17		50+	12	53.14
					13	52.14
	RC	3	34			
					14	51.14
	SS	18	52	41		
	RC	4	19			
					15	50.14
	SS	19	86	50+		
					16	49.14
GLACIAL TILL: Dense, grey silty sand with gravel, cobbles and boulders	RC	5	0			
					17	48.14
- some clay by 16.8m depth	SS	20	50	28		
					18	47.14
	RC	6	11			
					19	46.14
					20	45.14
	SS	21	0	50+		
	RC	7	14			
					21	44.14
	SS	22	0	50+		
					22	43.14
	RC	8	35			
22.88						
					23	42.14
BEDROCK: Good to excellent quality, grey limestone with occasional shale partings	RC	9	100	85		
					24	41.14

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

○ **Water Content %**

20 40 60 80

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

Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

FILE NO. **PG5792**

REMARKS

HOLE NO. **BH 6-21**

BORINGS BY CME-55 Low Clearance Drill **DATE** November 11, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						24	41.14		
BEDROCK: Good to excellent quality, grey limestone with occasional shale partings		RC	10	100	98	25	40.14		
25.73 End of Borehole (GWL @ 5.25m - Nov. 16, 2021)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Landsdowne Urban Park
Ottawa, Ontario

DATUM TBM - Top of manhole located near the southeast corner of the Aberdeen Pavilion. Geodetic elevation = 64.75m.

REMARKS

BORINGS BY CME 55 Power Auger

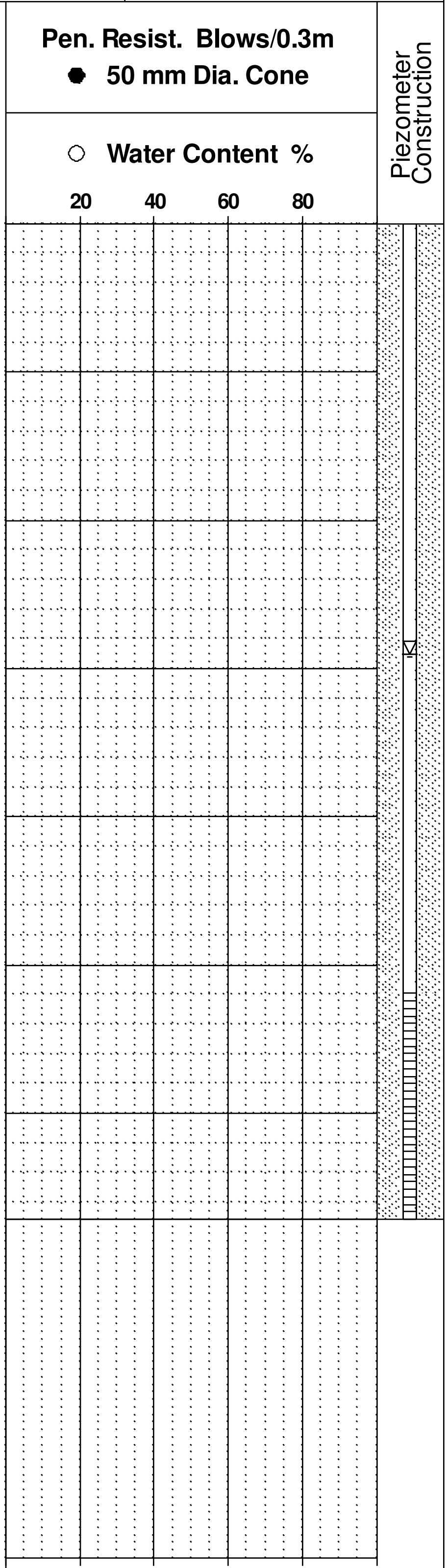
DATE 17 April 2013

FILE NO. PG2880

HOLE NO. BH 2-13

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
ASPHALTIC CONCRETE 0.05				
FILL: Sand with gravel 0.69	AU	1		
FILL: Brown sand with gravel, silt and clay 1.45	SS	2	67	27
FILL: Brown sand with gravel, silt and debris, trace organics 2.21	SS	3	63	5
FILL: Brown silty sand with gravel, clay and ceramic 2.97	SS	4	75	3
FILL: Organics with wood and sand 3.74	SS	5	100	2
Compact to very loose, brown-grey **SILTY SAND**	SS	6	83	12
	SS	7	67	6
6.02	SS	8	83	2
Compact, brown **SAND** with gravel, trace silt 6.71	SS	9	83	11
End of Borehole (GWL @ 2.9m depth based on field observations)				

DEPTH (m)	ELEV. (m)
0	64.87
1	63.87
2	62.87
3	61.87
4	60.87
5	59.87
6	58.87



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Landsdowne Urban Park
Ottawa, Ontario

DATUM TBM - Top of manhole located near the southeast corner of the Aberdeen Pavilion. Geodetic elevation = 64.75m.

REMARKS

BORINGS BY CME 55 Power Auger

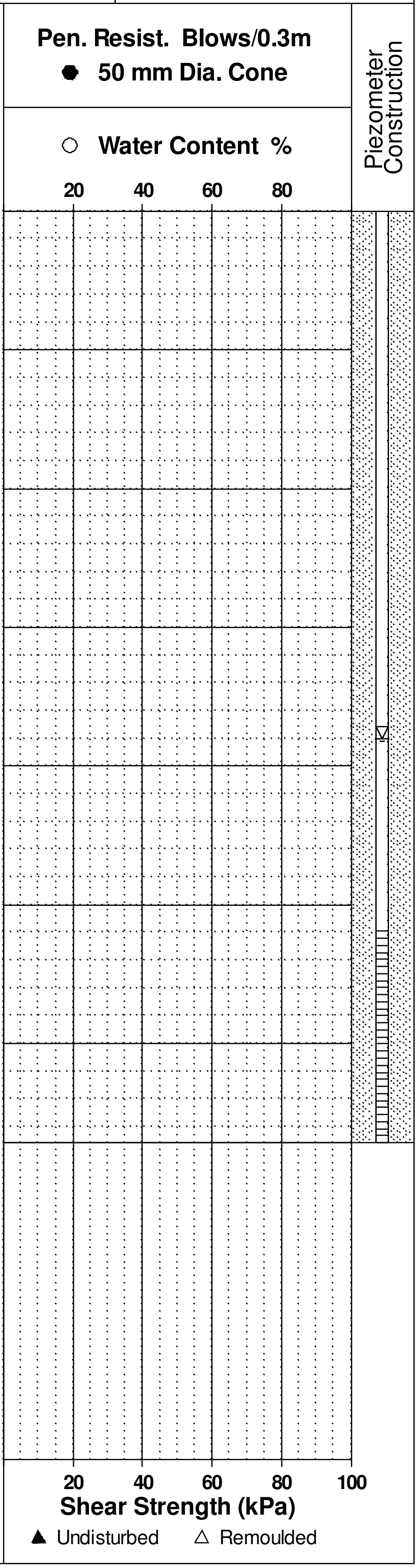
DATE 17 April 2013

FILE NO. PG2880

HOLE NO. BH 3-13

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown sand and gravel, trace cobbles	AU	1		
	SS	2	83	33
1.45				
FILL: Dark brown silty clay with organics, trace wood, glass and cermanic	SS	3	67	9
	SS	4	71	5
	SS	5	63	8
3.70				
Compact, brown **SILTY SAND,** trace organics	SS	6	63	15
	SS	7	67	20
5.26				
Loose, brown **SAND** wtih gravel, trace silt	SS	8	67	6
	SS	9	75	9
6.71				
End of Borehole				
(GWL @ 3.8m depth based on field observations)				

DEPTH (m)	ELEV. (m)
0	64.90
1	63.90
2	62.90
3	61.90
4	60.90
5	59.90
6	58.90



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Lansdowne Park Stormwater Management System
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 75 Power Auger

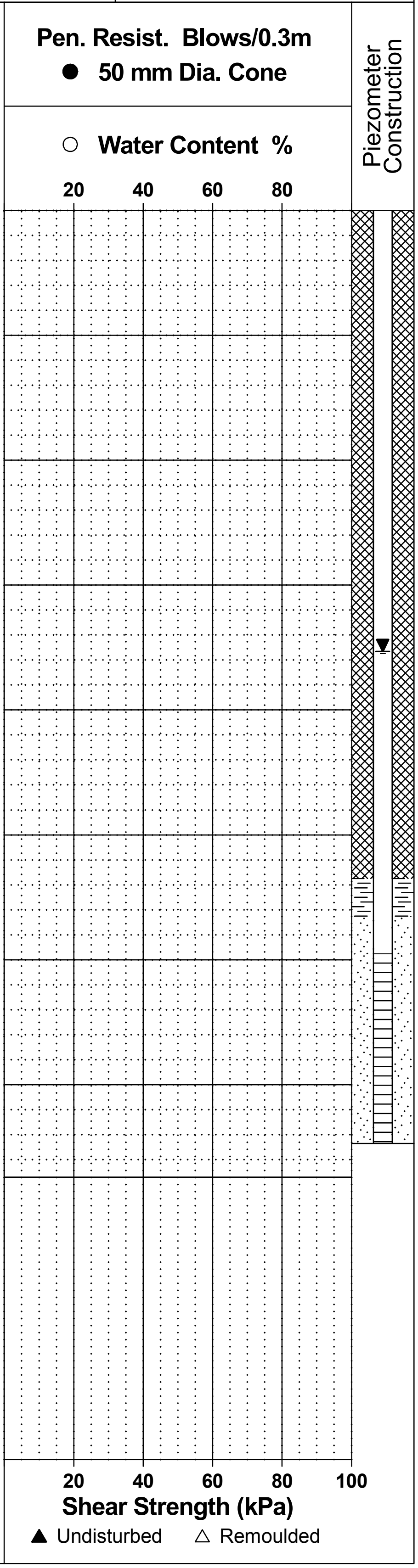
DATE September 8, 2010

FILE NO. **PG2207**

HOLE NO. **BH 3-10**

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Brown silty sand and gravel	0.61	AU	1		
Loose, brown **SILTY SAND**		SS	2	75	4
		SS	3	92	4
		SS	4	79	3
	3.73	SS	5	13	10
GLACIAL TILL: Loose to compact, brown sand with silt, gravel, cobbles and boulders		SS	6	75	7
		SS	7	71	3
		SS	8	75	15
		SS	9	100	17
	7.47	SS	10	100	17
End of Borehole (GWL @ 3.5m - Sep 13/10)					

DEPTH (m)	ELEV. (m)
0	64.75
1	63.75
2	62.75
3	61.75
4	60.75
5	59.75
6	58.75
7	57.75



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SOIL PROFILE & TEST DATA

Data Gap Analysis
Lansdowne Park (Ur-27)
Ottawa, Ontario

DATUM TBM: Fire Hydrant east of the Aberdeen Pavailion (geodetic elevation of 66.068m).

REMARKS

BORINGS BY CME 45 Power Auger

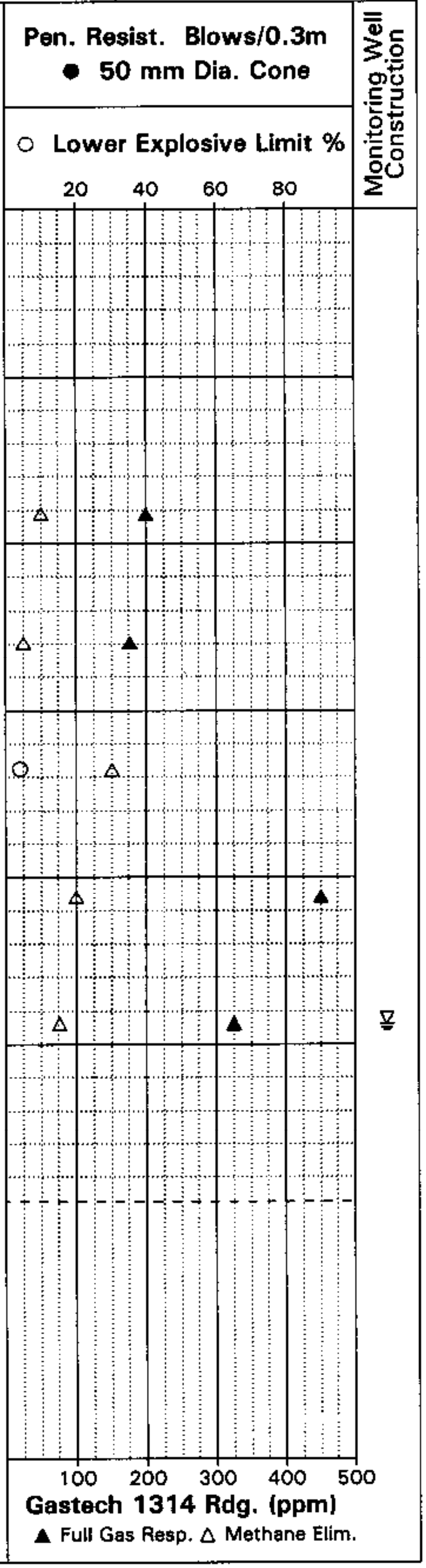
DATE 7 AUG 03

FILE NO. E2677

HOLE NO. BH03-2

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphalt	0.10				
FILL: Brown silty sand with grey gravel	1.52				
FILL: Brown silty sand - 0.1m thick layer of black oganic matter @ 3.4m	3.66	SS SS SS	1 2 3	21 50 58	5 7 3
Grey SILTY fine to coarse **SAND**	4.57	SS	4	50	14
Grey brown SANDY SILT	4.77				
Red brown SILTYfine to medium SAND	5.94	SS SS	5 6	50 42	9 6
End of Borehole (open hole GW @ 4.9m depth)					

DEPTH (m)	ELEV. (m)
0	64.86
1	63.86
2	62.86
3	61.86
4	60.86
5	59.86



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SOIL PROFILE & TEST DATA

Data Gap Analysis
Lansdowne Park (Ur-27)
Ottawa, Ontario

DATUM TBM: Fire Hydrant east of the Aberdeen Pavailion (geodetic elevation of 66.068m).

REMARKS

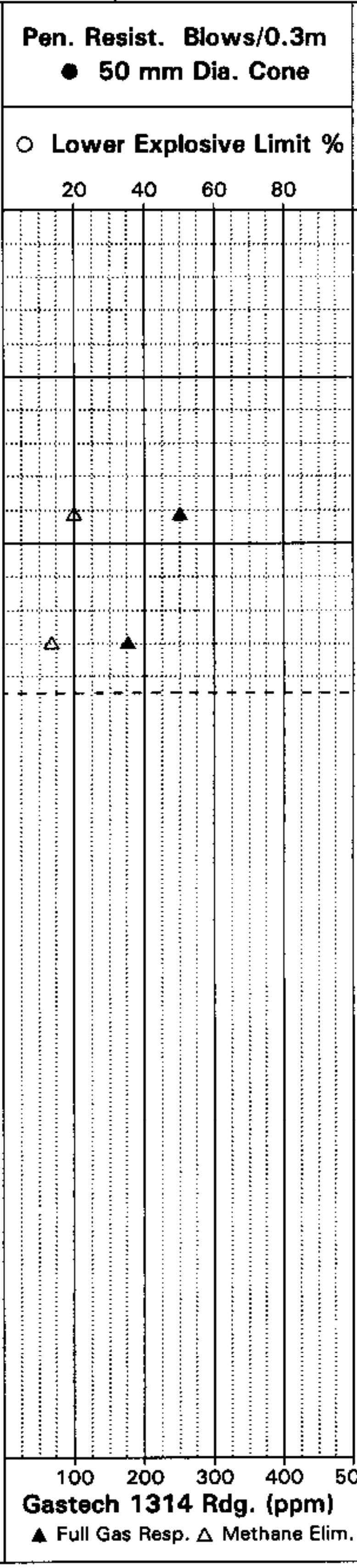
BORINGS BY CME 45 Power Auger

DATE 7 AUG 03

FILE NO. **E2677**

HOLE NO. **BH03-4**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.84
Asphalt 0.08							
FILL: Brown silty sand with grey gravel 1.37						1	63.84
Light brown SANDY SILT		SS	11	62	6		
						2	62.84
		SS	12	54	10		
2.90							
End of Borehole							



Monitoring Well Construction

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SOIL PROFILE & TEST DATA

Data Gap Analysis
Lansdowne Park (Ur-27)
Ottawa, Ontario

DATUM TBM: Fire Hydrant east of the Aberdeen Pavailion (geodetic elevation of 66.068m).

REMARKS

BORINGS BY CME 45 Power Auger

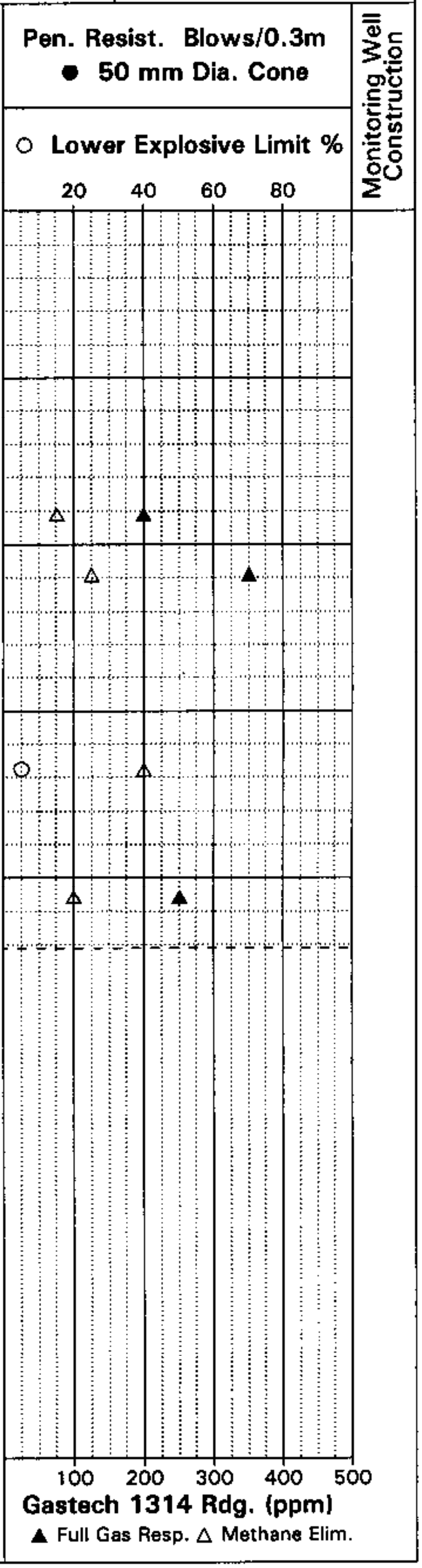
DATE 7 AUG 03

FILE NO. **E2677**

HOLE NO. **BH03-5**

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE						
Asphalt	0.10					
FILL: Brown silty sand with grey gravel	1.22					
FILL: Grey silty sand	2.13		SS	13	29	13
FILL: Black sandy silt with some clay, wood pieces, glass, twigs and organic matter	3.20		AU SS	14 15	 0	 2
Grey **SILTY** fine to coarse **SAND**	4.42		SS SS	16 17	50 38	6 19
End of Borehole						

DEPTH (m)	ELEV. (m)
0	64.65
1	63.65
2	62.65
3	61.65
4	60.65



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SOIL PROFILE & TEST DATA

Data Gap Analysis
Lansdowne Park (Ur-27)
Ottawa, Ontario

DATUM TBM: Fire Hydrant east of the Aberdeen Pavailion (geodetic elevation of 66.068m).

REMARKS

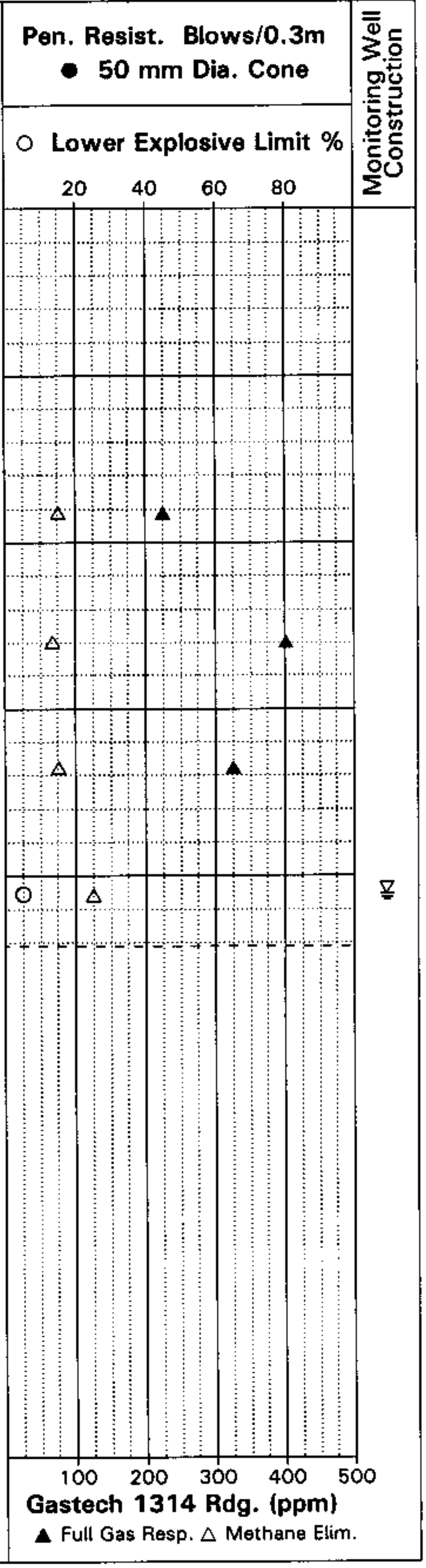
BORINGS BY CME 45 Power Auger

DATE 7 AUG 03

FILE NO. **E2677**

HOLE NO. **BH03-6**

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	64.89
Asphalt	0.08							
FILL: Brown silty sand with grey gravel	1.37						1	63.89
Brown SILTY SAND			SS	18	38	7	2	62.89
			SS	19	50	7	3	61.89
- light grey by 3.4m depth			SS	20	25	2		
	4.42		SS	21	58	11	4	60.89
End of Borehole (open hole GW@4.1m depth)								



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SOIL PROFILE & TEST DATA

Data Gap Analysis
Lansdowne Park (Ur-27)
Ottawa, Ontario

DATUM TBM: Fire Hydrant east of the Aberdeen Pavailion (geodetic elevation of 66.068m).

REMARKS

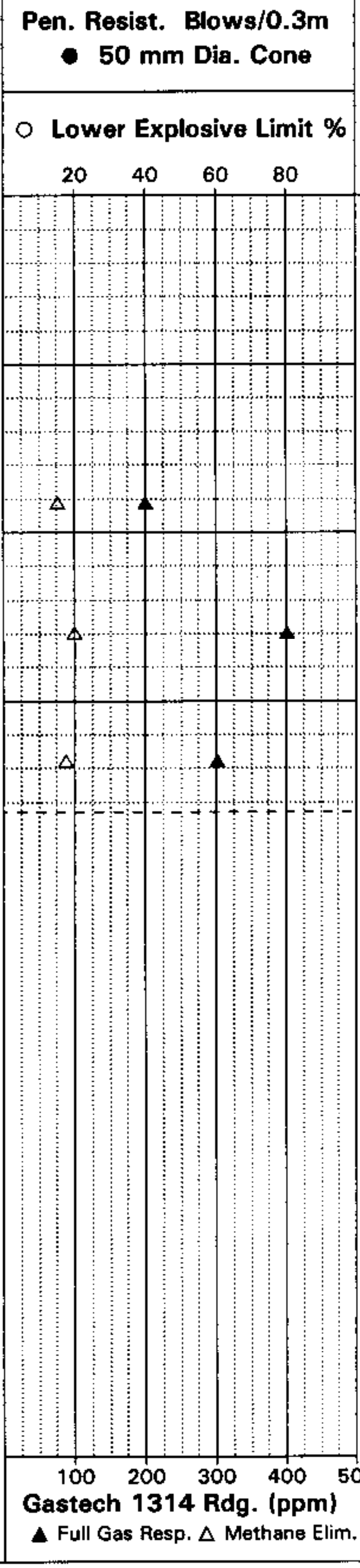
BORINGS BY CME 45 Power Auger

DATE 7 AUG 03

FILE NO. E2677

HOLE NO. BH03-7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.87
Asphalt 0.08							
FILL: Brown silty sand with grey gravel 0.91						1	63.87
FILL: Brown silty sand 2.13		SS	22	25	4	2	62.87
Light brown SILTY SAND		SS	23	58	9	3	61.87
3.66		SS	24	21	11		
End of Borehole							



Monitoring Well Construction

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28 Concourse Gate, Unit 1, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Phase II Environmental Site Assessment
Lansdowne Park, Bank Street
Ottawa, Ontario

DATUM TBM - Top of fire hydrant located 23m east of Aberdeen Pavilion. Assumed elevation = 100.00m.

REMARKS

BORINGS BY Truck-mount Drill

DATE 22 December 1998

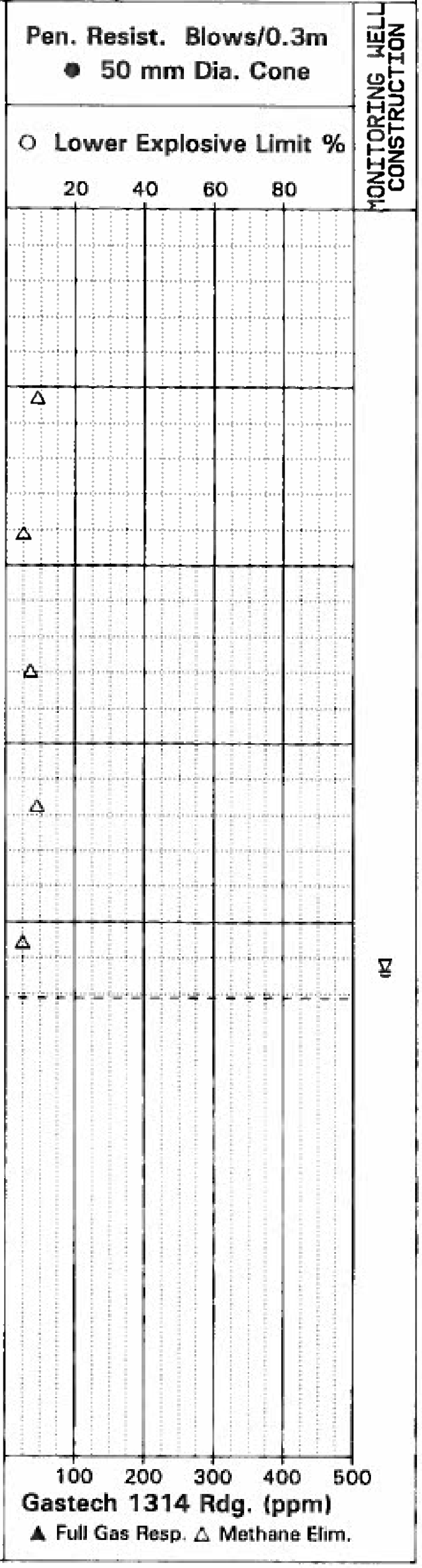
FILE NO. E1525

HOLE NO. BH26

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	98.70
Asphaltic concrete	0.10							
Grey crushed stone	0.28							
FILL: Brown sand, some silt and clay	1.45		SS	57	62	32	1	97.70
			SS	58	50	10	2	96.70
FILL: Brown silty sand with clay, cinders, metal pieces such as wire and strapping			SS	59		6	3	95.70
	3.73		SS	60		8		
Loose, brown SILTY SAND, trace of gravel	4.42		SS	61	67	28	4	94.70
End of Borehole								

(Open hole WL @ 4.27m depth)

Waste debris present between 1.45m and 3.73m depth.



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

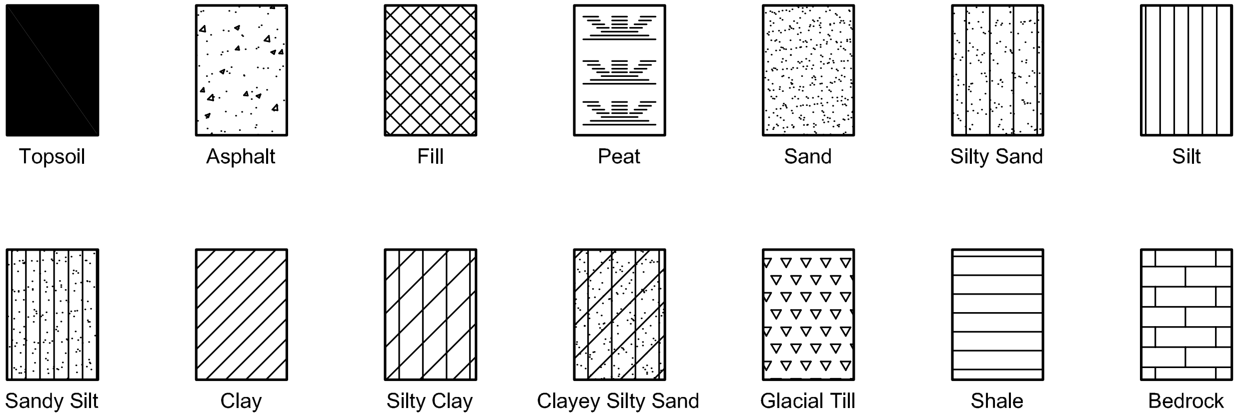
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

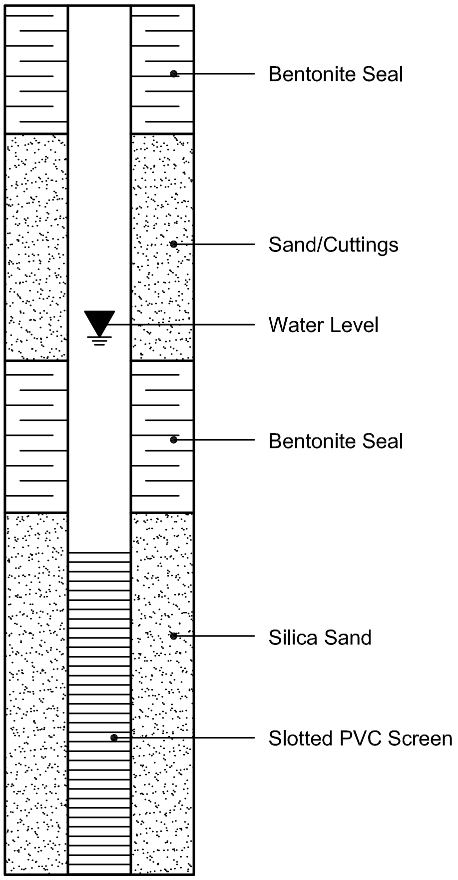
SYMBOLS AND TERMS (continued)

STRATA PLOT

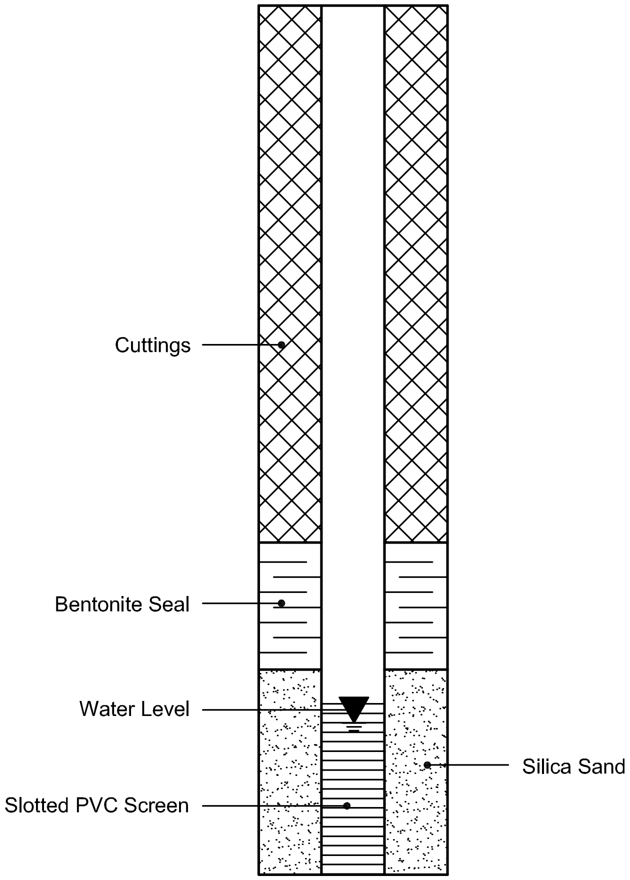


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION

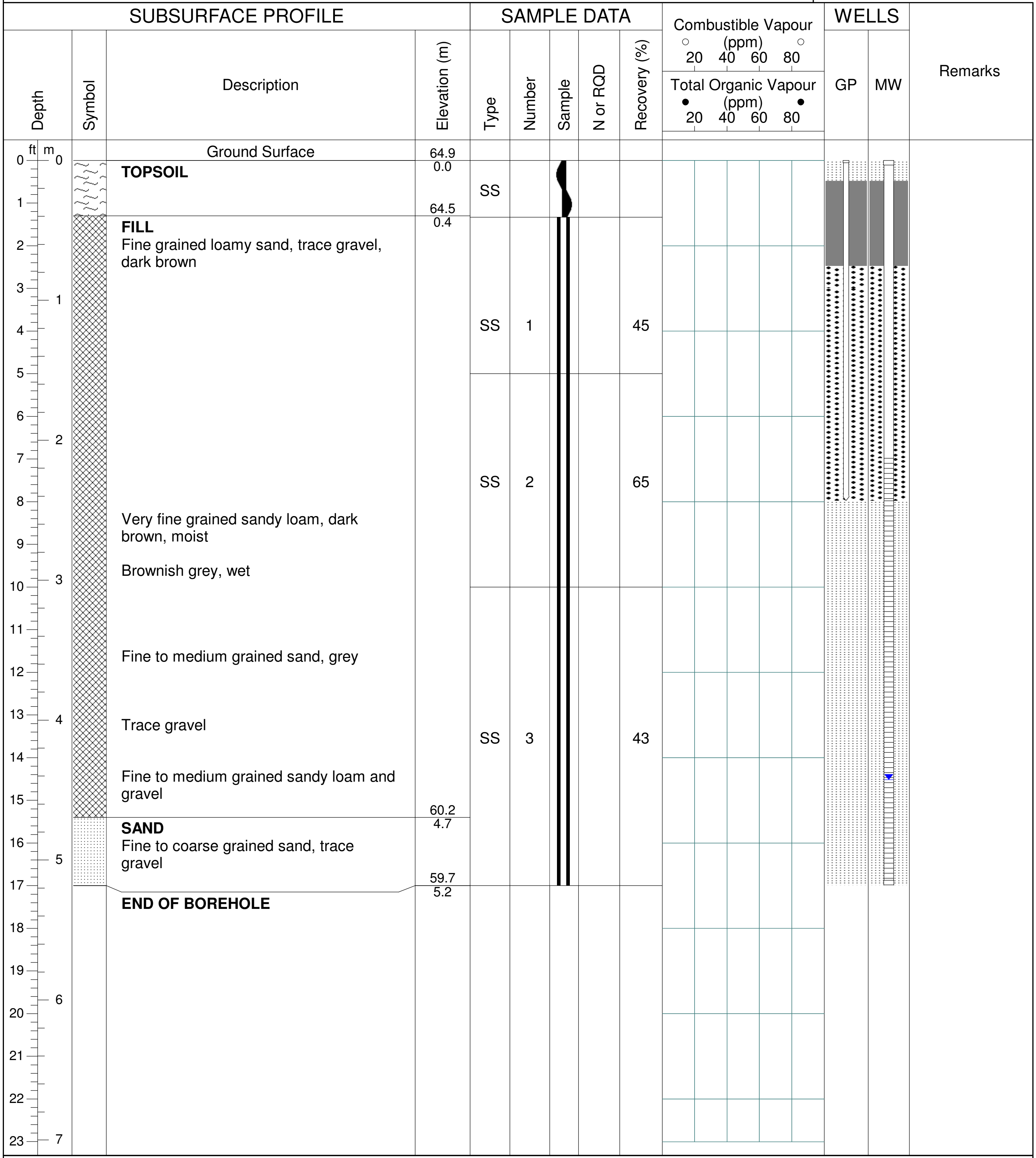


Stratigraphic and Instrumentation Log: MW15-6 / GP15-10

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

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

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



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

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

Elevation: 64.924 masl
Easting: 368843.807
Northing: 5029183.520

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

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

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

Stratigraphic and Instrumentation Log: MW15-7

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

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

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

SUBSURFACE PROFILE

Depth	Description	Elevation (m)
0 ft / 0 m	Ground Surface	64.51
0 ft / 0 m	**TOPSOIL**	0.00
1.3 ft / 0.40 m	**FILL** Gravel and sand, grey Fine loamy sand, greyish brown Wet Fine to medium grained sand, brown Fine grained sandy loam	64.12 0.40
12.2 ft / 3.71 m	**SAND** Fine to coarse grained sand, trace gravel, brown, wet Trace silt Slightly grey	60.80 3.71
20 ft / 6.10 m	**END OF BOREHOLE**	58.42 6.10

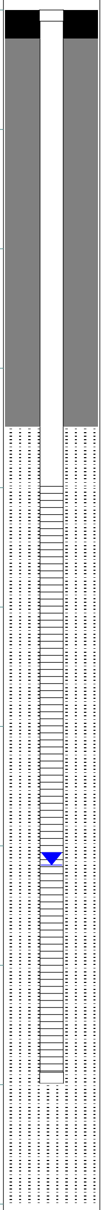
SAMPLE DATA

Type	Number	Sample	N or RQD	Recovery (%)
SS	1			68
SS	2			70
SS	3			65
SS	4			55

Combustible Vapour (ppm) ○ 250 750 1250

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

Monitoring Well Details



Remarks

Elevation: 64.513 masl
Easting: 368911.901
Northing: 5029169.410

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

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

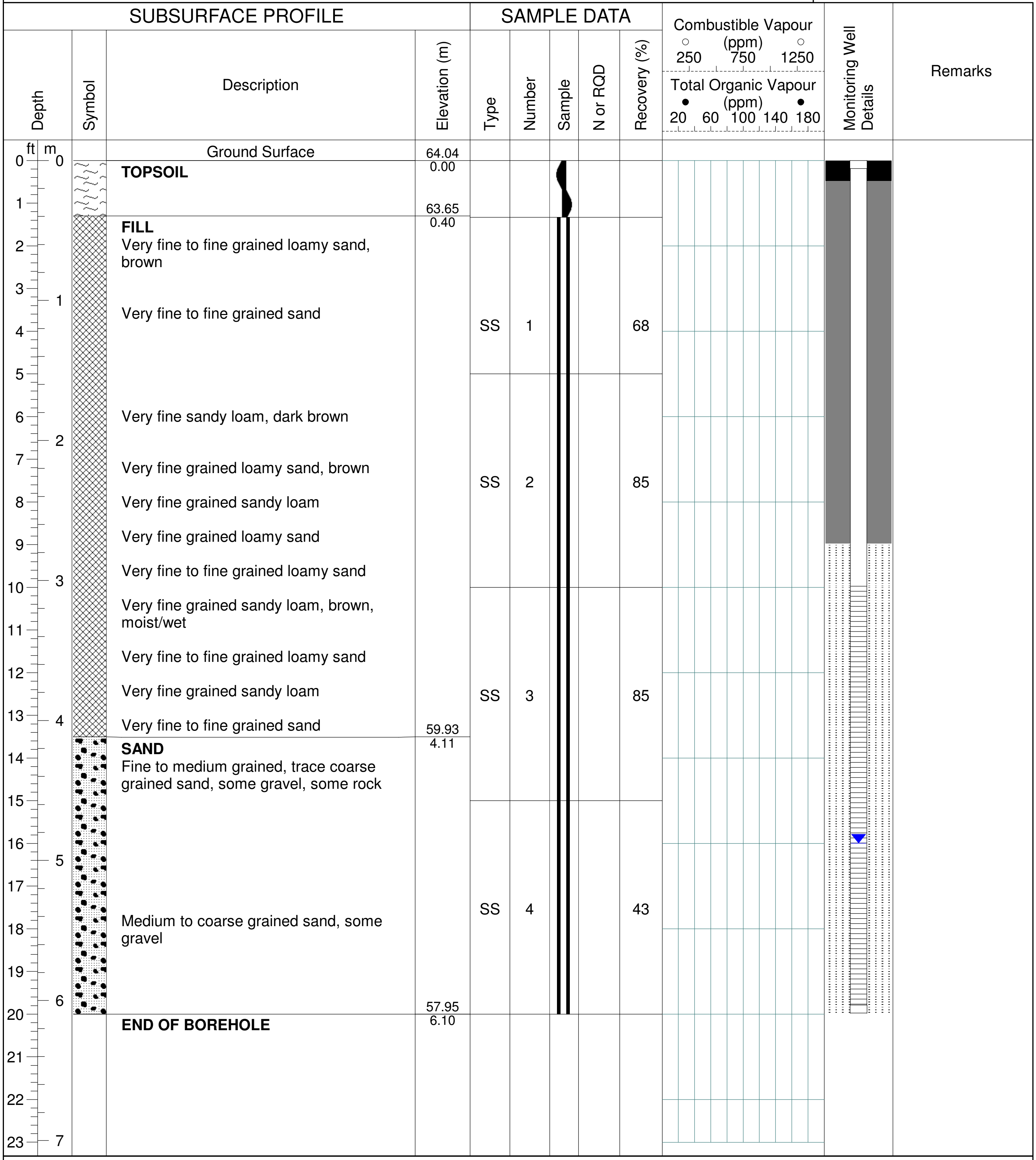
Datum: Geodetic
Checked by: KDH
Sheet: 1 of 1

Stratigraphic and Instrumentation Log: MW15-10

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

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

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



Description	Elevation (m)
Ground Surface	64.04
TOPSOIL	0.00
	63.65
FILL Very fine to fine grained loamy sand, brown	0.40
Very fine to fine grained sand	
Very fine sandy loam, dark brown	
Very fine grained loamy sand, brown	
Very fine grained sandy loam	
Very fine grained loamy sand	
Very fine to fine grained loamy sand	
Very fine grained sandy loam, brown, moist/wet	
Very fine to fine grained loamy sand	
Very fine grained sandy loam	
Very fine to fine grained sand	59.93
SAND Fine to medium grained, trace coarse grained sand, some gravel, some rock	4.11
Medium to coarse grained sand, some gravel	57.95
END OF BOREHOLE	6.10

Type	Number	Recovery (%)
SS	1	68
SS	2	85
SS	3	85
SS	4	43

Elevation: 64.043 masl
Easting: 368878.435
Northing: 5029083.949

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

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

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

Stratigraphic and Instrumentation Log: BH10-3

Project No: TZ101001
Location: Lansdowne Park
Logged By: JFT
Drill Date: March 1, 2010
Hole Size: 200 mm

Project Name: Lansdowne Park
Client: City of Ottawa
Entered By: JFT
Drill Method: Hollow Stem Auger
Drilled By: George Downing Estate Drilling

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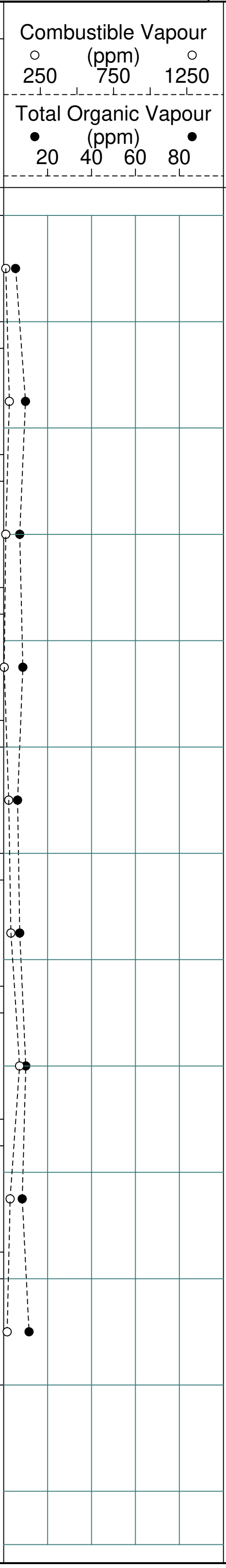
300-210 Colonnade Road
Ottawa, Ontario
K2E 7L5

SUBSURFACE PROFILE

Description	Elevation (m)
Ground Surface	0.00
ASPHALT	0.00
FILL Fine grained silty sand and gravel, dark brown, damp Fine grained sandy clayey silt, trace gravel, dark brown Fine grained sandy silt, trace gravel, dark brown Grey/black Waste: organics, glass, metal, ash/cinders Becomes moist Waste: organics, wood, ash/cinders Fine grained silty sand, dark grey	
SILTY SAND AND GRAVEL Fine to medium grained silty sand, gravel, grey moist Becomes wet, brown	-4.57 4.57
END OF BOREHOLE	-6.71 6.71

SAMPLE DATA

Type	Number	N or RQD	Recovery (%)	Remarks
AS	1	NA	N/A	
AS	2	NA	N/A	
SS	3	8	17	
SS	4	5	38	
SS	5	2	42	Sample submitted for PAH and Metals analyses.
SS	6	8	29	
SS	7	16	46	Sample submitted for PHC F1-F4 and BTEXS analyses.
SS	8	16	42	
SS	9	7	29	



Monitoring Well Details

Elevation: NA
Easting: 368890.60
Northing: 5029207.70

Casing Elevation: NA
Well Casing Size: NA
Vapour Unit: Eagle Gastech/Mini Rae

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

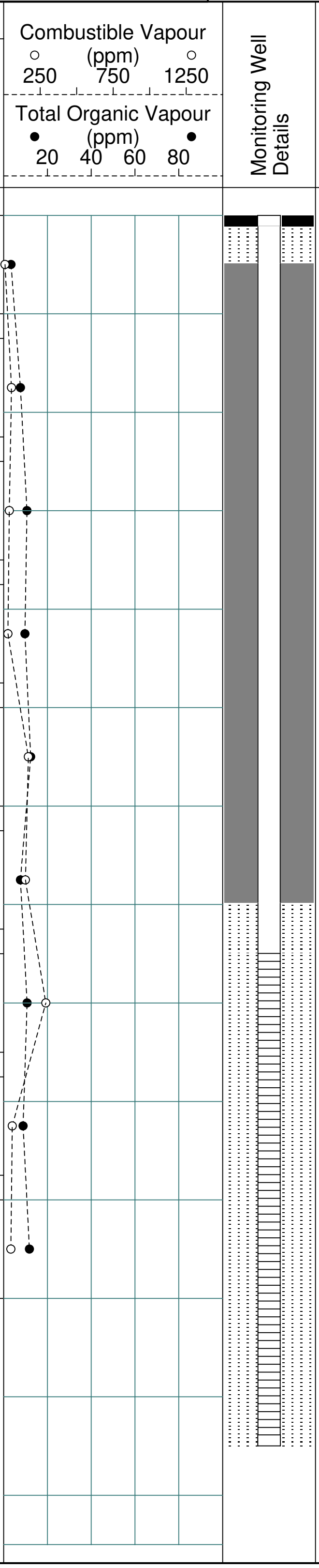
Stratigraphic and Instrumentation Log: MW10-10

Project No: TZ101001
Location: Lansdowne Park
Logged By: JFT
Drill Date: March 1, 2010
Hole Size: 200 mm

Project Name: Lansdowne Park
Client: City of Ottawa
Entered By: JFT
Drill Method: Hollow Stem Auger
Drilled By: George Downing Estate Drilling

300-210 Colonnade Road
Ottawa, Ontario
K2E 7L5

SUBSURFACE PROFILE			SAMPLE DATA				Remarks
Depth (ft / m)	Description	Elevation (m)	Type	Number	N or RQD	Recovery (%)	
0 / 0	Ground Surface	64.75 0.00					
	ASPHALT						
1	**FILL** Fine grained silty sand and gravel, grey, damp		AS	1	N A	N/A	
3 / 1	Trace clay		AS	2	N A	N/A	
6 / 2	Fine grained silty sand, grey		SS	3	7	46	
8	Waste: wood		SS	4	6	63	Sample submitted for PAH and Metals analyses.
10 / 3	**SILTY SAND** Fine to medium grained silty sand, grey	61.70 3.05	SS	5	8	38	
12.5 / 4	**SILTY SAND AND GRAVEL** Fine to medium grained silty sand and gravel, grey	60.94 3.81	SS	6	22	29	
15	Brown		SS	7	18	17	Sample submitted for PHC F1-F4 and BTEXS analyses.
17.5	Becomes wet		SS	8	7	50	
21			SS	9	13	54	
22.5 / 7	Flowing sands (did not sample)						
25	**END OF BOREHOLE**	57.13 7.62					



Elevation: 64.75
Easting: 368859.64
Northing: 5029217.086

Casing Elevation: 64.69
Well Casing Size: 32 mm
Vapour Unit: Eagle Gastech/Mini Rae

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

Stratigraphic and Instrumentation Log: MW10-11

Project No: TZ101001
Location: Lansdowne Park
Logged By: JFT
Drill Date: March 2, 2010
Hole Size: 200 mm

Project Name: Lansdowne Park
Client: City of Ottawa
Entered By: JFT
Drill Method: Hollow Stem Auger
Drilled By: George Downing Estate Drilling

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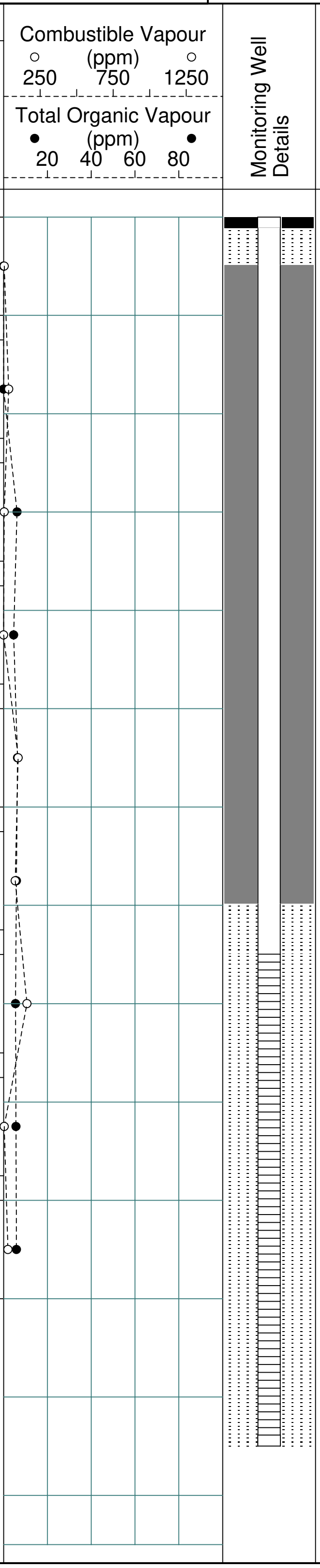
Ottawa, Ontario
K2E 7L5

SUBSURFACE PROFILE		SAMPLE DATA				
Description	Elevation (m)	Type	Number	N or RQD	Recovery (%)	Remarks
Ground Surface	64.89					
ASPHALT	0.00					
FILL Fine grained silty sand and gravel, grey, damp		AS	1	NA	N/A	
Trace gravel, brown		AS	2	NA	N/A	
Waste: wood		SS	3	10	33	
Fine grained silty sand, grey/brown Waste: concrete		SS	4	11	21	
Waste: wood, concrete		SS	5	11	29	Sample submitted for PAH analyses.
SILTY SAND Fine to medium grained silty sand, brown	61.08 3.81	SS	6	8	50	
Fine grained sandy silt, grey/brown						
SILTY SAND AND GRAVEL Fine to medium grained silty sand, gravel, brown, moist	60.32 4.57	SS	7	20	29	Sample submitted for PHC F1-F4 and BTEXS analyses.
Becomes wet		SS	8	14	46	
		SS	9	9	33	
Flowing sands (did not sample)						
END OF BOREHOLE	57.27 7.62					



Elevation: 64.89
Easting: 368895.383
Northing: 5029169.555

Casing Elevation: 64.8
Well Casing Size: 32 mm
Vapour Unit: Eagle Gastech/Mini Rae

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

Stratigraphic and Instrumentation Log: MW10-28

Project No: TZ101001
Location: Lansdowne Park
Logged By: JFT
Drill Date: March 18, 2010
Hole Size: 200 mm

Project Name: Lansdowne Park
Client: City of Ottawa
Entered By: JFT
Drill Method: Hollow Stem Auger
Drilled By: George Downing Estate Drilling

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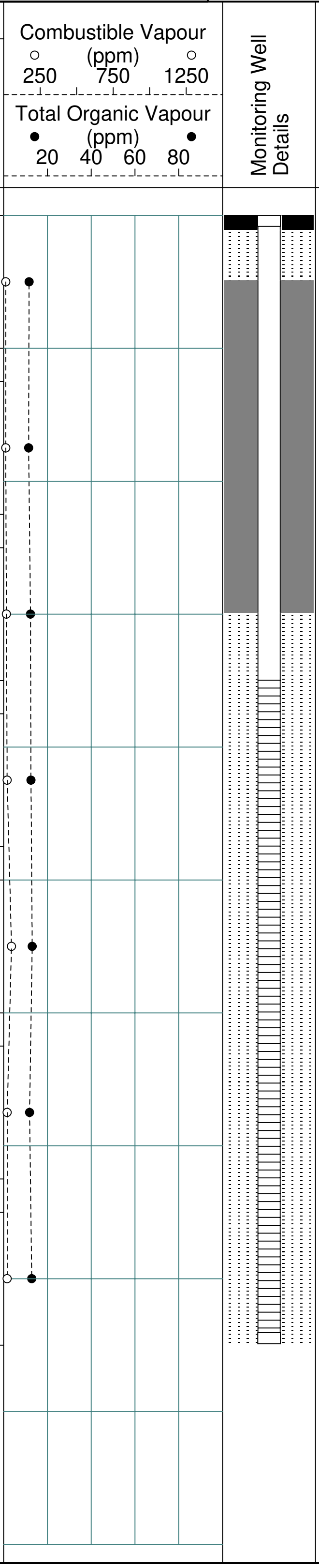
Ottawa, Ontario
K2E 7L5

SUBSURFACE PROFILE

Description	Elevation (m)
Ground Surface	64.81
ASPHALT	0.00
FILL Fine grained silty sand, some gravel, grey, damp Trace gravel, grey/brown	
SILTY SAND Fine grained silty sand, brown	63.29 1.52
SANDY SILT Fine grained sandy silt, trace organics, dark brown	62.22 2.59
SILTY SAND Fine grained silty sand, brown, moist	61.76 3.05
SANDY SILT Fine grained sandy silt, brown, moist	61.00 3.81
SILTY SAND Fine to medium grained silty sand, grey, wet	60.54 4.27
SANDY SILT Fine grained sandy silt, brown/grey, wet	60.24 4.57
END OF BOREHOLE	59.63 5.18

SAMPLE DATA

Type	Number	N or RQD	Recovery (%)	Remarks
AS	1	N A	N/A	
AS	2	N A	N/A	
SS	3	6	38	
SS	4	3	92	
SS	5	2	29	Sample submitted for PHC F1-F4 and BTEXS analyses.
SS	6	4	67	
SS	7	2	71	



Depth: ft 0–20; m 0–6

Elevation: 64.81
Easting: 368922.139
Northing: 5029188.699

Casing Elevation: 64.74
Well Casing Size: 32 mm
Vapour Unit: Eagle Gastech/Mini Rae

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

Stratigraphic and Instrumentation Log: MW10-29

Project No: TZ101001
Location: Lansdowne Park
Logged By: JFT
Drill Date: March 18, 2010
Hole Size: 200 mm

Project Name: Lansdowne Park
Client: City of Ottawa
Entered By: JFT
Drill Method: Hollow Stem Auger
Drilled By: George Downing Estate Drilling

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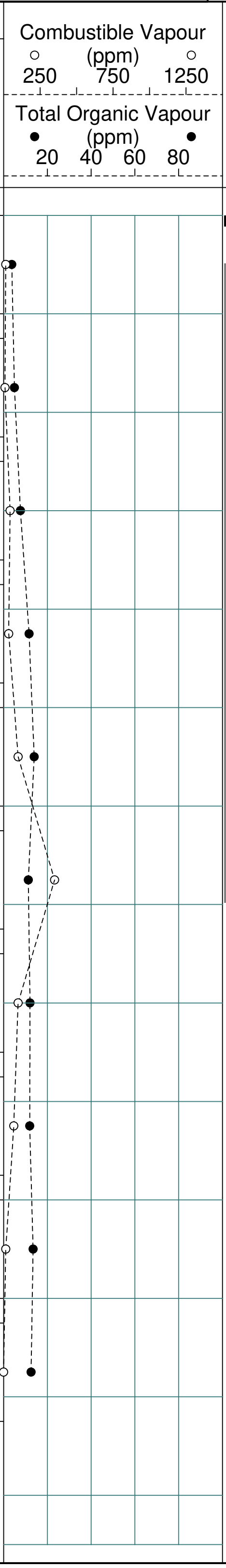
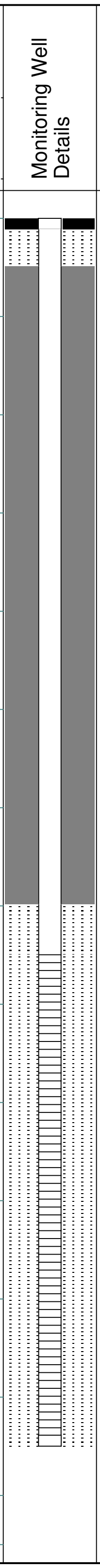
300-210 Colonnade Road
Ottawa, Ontario
K2E 7L5

SUBSURFACE PROFILE / SAMPLE DATA

Description	Elevation (m)	Type	Number	N or RQD	Recovery (%)	Remarks
Ground Surface	64.92					
ASPHALT	0.00					
FILL Fine grained silty sand and gravel, grey, damp		AS	1	NA	N/A	
Trace clay		AS	2	NA	N/A	Sample submitted for Metals and pH analyses.
SAND AND SILT Fine grained sand and silt, brown	63.40 1.52	SS	3	17	33	
SILTY SAND Fine to medium grained silty sand, brown	62.63 2.29	SS	4	5	75	
		SS	5	10	50	
		SS	6	10	54	Sample submitted for PHC F1-F4 and BTEXS analyses.
		SS	7	12	50	
SILTY SAND AND GRAVEL Fine to medium grained silty sand, pieces of rock	59.89 5.03					
Fine to coarse grained silty sand, some gravel, trace pieces of rock, brown/grey, moist		SS	8	17	42	
Becomes wet		SS	9	16	50	
Fine to medium grained silty sand, trace gravel, wet		SS	10	12	54	
END OF BOREHOLE	57.30 7.62					





Elevation: 64.92
Easting: 368840.183
Northing: 5029137.578

Casing Elevation: 64.74
Well Casing Size: 32 mm
Vapour Unit: Eagle Gastech/Mini Rae

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

PROJECT INFORMATION	
ENGINEERED PRODUCT MANAGER:	HAIDER NASRULLAH 647-850-9417 HAIDER.NASRULLAH@ADSPIPE.COM
ADS SALES REP:	BRAD DUNLOP 613-893-7336 BRAD.DUNLOP@ADSPIPE.COM
PROJECT NO:	S426399
ONTARIO SITE COORDINATOR:	RYAN RUBENSTEIN 519-710-3687 RYAN.RUBENSTEIN@ADSPIPE.COM

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Advanced Drainage Systems, Inc.

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OTTAWA, ON.

MC-3500 STORMTECH CHAMBER SPECIFICATIONS

1. CHAMBERS SHALL BE STORMTECH MC-3500.

2. CHAMBERS SHALL BE ARCH-SHAPED AND SHALL BE MANUFACTURED FROM VIRGIN, IMPACT-MODIFIED POLYPROPYLENE COPOLYMERS.

3. CHAMBERS SHALL BE CERTIFIED TO CSA B184, "POLYMERIC SUB-SURFACE STORMWATER MANAGEMENT STRUCTURES", AND MEET THE REQUIREMENTS OF ASTM F2418, "STANDARD SPECIFICATION FOR POLYPROPYLENE (PP) CORRUGATED WALL STORMWATER COLLECTION CHAMBERS" CHAMBER CLASSIFICATION 45x76 DESIGNATION SS.

4. CHAMBER ROWS SHALL PROVIDE CONTINUOUS, UNOBSTRUCTED INTERNAL SPACE WITH NO INTERNAL SUPPORTS THAT WOULD IMPEDE FLOW OR LIMIT ACCESS FOR INSPECTION.

5. THE STRUCTURAL DESIGN OF THE CHAMBERS, THE STRUCTURAL BACKFILL, AND THE INSTALLATION REQUIREMENTS SHALL ENSURE THAT THE LOAD FACTORS SPECIFIED IN THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, SECTION 12.12, ARE MET FOR: 1) LONG-DURATION DEAD LOADS AND 2) SHORT-DURATION LIVE LOADS, BASED ON THE CSA S6 CL-625 TRUCK AND THE AASHTO DESIGN TRUCK WITH CONSIDERATION FOR IMPACT AND MULTIPLE VEHICLE PRESENCES.

6. CHAMBERS SHALL BE DESIGNED, TESTED AND ALLOWABLE LOAD CONFIGURATIONS DETERMINED IN ACCORDANCE WITH ASTM F2787, "STANDARD PRACTICE FOR STRUCTURAL DESIGN OF THERMOPLASTIC CORRUGATED WALL STORMWATER COLLECTION CHAMBERS". LOAD CONFIGURATIONS SHALL INCLUDE: 1) INSTANTANEOUS (<1 MIN) AASHTO DESIGN TRUCK LIVE LOAD ON MINIMUM COVER 2) MAXIMUM PERMANENT (75-YR) COVER LOAD AND 3) ALLOWABLE COVER WITH PARKED (1-WEEK) AASHTO DESIGN TRUCK.

7. REQUIREMENTS FOR HANDLING AND INSTALLATION:
   - TO MAINTAIN THE WIDTH OF CHAMBERS DURING SHIPPING AND HANDLING, CHAMBERS SHALL HAVE INTEGRAL, INTERLOCKING STACKING LUGS.
   - TO ENSURE A SECURE JOINT DURING INSTALLATION AND BACKFILL, THE HEIGHT OF THE CHAMBER JOINT SHALL NOT BE LESS THAN 75 mm (3").
   - TO ENSURE THE INTEGRITY OF THE ARCH SHAPE DURING INSTALLATION, a) THE ARCH STIFFNESS CONSTANT SHALL BE GREATER THAN OR EQUAL TO 450 LBS/FT/%. THE ASC IS DEFINED IN SECTION 6.2.8 OF ASTM F2418. AND b) TO RESIST CHAMBER DEFORMATION DURING INSTALLATION AT ELEVATED TEMPERATURES (ABOVE 23° C / 73° F), CHAMBERS SHALL BE PRODUCED FROM REFLECTIVE GOLD OR YELLOW COLORS.

8. ONLY CHAMBERS THAT ARE APPROVED BY THE SITE DESIGN ENGINEER WILL BE ALLOWED. UPON REQUEST BY THE SITE DESIGN ENGINEER OR OWNER, THE CHAMBER MANUFACTURER SHALL SUBMIT A STRUCTURAL EVALUATION FOR APPROVAL BEFORE DELIVERING CHAMBERS TO THE PROJECT SITE AS FOLLOWS:
   - THE STRUCTURAL EVALUATION SHALL BE SEALED BY A REGISTERED PROFESSIONAL ENGINEER.
   - THE STRUCTURAL EVALUATION SHALL DEMONSTRATE THAT THE SAFETY FACTORS ARE GREATER THAN OR EQUAL TO 1.95 FOR DEAD LOAD AND 1.75 FOR LIVE LOAD, THE MINIMUM REQUIRED BY ASTM F2787 AND BY SECTIONS 3 AND 12.12 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS FOR THERMOPLASTIC PIPE.
   - THE TEST DERIVED CREEP MODULUS AS SPECIFIED IN ASTM F2418 SHALL BE USED FOR PERMANENT DEAD LOAD DESIGN EXCEPT THAT IT SHALL BE THE 75-YEAR MODULUS USED FOR DESIGN.

9. CHAMBERS AND END CAPS SHALL BE PRODUCED AT AN ISO 9001 CERTIFIED MANUFACTURING FACILITY.

10. MANIFOLD SIZE TO BE DETERMINED BY SITE DESIGN ENGINEER. SEE TECHNICAL NOTE 6.32 FOR MANIFOLD SIZING GUIDANCE. DUE TO THE ADAPTATION OF THIS CHAMBER SYSTEM TO SPECIFIC SITE AND DESIGN CONSTRAINTS, IT MAY BE NECESSARY TO CUT AND COUPLE ADDITIONAL PIPE TO STANDARD MANIFOLD COMPONENTS IN THE FIELD.

11. ADS DOES NOT DESIGN OR PROVIDE MEMBRANE LINER SYSTEMS. TO MINIMIZE THE LEAKAGE POTENTIAL OF LINER SYSTEMS, THE MEMBRANE LINER SYSTEM SHOULD BE DESIGNED BY A KNOWLEDGEABLE GEOTEXTILE PROFESSIONAL AND INSTALLED BY A QUALIFIED CONTRACTOR.

IMPORTANT - NOTES FOR THE BIDDING AND INSTALLATION OF MC-3500 CHAMBER SYSTEM

1. STORMTECH MC-3500 CHAMBERS SHALL NOT BE INSTALLED UNTIL THE MANUFACTURER'S REPRESENTATIVE HAS COMPLETED A PRE-CONSTRUCTION MEETING WITH THE INSTALLERS.

2. STORMTECH MC-3500 CHAMBERS SHALL BE INSTALLED IN ACCORDANCE WITH THE "STORMTECH MC-3500/MC-4500 CONSTRUCTION GUIDE".

3. CHAMBERS ARE NOT TO BE BACKFILLED WITH A DOZER OR AN EXCAVATOR SITUATED OVER THE CHAMBERS. STORMTECH RECOMMENDS 3 BACKFILL METHODS:
   - STONESHOOTER LOCATED OFF THE CHAMBER BED.
   - BACKFILL AS ROWS ARE BUILT USING AN EXCAVATOR ON THE FOUNDATION STONE OR SUBGRADE.
   - BACKFILL FROM OUTSIDE THE EXCAVATION USING A LONG BOOM HOE OR EXCAVATOR.

4. THE FOUNDATION STONE SHALL BE LEVELED AND COMPACTED PRIOR TO PLACING CHAMBERS.

5. JOINTS BETWEEN CHAMBERS SHALL BE PROPERLY SEATED PRIOR TO PLACING STONE.

6. MAINTAIN MINIMUM - 230 mm (9") SPACING BETWEEN THE CHAMBER ROWS.

7. INLET AND OUTLET MANIFOLDS MUST BE INSERTED A MINIMUM OF 300 mm (12") INTO CHAMBER END CAPS.

8. EMBEDMENT STONE SURROUNDING CHAMBERS MUST BE A CLEAN, CRUSHED, ANGULAR STONE OR RECYCLED CONCRETE; AASHTO M43 #3, 357, 4, 467, 5, 56, OR 57.

9. STONE MUST BE PLACED ON THE TOP CENTER OF THE CHAMBER TO ANCHOR THE CHAMBERS IN PLACE AND PRESERVE ROW SPACING.

10. THE CONTRACTOR MUST REPORT ANY DISCREPANCIES WITH CHAMBER FOUNDATION MATERIALS BEARING CAPACITIES TO THE SITE DESIGN ENGINEER.

11. ADS RECOMMENDS THE USE OF "FLEXSTORM CATCH IT" INSERTS DURING CONSTRUCTION FOR ALL INLETS TO PROTECT THE SUBSURFACE STORMWATER MANAGEMENT SYSTEM FROM CONSTRUCTION SITE RUNOFF.

NOTES FOR CONSTRUCTION EQUIPMENT

1. STORMTECH MC-3500 CHAMBERS SHALL BE INSTALLED IN ACCORDANCE WITH THE "STORMTECH MC-3500/MC-4500 CONSTRUCTION GUIDE".

2. THE USE OF EQUIPMENT OVER MC-3500 CHAMBERS IS LIMITED:
   - NO EQUIPMENT IS ALLOWED ON BARE CHAMBERS.
   - NO RUBBER TIRED LOADER, DUMP TRUCK, OR EXCAVATORS ARE ALLOWED UNTIL PROPER FILL DEPTHS ARE REACHED IN ACCORDANCE WITH THE "STORMTECH MC-3500/MC-4500 CONSTRUCTION GUIDE".
   - WEIGHT LIMITS FOR CONSTRUCTION EQUIPMENT CAN BE FOUND IN THE "STORMTECH MC-3500/MC-4500 CONSTRUCTION GUIDE".

3. FULL 900 mm (36") OF STABILIZED COVER MATERIALS OVER THE CHAMBERS IS REQUIRED FOR DUMP TRUCK TRAVEL OR DUMPING.

USE OF A DOZER TO PUSH EMBEDMENT STONE BETWEEN THE ROWS OF CHAMBERS MAY CAUSE DAMAGE TO CHAMBERS AND IS NOT AN ACCEPTABLE BACKFILL METHOD. ANY CHAMBERS DAMAGED BY USING THE "DUMP AND PUSH" METHOD ARE NOT COVERED UNDER THE STORMTECH STANDARD WARRANTY.

CONTACT STORMTECH AT 1-800-821-6710 WITH ANY QUESTIONS ON INSTALLATION REQUIREMENTS OR WEIGHT LIMITS FOR CONSTRUCTION EQUIPMENT.

IMPORTANT - THIS PROJECT REQUIRES COMPACTION OF EMBEDMENT STONE AND REQUIREMENTS FOR STONE HARDNESS AND SHAPE WHICH ARE NOT SPECIFIED IN OTHER STORMTECH DOCUMENTS. CONTRACTORS MUST FOLLOW THE SPECIAL PROVISIONS IN THIS PLAN SET.

PROPOSED LAYOUT

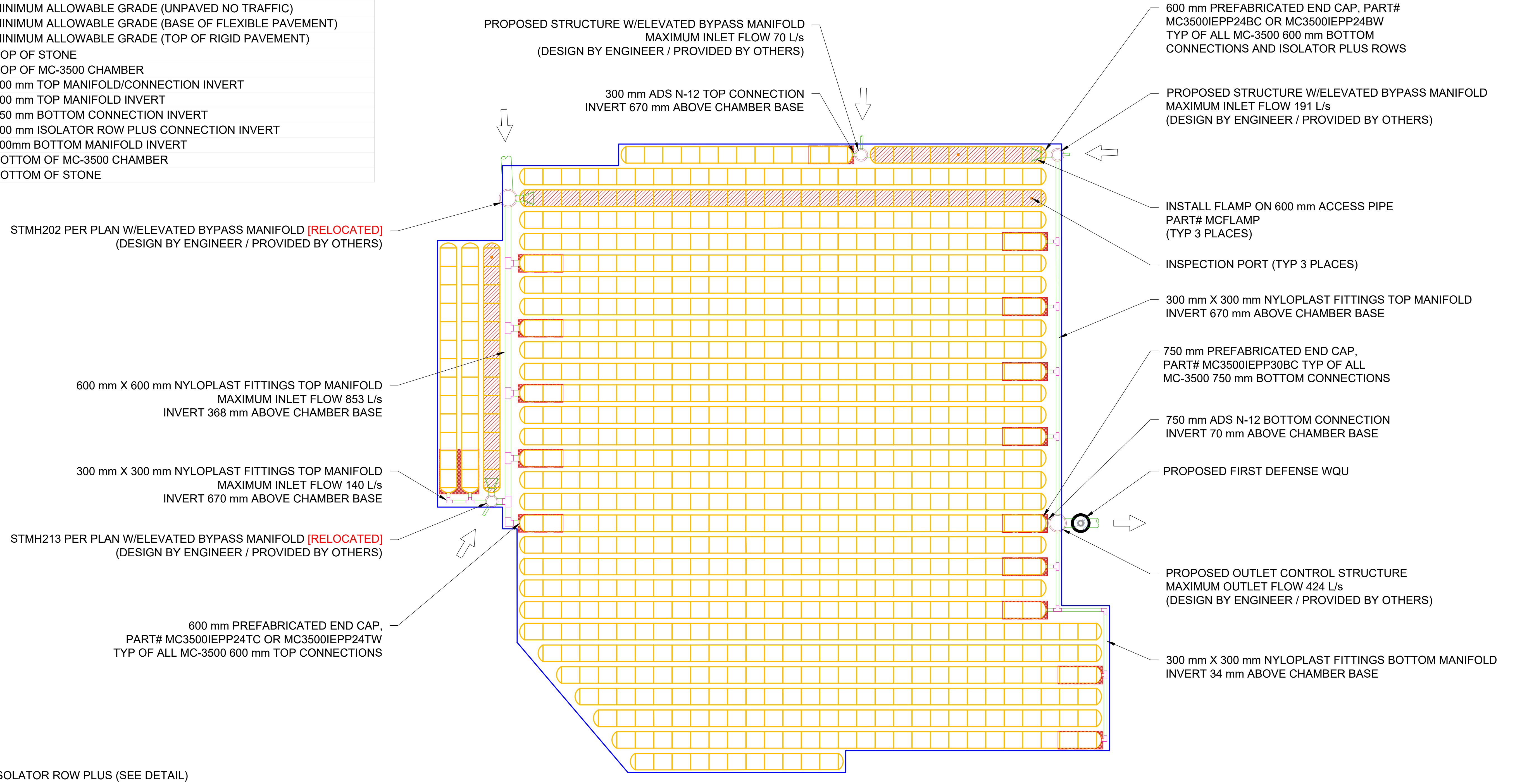
830	STORMTECH MC-3500 CHAMBERS
66	STORMTECH MC-3500 END CAPS
305	STONE ABOVE (mm)
900	STONE BELOW (mm)
40	% STONE VOID
4,302.5	**INSTALLED SYSTEM VOLUME (m³) ABOVE ELEVATION 63.060 (PERIMETER STONE INCLUDED)**
5,013.5	SYSTEM AREA (m²)
299.6	SYSTEM PERIMETER (m)

PROPOSED ELEVATIONS

66.28±	MAXIMUM GRADE PER ENGINEER'S PLANS
64.779	MINIMUM ALLOWABLE GRADE (UNPAVED WITH TRAFFIC)
64.626	MINIMUM ALLOWABLE GRADE (UNPAVED NO TRAFFIC)
64.626	MINIMUM ALLOWABLE GRADE (BASE OF FLEXIBLE PAVEMENT)
64.626	MINIMUM ALLOWABLE GRADE (TOP OF RIGID PAVEMENT)
64.474	TOP OF STONE
64.169	TOP OF MC-3500 CHAMBER
63.696	300 mm TOP MANIFOLD/CONNECTION INVERT
63.394	600 mm TOP MANIFOLD INVERT
63.096	750 mm BOTTOM CONNECTION INVERT
63.078	600 mm ISOLATOR ROW PLUS CONNECTION INVERT
63.060	300mm BOTTOM MANIFOLD INVERT
63.026	BOTTOM OF MC-3500 CHAMBER
62.126	BOTTOM OF STONE

TIER 2 DEEP COVER SPECIAL PROVISIONS

1. INSTALLATION REQUIREMENTS SHALL BE AS SPECIFIED IN THE STORMTECH DESIGN MANUALS AND CONSTRUCTION GUIDES EXCEPT AS MODIFIED IN THESE SPECIAL PROVISIONS.
2. ATTENTION IS CALLED TO "TABLE 1 - ACCEPTABLE FILL MATERIALS" IN THE STORMTECH CONSTRUCTION GUIDE AND ALL OTHER APPEARANCES OF THE "ACCEPTABLE FILL MATERIALS" TABLE. FOR AREAS OF THE SYSTEM WITH COVER ABOVE 11 FEET (3.4 m) FOR THE MC-4500/MC-7200 AND ABOVE 12 FEET (3.7 m) FOR THE MC-3500, EMBEDMENT STONE SHALL BE COMPACTED WITH 1-3 PASSES OF A WALK BEHIND VIBRATORY PLATE COMPACTOR OR JUMPING JACK IN NO GREATER THAN 12" (300 mm) LIFTS.
3. STONE SHALL BE CLEAN, CRUSHED, AND ANGULAR AND SHALL CONFORM TO THE SPECIFICATIONS DESIGNATED IN THE ACCEPTABLE FILL MATERIALS TABLE.
4. STONE SHALL BE HARD AND DURABLE. IT IS THE ENGINEER'S OR CONTRACTOR'S RESPONSIBILITY TO SELECT HARD AND DURABLE STONE. STORMTECH CONSIDERS AN LA ABRASION VALUE OF LESS THAN OR EQUAL TO 30 TO BE HARD STONE.
5. FOUNDATION STONE SHALL BE MECHANICALLY COMPACTED WITH A VIBRATORY ROLLER OR VIBRATORY PLATE IN 6" (152 mm) LIFTS.
6. EMBEDMENT STONE MUST BE DUMPED IN PLACE BY A STONE SHOOTER OR CONVEYOR OR EXCAVATOR.
7. INSPECTION DURING THE INSTALLATION BY THE ENGINEER, OWNER OR OTHER REPRESENTATIVE IS RECOMMENDED. THE INSPECTION SHALL INCLUDE OBSERVATIONS OF THE CHAMBER SYMMETRY DURING BACKFILLING TO ENSURE THE CONTRACTOR'S METHODS ARE NOT CAUSING UNACCEPTABLE DISTORTION OF THE CHAMBERS.
8. AN ADS FIELD TECHNICIAN WILL CONDUCT A PRE-CONSTRUCTION MEETING TO TRAIN REPRESENTATIVES INSTALLING THE CHAMBERS AND THOSE WHO MAY BE PERFORMING INSTALLATION INSPECTIONS.



▨	ISOLATOR ROW PLUS (SEE DETAIL)
▩	PLACE MINIMUM 5.33 m OF ADSPLUS125 WOVEN GEOTEXTILE OVER BEDDING STONE AND UNDERNEATH CHAMBER FEET FOR SCOUR PROTECTION AT ALL CHAMBER INLET ROWS
—	THERMOPLASTIC LINER (SEE TECHNICAL NOTE 6.50 / PROVIDED BY OTHERS / DESIGN BY OTHERS)

- **NOT FOR CONSTRUCTION**: THIS LAYOUT IS FOR DIMENSIONAL PURPOSES ONLY TO PROVE CONCEPT & THE REQUIRED STORAGE VOLUME CAN BE ACHIEVED ON SITE.

LANSDOWNE 2.0
OTTAWA, ON.

DATE: 8/28/24
DRAWN: RCT
PROJECT #: S426399
CHECKED: RCT

DATE	DRWN	CHKD	DESCRIPTION
12/13/24	JR	JPR	REVISED TO TIER II
8/29/24	RCT	RCT	NEW VOLUME 4300M3/ADDED CHAMBERS

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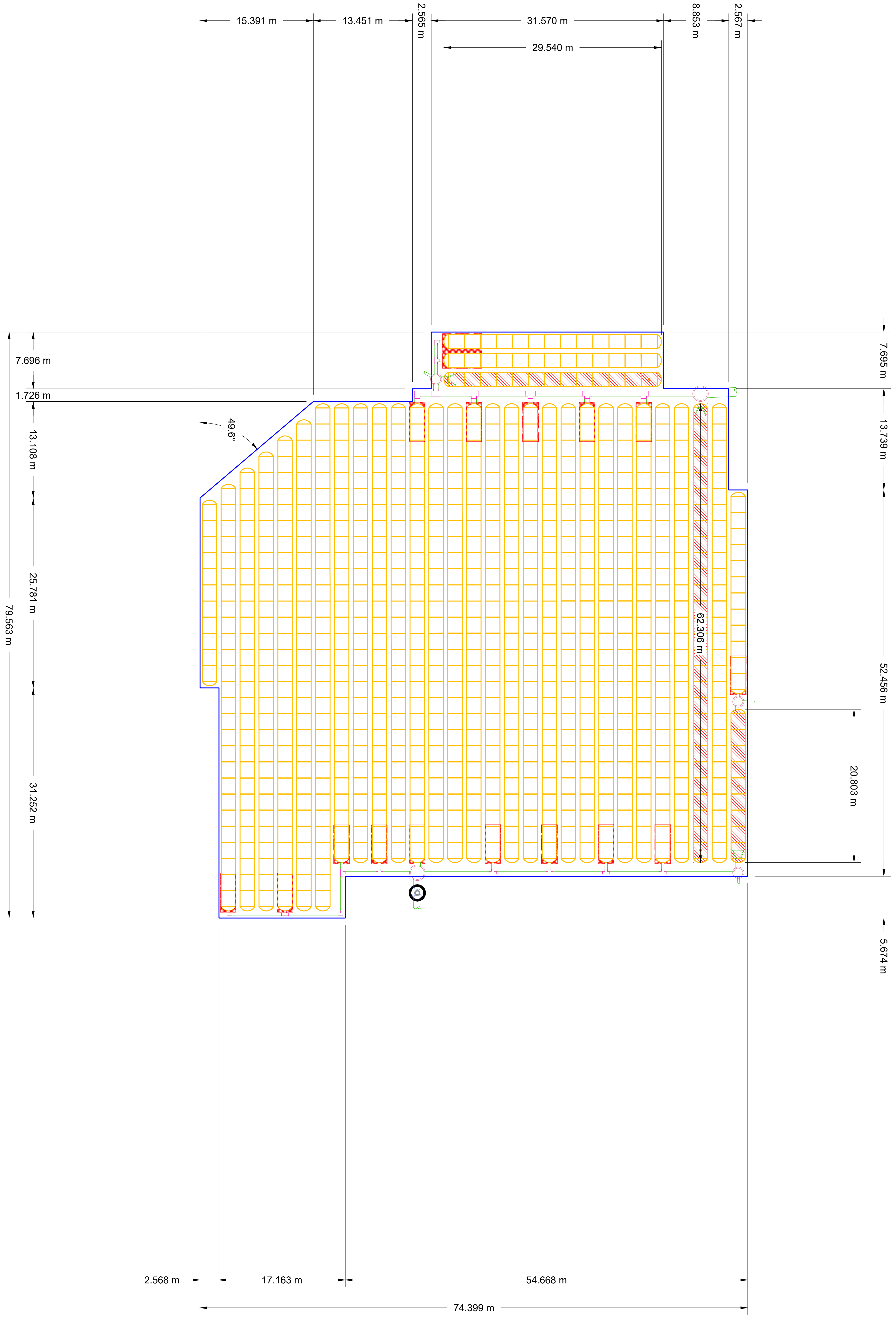
ADS™
4640 TRUEMAN BLVD
HILLIARD, OH 43026

SCALE = 1 : 500

THIS DRAWING HAS BEEN PREPARED BASED ON INFORMATION PROVIDED TO ADS UNDER THE DIRECTION OF THE SITE DESIGN ENGINEER OR OTHER PROJECT REPRESENTATIVE. THE SITE DESIGN ENGINEER SHALL REVIEW THIS DRAWING PRIOR TO CONSTRUCTION. IT IS THE ULTIMATE RESPONSIBILITY OF THE SITE DESIGN ENGINEER TO ENSURE THAT THE PRODUCT(S) DEPICTED AND ALL ASSOCIATED DETAILS MEET ALL APPLICABLE LAWS, REGULATIONS, AND PROJECT REQUIREMENTS.

SHEET
2 OF 6



SHEET
3 OF 6

4640 TRUEMAN BLVD
HILLIARD, OH 43026

SCALE = 1 : 500

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DATE	DRWN	CHKD	DESCRIPTION
12/13/24	JR	JPR	REVISED TO TIER II
8/29/24	RCT	RCT	NEW VOLUME 4300M3/ADDED CHAMBERS

LANSDOWNE 2.0

OTTAWA, ON.

DATE:	8/28/24	DRAWN:	RCT
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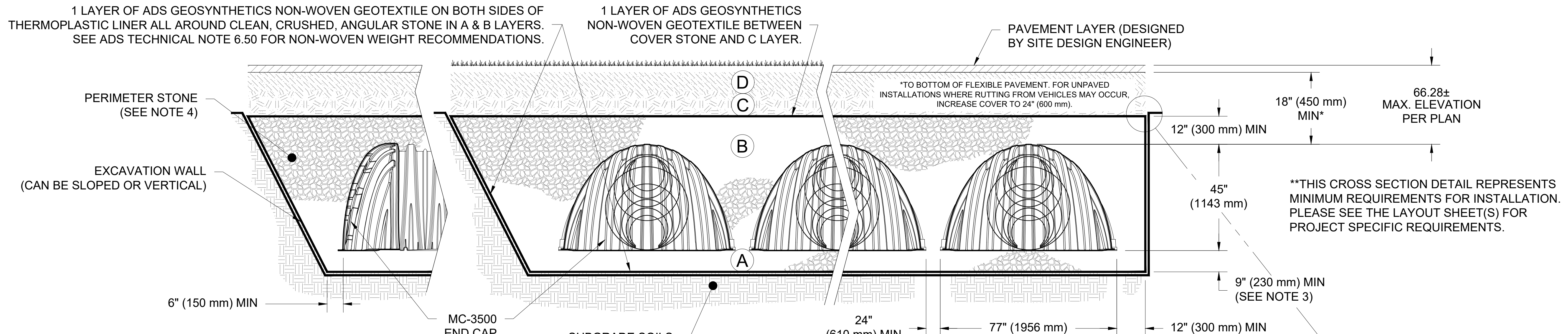
ACCEPTABLE FILL MATERIALS: STORMTECH MC-3500 CHAMBER SYSTEMS

MATERIAL LOCATION		DESCRIPTION	AASHTO MATERIAL CLASSIFICATIONS	COMPACTION / DENSITY REQUIREMENT
D	**FINAL FILL:** FILL MATERIAL FOR LAYER 'D' STARTS FROM THE TOP OF THE 'C' LAYER TO THE BOTTOM OF FLEXIBLE PAVEMENT OR UNPAVED FINISHED GRADE ABOVE. NOTE THAT PAVEMENT SUBBASE MAY BE PART OF THE 'D' LAYER	ANY SOIL/ROCK MATERIALS, NATIVE SOILS, OR PER ENGINEER'S PLANS. CHECK PLANS FOR PAVEMENT SUBGRADE REQUIREMENTS.	N/A	PREPARE PER SITE DESIGN ENGINEER'S PLANS. PAVED INSTALLATIONS MAY HAVE STRINGENT MATERIAL AND PREPARATION REQUIREMENTS.
C	**INITIAL FILL:** FILL MATERIAL FOR LAYER 'C' STARTS FROM THE TOP OF THE EMBEDMENT STONE ('B' LAYER) TO 18" (450 mm) ABOVE THE TOP OF THE CHAMBER. NOTE THAT PAVEMENT SUBBASE MAY BE A PART OF THE 'C' LAYER.	GRANULAR WELL-GRADED SOIL/AGGREGATE MIXTURES, <35% FINES OR PROCESSED AGGREGATE. MOST PAVEMENT SUBBASE MATERIALS CAN BE USED IN LIEU OF THIS LAYER.	AASHTO M145[1] A-1, A-2-4, A-3 OR AASHTO M43[1] 3, 357, 4, 467, 5, 56, 57, 6, 67, 68, 7, 78, 8, 89, 9, 10	BEGIN COMPACTIONS AFTER 18" (450 mm) OF MATERIAL OVER THE CHAMBERS IS REACHED. COMPACT ADDITIONAL LAYERS IN 12" (300 mm) MAX LIFTS TO A MIN. 95% PROCTOR DENSITY FOR WELL GRADED MATERIAL AND 95% RELATIVE DENSITY FOR PROCESSED AGGREGATE MATERIALS.
B	**EMBEDMENT STONE:** FILL SURROUNDING THE CHAMBERS FROM THE FOUNDATION STONE ('A' LAYER) TO THE 'C' LAYER ABOVE.	CLEAN, CRUSHED, ANGULAR STONE OR RECYCLED CONCRETE[5]	AASHTO M43[1] 3, 357, 4, 467, 5, 56, 57	COMPACTION REQUIRED. SEE SPECIAL REQUIREMENTS ON LAYOUT PAGE.
A	**FOUNDATION STONE:** FILL BELOW CHAMBERS FROM THE SUBGRADE UP TO THE FOOT (BOTTOM) OF THE CHAMBER.	CLEAN, CRUSHED, ANGULAR STONE OR RECYCLED CONCRETE[5]	AASHTO M43[1] 3, 357, 4, 467, 5, 56, 57	PLATE COMPACT OR ROLL TO ACHIEVE A FLAT SURFACE.[2,3]

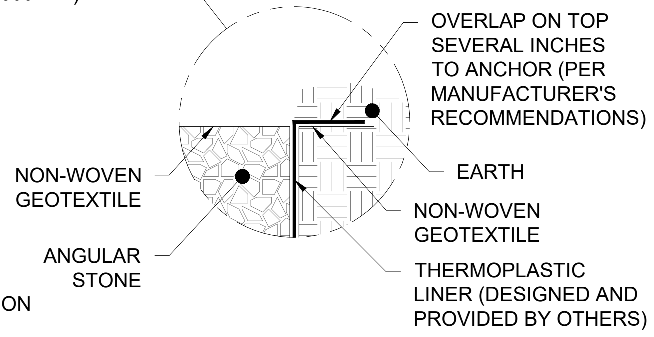
PLEASE NOTE:

1. THE LISTED AASHTO DESIGNATIONS ARE FOR GRADATIONS ONLY. THE STONE MUST ALSO BE CLEAN, CRUSHED, ANGULAR. FOR EXAMPLE, A SPECIFICATION FOR #4 STONE WOULD STATE: "CLEAN, CRUSHED, ANGULAR NO. 4 (AASHTO M43) STONE".
2. STORMTECH COMPACTION REQUIREMENTS ARE MET FOR 'A' LOCATION MATERIALS WHEN PLACED AND COMPACTED IN 9" (230 mm) (MAX) LIFTS USING TWO FULL COVERAGES WITH A VIBRATORY COMPACTOR.
3. WHERE INFILTRATION SURFACES MAY BE COMPROMISED BY COMPACTION, FOR STANDARD DESIGN LOAD CONDITIONS, A FLAT SURFACE MAY BE ACHIEVED BY RAKING OR DRAGGING WITHOUT COMPACTION EQUIPMENT. FOR SPECIAL LOAD DESIGNS, CONTACT STORMTECH FOR COMPACTION REQUIREMENTS.
4. ONCE LAYER 'C' IS PLACED, ANY SOIL/MATERIAL CAN BE PLACED IN LAYER 'D' UP TO THE FINISHED GRADE. MOST PAVEMENT SUBBASE SOILS CAN BE USED TO REPLACE THE MATERIAL REQUIREMENTS OF LAYER 'C' OR 'D' AT THE SITE DESIGN ENGINEER'S DISCRETION.
5. WHERE RECYCLED CONCRETE AGGREGATE IS USED IN LAYERS 'A' OR 'B' THE MATERIAL SHOULD ALSO MEET THE ACCEPTABILITY CRITERIA OUTLINED IN TECHNICAL NOTE 6.20 "RECYCLED CONCRETE STRUCTURAL BACKFILL".



**THIS CROSS SECTION DETAIL REPRESENTS MINIMUM REQUIREMENTS FOR INSTALLATION. PLEASE SEE THE LAYOUT SHEET(S) FOR PROJECT SPECIFIC REQUIREMENTS.

LANSDOWNE 2.0 SPECIFIC CROSS SECTION



THERMOPLASTIC LINER DETAIL

NOTES:

1. CHAMBERS SHALL MEET THE REQUIREMENTS OF ASTM F2418, "STANDARD SPECIFICATION FOR POLYPROPYLENE (PP) CORRUGATED WALL STORMWATER COLLECTION CHAMBERS" CHAMBER CLASSIFICATION 45x76 DESIGNATION SS.
2. MC-3500 CHAMBERS SHALL BE DESIGNED IN ACCORDANCE WITH ASTM F2787 "STANDARD PRACTICE FOR STRUCTURAL DESIGN OF THERMOPLASTIC CORRUGATED WALL STORMWATER COLLECTION CHAMBERS".
3. THE SITE DESIGN ENGINEER IS RESPONSIBLE FOR ASSESSING THE BEARING RESISTANCE (ALLOWABLE BEARING CAPACITY) OF THE SUBGRADE SOILS AND THE DEPTH OF FOUNDATION STONE WITH CONSIDERATION FOR THE RANGE OF EXPECTED SOIL MOISTURE CONDITIONS. REFERENCE STORMTECH DESIGN MANUAL FOR BEARING CAPACITY GUIDANCE.
4. PERIMETER STONE MUST BE EXTENDED HORIZONTALLY TO THE EXCAVATION WALL FOR BOTH VERTICAL AND SLOPED EXCAVATION WALLS.
5. REQUIREMENTS FOR HANDLING AND INSTALLATION:
    - TO MAINTAIN THE WIDTH OF CHAMBERS DURING SHIPPING AND HANDLING, CHAMBERS SHALL HAVE INTEGRAL, INTERLOCKING STACKING LUGS.
    - TO ENSURE A SECURE JOINT DURING INSTALLATION AND BACKFILL, THE HEIGHT OF THE CHAMBER JOINT SHALL NOT BE LESS THAN 3”.
    - TO ENSURE THE INTEGRITY OF THE ARCH SHAPE DURING INSTALLATION, a) THE ARCH STIFFNESS CONSTANT AS DEFINED IN SECTION 6.2.8 OF ASTM F2418 SHALL BE GREATER THAN OR EQUAL TO 500 LBS/FT/%. AND b) TO RESIST CHAMBER DEFORMATION DURING INSTALLATION AT ELEVATED TEMPERATURES (ABOVE 73° F / 23° C), CHAMBERS SHALL BE PRODUCED FROM REFLECTIVE GOLD OR YELLOW COLORS.

LANSDOWNE 2.0
OTTAWA, ON.

DATE:	8/28/24	DRAWN:	RCT
PROJECT #:	S426399	CHECKED:	RCT

DATE	DRWN	CHKD	DESCRIPTION
12/13/24	JR	JPR	REVISED TO TIER II
8/29/24	RCT	RCT	NEW VOLUME 4300M3/ADDED CHAMBERS

4640 TRUEMAN BLVD
HILLIARD, OH 43026

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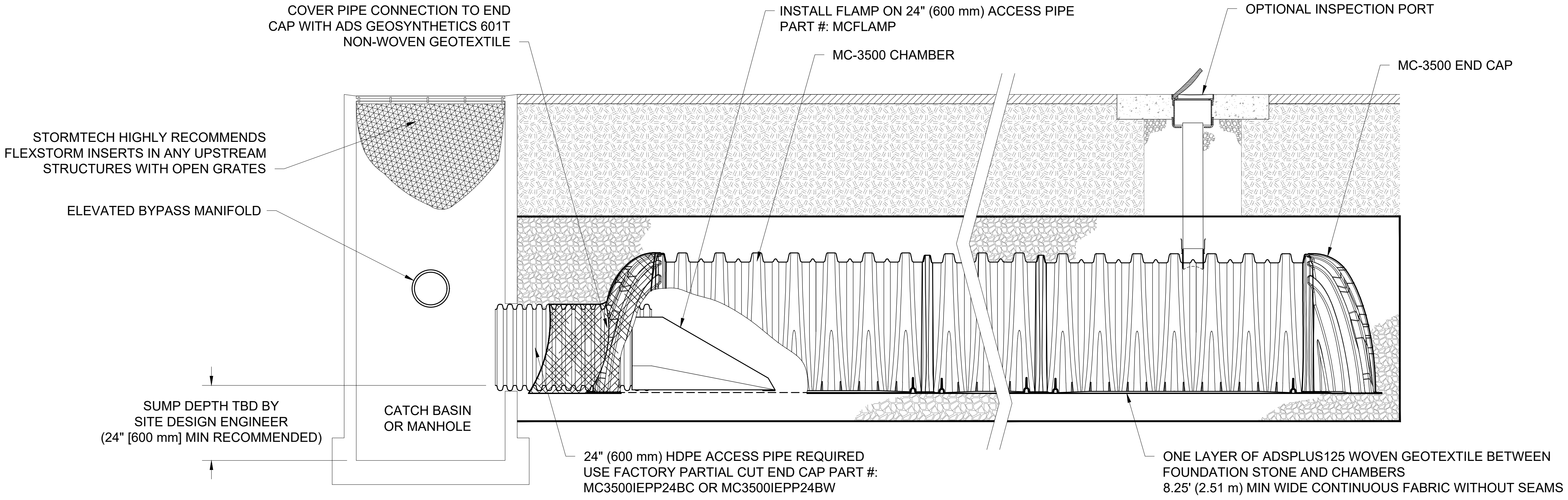
SHEET 4 OF 6



MC-3500 ISOLATOR ROW PLUS DETAIL
NTS

INSPECTION & MAINTENANCE

STEP 1) INSPECT ISOLATOR ROW PLUS FOR SEDIMENT
- A. INSPECTION PORTS (IF PRESENT)
  - A.1. REMOVE/OPEN LID ON NYLOPLAST INLINE DRAIN
  - A.2. REMOVE AND CLEAN FLEXSTORM FILTER IF INSTALLED
  - A.3. USING A FLASHLIGHT AND STADIA ROD, MEASURE DEPTH OF SEDIMENT AND RECORD ON MAINTENANCE LOG
  - A.4. LOWER A CAMERA INTO ISOLATOR ROW PLUS FOR VISUAL INSPECTION OF SEDIMENT LEVELS (OPTIONAL)
  - A.5. IF SEDIMENT IS AT, OR ABOVE, 3" (80 mm) PROCEED TO STEP 2. IF NOT, PROCEED TO STEP 3.
- B. ALL ISOLATOR PLUS ROWS
  - B.1. REMOVE COVER FROM STRUCTURE AT UPSTREAM END OF ISOLATOR ROW PLUS
  - B.2. USING A FLASHLIGHT, INSPECT DOWN THE ISOLATOR ROW PLUS THROUGH OUTLET PIPE
    - i) MIRRORS ON POLES OR CAMERAS MAY BE USED TO AVOID A CONFINED SPACE ENTRY
    - ii) FOLLOW OSHA REGULATIONS FOR CONFINED SPACE ENTRY IF ENTERING MANHOLE
  - B.3. IF SEDIMENT IS AT, OR ABOVE, 3" (80 mm) PROCEED TO STEP 2. IF NOT, PROCEED TO STEP 3.

STEP 2) CLEAN OUT ISOLATOR ROW PLUS USING THE JETVAC PROCESS
- A. A FIXED CULVERT CLEANING NOZZLE WITH REAR FACING SPREAD OF 45" (1.1 m) OR MORE IS PREFERRED
- B. APPLY MULTIPLE PASSES OF JETVAC UNTIL BACKFLUSH WATER IS CLEAN
- C. VACUUM STRUCTURE SUMP AS REQUIRED

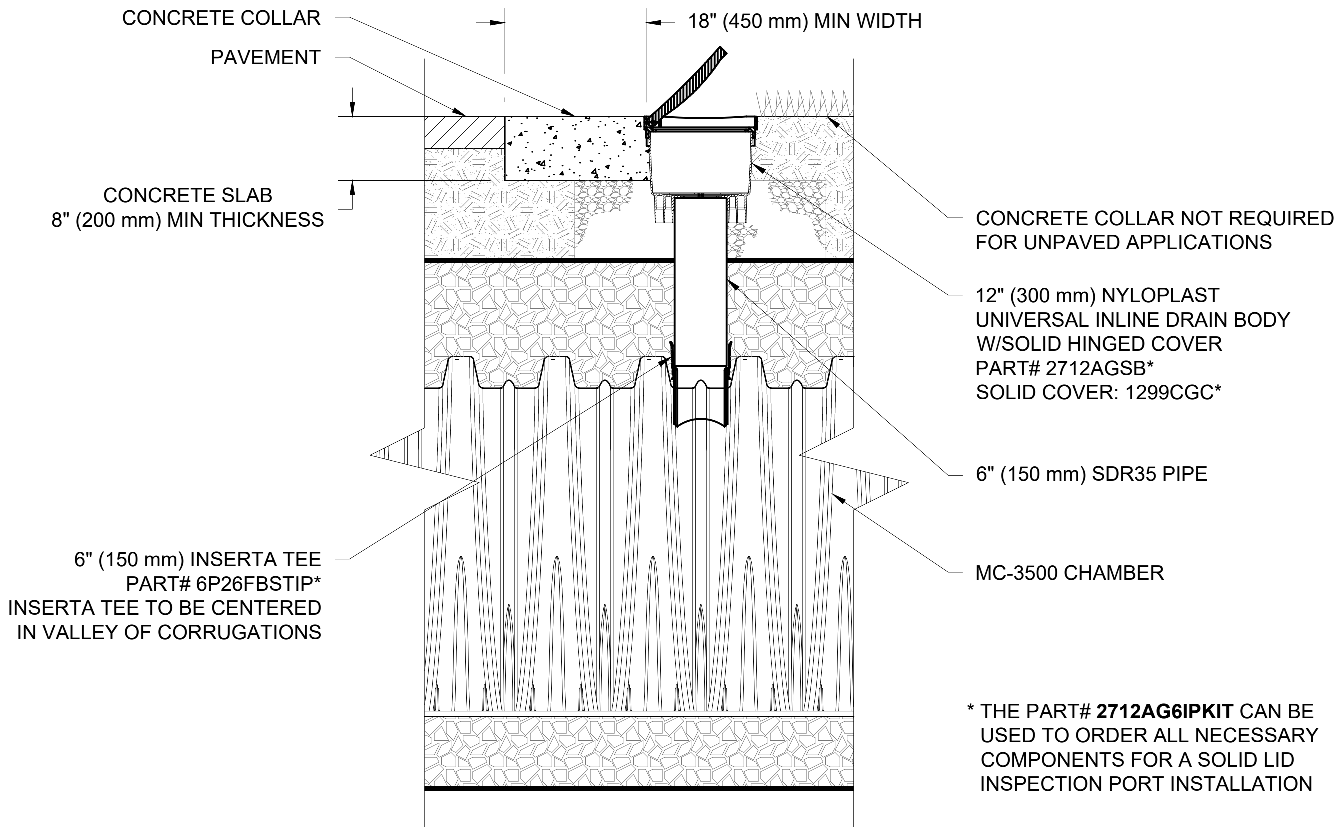
STEP 3) REPLACE ALL COVERS, GRATES, FILTERS, AND LIDS; RECORD OBSERVATIONS AND ACTIONS.

STEP 4) INSPECT AND CLEAN BASINS AND MANHOLES UPSTREAM OF THE STORMTECH SYSTEM.

NOTES

1. INSPECT EVERY 6 MONTHS DURING THE FIRST YEAR OF OPERATION. ADJUST THE INSPECTION INTERVAL BASED ON PREVIOUS OBSERVATIONS OF SEDIMENT ACCUMULATION AND HIGH WATER ELEVATIONS.

2. CONDUCT JETTING AND VACTORING ANNUALLY OR WHEN INSPECTION SHOWS THAT MAINTENANCE IS NECESSARY.



* THE PART# **2712AG6IPKIT** CAN BE USED TO ORDER ALL NECESSARY COMPONENTS FOR A SOLID LID INSPECTION PORT INSTALLATION

MC-3500 6" (150 mm) INSPECTION PORT DETAIL
NTS

LANSDOWNE 2.0

OTTAWA, ON.

DATE: 8/28/24
DRAWN: RCT
PROJECT #: S426399
CHECKED: RCT

DATE	DRWN	CHKD	DESCRIPTION
12/13/24	JR	JPR	REVISED TO TIER II
8/29/24	RCT	RCT	NEW VOLUME 4300M3/ADDED CHAMBERS

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4640 TRUEMAN BLVD
HILLIARD, OH 43026

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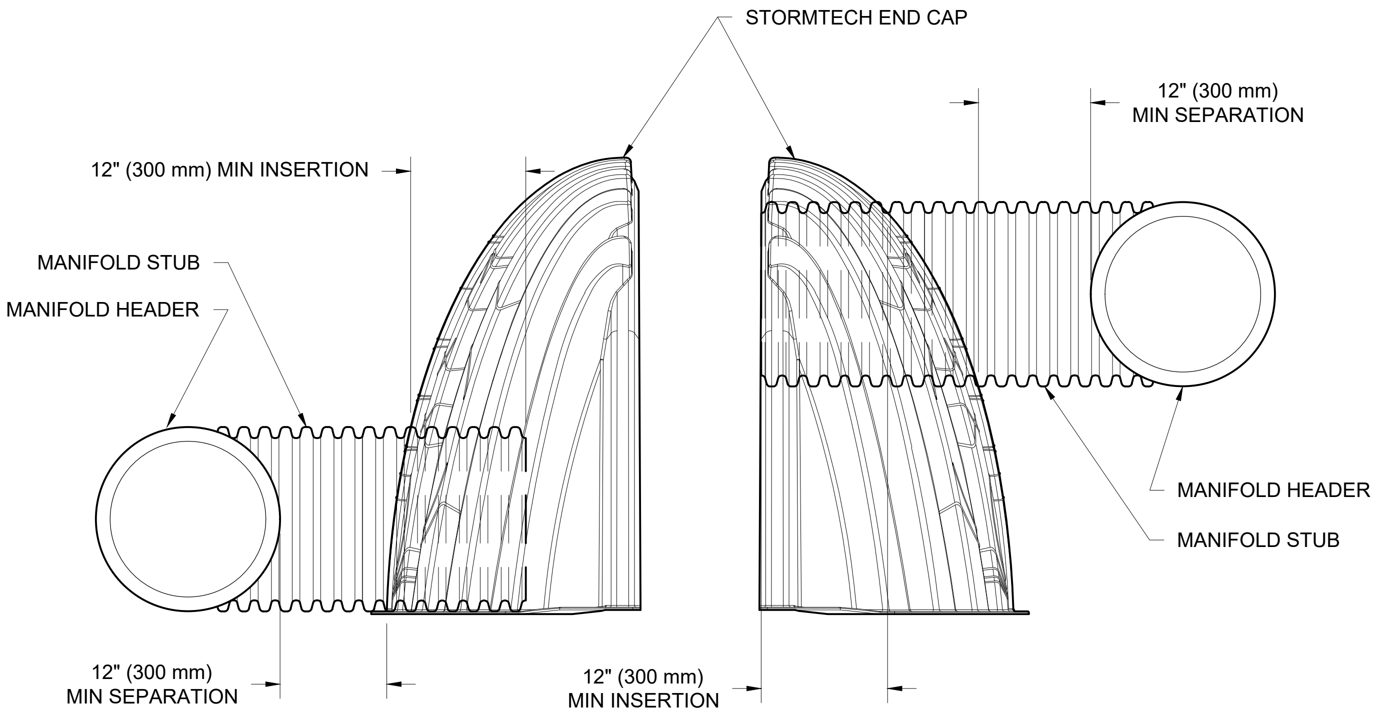
SHEET 5 OF 6

MC-SERIES END CAP INSERTION DETAIL

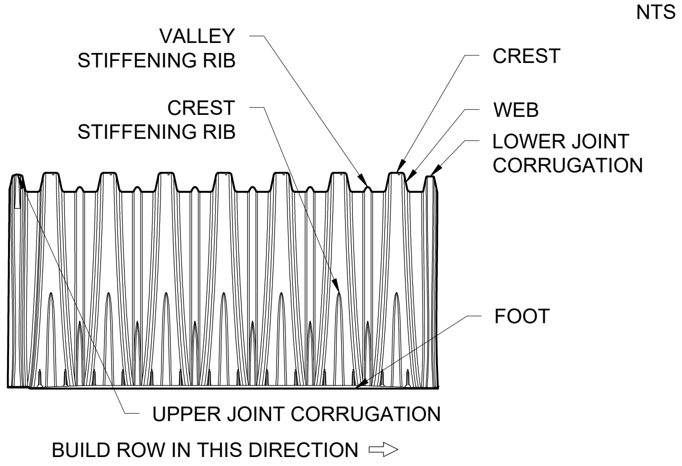
NTS

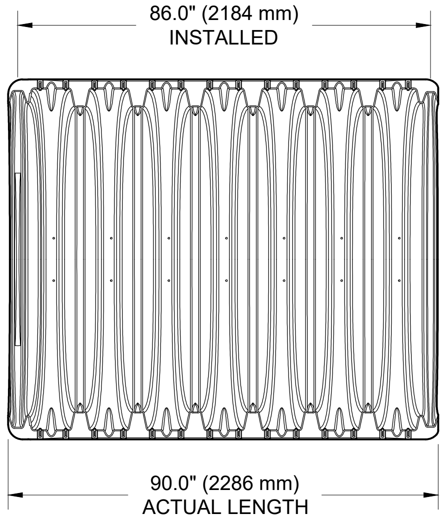


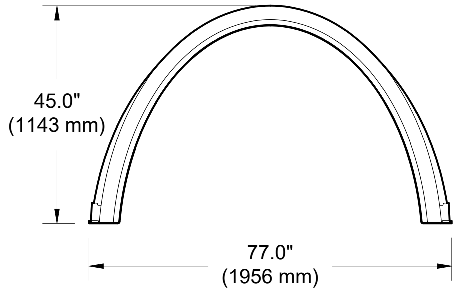
NOTE: MANIFOLD STUB MUST BE LAID HORIZONTAL
FOR A PROPER FIT IN END CAP OPENING.

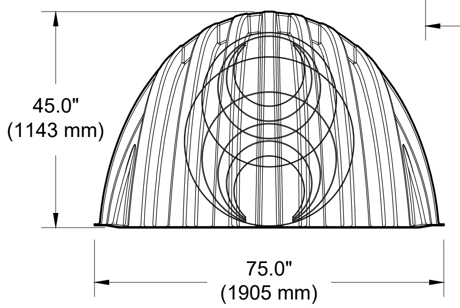
MC-3500 TECHNICAL SPECIFICATION

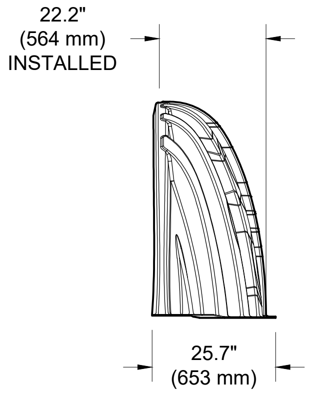
NTS











NOMINAL CHAMBER SPECIFICATIONS

SIZE (W X H X INSTALLED LENGTH)	77.0" X 45.0" X 86.0"	(1956 mm X 1143 mm X 2184 mm)
CHAMBER STORAGE	109.9 CUBIC FEET	(3.11 m³)
MINIMUM INSTALLED STORAGE*	175.0 CUBIC FEET	(4.96 m³)
WEIGHT	134 lbs.	(60.8 kg)

NOMINAL END CAP SPECIFICATIONS

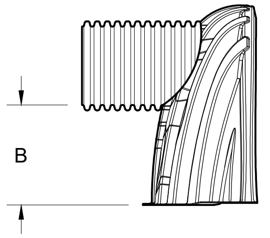
SIZE (W X H X INSTALLED LENGTH)	75.0" X 45.0" X 22.2"	(1905 mm X 1143 mm X 564 mm)
END CAP STORAGE	14.9 CUBIC FEET	(0.42 m³)
MINIMUM INSTALLED STORAGE*	45.1 CUBIC FEET	(1.28 m³)
WEIGHT	49 lbs.	(22.2 kg)

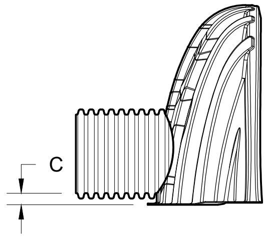
*ASSUMES 12" (305 mm) STONE ABOVE, 9" (229 mm) STONE FOUNDATION, 6" (152 mm) STONE BETWEEN CHAMBERS, 6" (152 mm) STONE PERIMETER IN FRONT OF END CAPS AND 40% STONE POROSITY.

PARTIAL CUT HOLES AT BOTTOM OF END CAP FOR PART NUMBERS ENDING WITH "B"
PARTIAL CUT HOLES AT TOP OF END CAP FOR PART NUMBERS ENDING WITH "T"
END CAPS WITH A PREFABRICATED WELDED STUB END WITH "W"
END CAPS WITH A WELDED CROWN PLATE END WITH "C"

PART #	STUB	B	C
MC3500IEPP06T	6" (150 mm)	33.21" (844 mm)	---
MC3500IEPP06B		---	0.66" (17 mm)
MC3500IEPP08T	8" (200 mm)	31.16" (791 mm)	---
MC3500IEPP08B		---	0.81" (21 mm)
MC3500IEPP10T	10" (250 mm)	29.04" (738 mm)	---
MC3500IEPP10B		---	0.93" (24 mm)
MC3500IEPP12T	12" (300 mm)	26.36" (670 mm)	---
MC3500IEPP12B		---	1.35" (34 mm)
MC3500IEPP15T	15" (375 mm)	23.39" (594 mm)	---
MC3500IEPP15B		---	1.50" (38 mm)
MC3500IEPP18TC	18" (450 mm)	20.03" (509 mm)	---
MC3500IEPP18TW			
MC3500IEPP18BC		---	1.77" (45 mm)
MC3500IEPP18BW			
MC3500IEPP24TC	24" (600 mm)	14.48" (368 mm)	---
MC3500IEPP24TW			
MC3500IEPP24BC		---	2.06" (52 mm)
MC3500IEPP24BW			
MC3500IEPP30BC	30" (750 mm)	---	2.75" (70 mm)

NOTE: ALL DIMENSIONS ARE NOMINAL





CUSTOM PARTIAL CUT INVERTS ARE AVAILABLE UPON REQUEST. INVENTORIED MANIFOLDS INCLUDE 12-24" (300-600 mm) SIZE ON SIZE AND 15-48" (375-1200 mm) ECCENTRIC MANIFOLDS. CUSTOM INVERT LOCATIONS ON THE MC-3500 END CAP CUT IN THE FIELD ARE NOT RECOMMENDED FOR PIPE SIZES GREATER THAN 10" (250 mm). THE INVERT LOCATION IN COLUMN 'B' ARE THE HIGHEST POSSIBLE FOR THE PIPE SIZE.

LANSDOWNE 2.0
OTTAWA, ON.

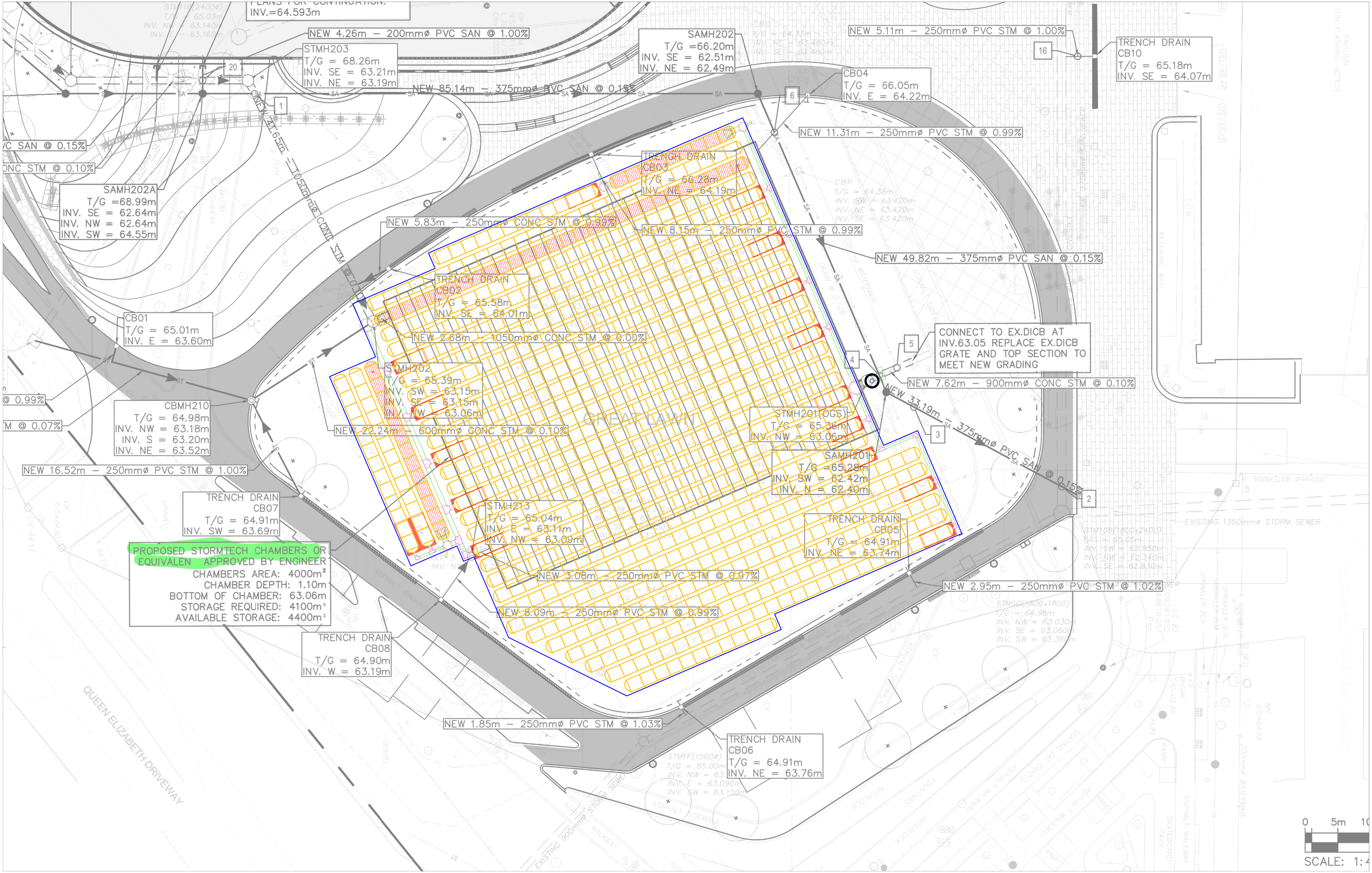
DATE: 8/28/24
DRAWN: RCT
PROJECT #: S426399
CHECKED: RCT

DATE	DRWN	CHKD	DESCRIPTION
12/13/24	JR	JPR	REVISED TO TIER II
8/29/24	RCT	RCT	NEW VOLUME 4300M3/ADDED CHAMBERS

StormTech®
Chamber System
1-800-821-6710 | WWW.STORMTECH.COM

ADS™
4640 TRUEMAN BLVD
HILLIARD, OH 43026

SHEET 6 OF 6


INV.=64.593m
NEW 4.26m – 200mmø PVC SAN @ 1.00%
STMH203
T/G = 68.26m
INV. SE = 63.21m
INV. NE = 63.19m
SAMH202
T/G =66.20m
INV. SE = 62.51m
INV. NE = 62.49m
NEW 5.11m – 250mmø PVC STM @ 1.00%
16
TRENCH DRAIN
CB10
T/G = 65.18m
INV. SE = 64.07m
20
1
NEW 85.14m – 375mmø PVC SAN @ 0.15%
6
CB04
T/G = 66.05m
INV. E = 64.22m
NEW 11.31m – 250mmø PVC STM @ 0.99%
NEW 41.65m – 1050mmø CONC STM @ 0.10%
C SAN @ 0.15%
ONC STM @ 0.10%
SAMH202A
T/G =68.99m
INV. SE = 62.64m
INV. NW = 62.64m
INV. SW = 64.55m
TRENCH DRAIN
CB03
T/G = 66.28m
INV. NE = 64.19m
NEW 5.83m – 250mmø CONC STM @ 0.99%
NEW 8.15m – 250mmø PVC STM @ 0.99%
NEW 49.82m – 375mmø PVC SAN @ 0.15%
TRENCH DRAIN
CB02
T/G = 65.58m
INV. SE = 64.01m
CB01
T/G = 65.01m
INV. E = 63.60m
NEW 2.68m – 1050mmø CONC STM @ 0.00%
5
CONNECT TO EX.DICB AT
INV.63.05 REPLACE EX.DICB
GRATE AND TOP SECTION TO
MEET NEW GRADING
4
NEW 7.62m – 900mmø CONC STM @ 0.10%
STMH202
T/G = 65.39m
INV. SW = 63.15m
INV. SE = 63.15m
INV. NW = 63.06m
@ 0.99%
M @ 0.07%
CBMH210
T/G = 64.98m
INV. NW = 63.18m
INV. S = 63.20m
INV. NE = 63.52m
NEW 22.24m – 600mmø CONC STM @ 0.10%
GREAT LAWN
STMH201(OGS)
T/G = 65.36m
INV. NW = 63.06m
NEW 33.19m – 375mmø PVC SAN @ 0.15%
3
SAMH201
T/G =65.28m
INV. SW = 62.42m
INV. N = 62.40m
2
NEW 16.52m – 250mmø PVC STM @ 1.00%
TRENCH DRAIN
CB07
T/G = 64.91m
INV. SW = 63.69m
STMH213
T/G = 65.04m
INV. E = 63.11m
INV. NW = 63.09m
TRENCH DRAIN
CB05
T/G = 64.91m
INV. NE = 63.74m
EXISTING 1350mmø STORM SEWER
PROPOSED STORMTECH CHAMBERS OR
EQUIVALEN APPROVED BY ENGINEER
CHAMBERS AREA: 4000m²
CHAMBER DEPTH: 1.10m
BOTTOM OF CHAMBER: 63.06m
STORAGE REQUIRED: 4100m³
AVAILABLE STORAGE: 4400m³
NEW 3.08m – 250mmø PVC STM @ 0.97%
NEW 2.95m – 250mmø PVC STM @ 1.02%
NEW 8.09m – 250mmø PVC STM @ 0.99%
TRENCH DRAIN
CB08
T/G = 64.90m
INV. W = 63.19m
QUEEN ELIZABETH DRIVEWAY
NEW 1.85m – 250mmø PVC STM @ 1.03%
TRENCH DRAIN
CB06
T/G = 64.91m
INV. NE = 63.76m
EXISTING 900mmø STORM SEWER
0
5m
SCALE: 1:

PATERSON GROUP

☒ APPROVED ☐ REJECTED
☐ REVISE AND RESUBMIT ☐ REVIEWED AS NOTED

DATE

THIS REVIEW AND SEAL ARE FROM A GEOTECHNICAL PERSPECTIVE ONLY AND NOT A VERIFICATION ON THE QUANTITIES AND DIMENSIONS, WHICH ARE THE CONTRACTORS SOLE RESPONSIBILITY IN ORDER TO MEET ALL THE REQUIREMENTS OF THE CONTRACT AND SPECIFICATION DOCUMENTS. ALTERNATIVE PRODUCTS THAT MEET THE SAME TECHNICAL REQUIREMENTS CAN BE CONSIDERED AS LONG AS REVIEWED AND APPROVED BY PATERSON.

LICENSED PROFESSIONAL ENGINEER
May 5, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Technical Note

TN 6.50 Thermoplastic Liners for Detention Systems

Overview

StormTech chambers offer the distinct advantage and versatility that allow them to be designed as an open bottom detention or retention system. In fact, the vast majority of StormTech installations and designs are open bottom detention systems. Using an open bottom system enables treatment of the storm water through the underlying soils and provides a volume safety factor based on the infiltrative capacity of the underlying soils.

In some applications, however, open bottom detention systems may not be allowed. This memo provides guidance for the design and installation of thermoplastic liners for detention systems using StormTech chambers. The major points of the memo are:

- Infiltration of stormwater is generally a desirable stormwater management practice, often required by regulations. Lined systems should only be specified where unique site conditions preclude significant infiltration.
- Thermoplastic liners provide cost effective and viable means to contain stormwater in StormTech subsurface systems where infiltration is undesirable.
- PVC and LLDPE are the most cost effective, installed membrane materials.
- Enhanced puncture resistance from angular aggregate on the water side and from protrusions on the soil side can be achieved by placing a non-woven geotextile on each side of the geomembrane. A sand underlayment in lieu of the geotextile on the soil side may be considered when cost effective.
- StormTech does not design, fabricate, sell or install thermoplastic liners. StormTech recommends consulting with liner professionals for final design and installation advice.

Membrane Materials

Polyvinyl chloride (PVC) is an effective liner material for StormTech systems. PVC offers good chemical resistance to contaminant concentrations typical of highway runoff and to chlorides from road salting applications. Non-reinforced 30 mil PVC liners are recommended for StormTech systems. PVC is flexible. It can be folded without damage and is typically prefabricated and shipped to the jobsite. Panels as large as 20,000 sq. ft. can be prefabricated into a 4000 lb panel (30 mil is 0.195 lbs/sq. ft., SG = 1.2). PVC has the versatility to be field solvent welded, taped or field heat welded. A very significant advantage of PVC is that an excavation contractor can install a PVC liner without specialty crews. Solvent welding of seams, patches and pipe boots can all be done by the excavation contractor making PVC the lowest cost liner alternative.

The PVC compound include
xtend the service
life under exposure to sunlig
ent surface pool,
these additives can leach in
VC compounds
referred to as "fish safe" are sometimes used for surface pond liners and may be considered for StormTech liners. However, since StormTech systems are subsurface, there is no opportunity for UV attack by sunlight. Also, since stormwater is detained for short durations, typically 48 hours or less, there is little opportunity for accumulation of leachates. Therefore, PVC is an excellent membrane material for thermoplastic liner detention systems.

NOT APPROVED FOR THE SUBJECT SITE

Recommended Configuration: 30 mil PVC with 8-ounce non-woven geotextile underlayment and overlayment, open top with high flow bypass.

Recommended Restriction: Do not use for fuel spill containment.

Linear low density polyethylene (LLDPE) is a very inert material that offers excellent chemical resistance and is “fish safe”. LLDPE is an effective liner system for StormTech systems, particularly for small projects where the entire liner can be prefabricated in one piece or when using taped seams. LLDPE is flexible up to 30 mil but thicknesses greater than 30 mil should not be folded without potential damage. 30 mil LLDPE is recommended. Extra care should be taken to protect against puncture. A minimum 8-ounce non-woven fabric underlayment and 12-ounce overlayment should be specified. The underlayment should be increased to 12-ounce where water tightness is essential and increased puncture risk exists. Panels as large as 27,000 sq. ft. can be prefabricated into a 4000 lb roll (30 mil is 0.15 lbs/sq. ft.). LLDPE has a specific gravity less than 1.0. LLDPE seams can ~~be taped~~ or field heat welded. Installation costs may increase if field seaming by a specialty contractor is required.

Recommended Configuration: 30 mil LLDPE with 8-ounce non-woven geotextile underlayment and 12-ounce overlayment, open top with high flow bypass.

Recommended Restriction: Do not use for fuel spill containment.

A 40 mil LLDPE LINER, OR EQUIVALENT OTHER REVIEWED BY PATERSON, HAS BEEN APPROVED FOR THE SUBJECT SITE

~~***Reinforced Polypropylene (RPP), EPDM** and **XR-5*** are excellent materials for lining systems due to their flexibility, durability and excellent chemical and UV resistance. Although excellent lining materials, they generally exceed the engineering requirements for typical applications and are higher in cost than PVC or LLDPE. For fuel and oil concentrations normally found in storm water from parking and roadways, PVC, LLDPE and PP are suitable. However, if containment of aggressive contaminants, fuels or fuel spills are anticipated, a liner professional should be consulted. XR-5 in thicknesses of 30 mil or more, with welded seams may be suitable.~~

~~***Polyethylene (PE)*** materials are generally inert, offer excellent chemical resistance and are “fish safe”. Although ***medium density polyethylene (MDPE)*** liners are widely used for sanitary landfills and fish ponds, they are generally much higher in total cost and are not likely to be cost effective lining materials. ***High density polyethylene (HDPE)*** is not flexible enough to resist puncture and conform to the excavation. Cost aside, MDPE is an acceptable liner material for StormTech systems but should be limited to subgrades that are well prepared, without protrusions and must be field seamed.~~

Geotextile Materials

APPROVED OR EQUIVALENT OTHER REVIEWED BY PATERSON. IT SHOULD BE NOTED THAT A 12-OUNCE GEOTEXTILE MATERIAL SHOULD BE USED AS PROTECTION FOR THE 40 mil LLDPE THERMOPLASTIC LINER

6-ounce	AASHTO M288 Class 2 non-woven separation geotextile over the top of stone (ADS 601 or equal)
8-ounce	AASHTO M288 Class 2 non-woven geotextile for use as protection layer for PVC, RPP and LLDPE (ADS 801 or equal)
12-ounce	AREMA Chapter 1 Part 10 Category “Regular” non-woven geotextile for use as protection layer for LLDPE and other PE membranes (ADS 1201 or equal)

Seaming Options

1. ***Prefabricated vs. Field*** Prefabricated seams are preferable to field seams for all liner materials whenever possible.
2. ***Solvent Welded*** PVC only, low cost
3. ***Heat Welded*** Costly, require trained seamer, for all liner materials **APPROVED**
4. ***Taped*** Cost effective, M50-RC Gray distributed by Titus Industrial Group recommended, single sided, 4” width, for all liner materials. No water tightness data is available.
5. ***Overlapped*** Not water tight, no leakage rates available, suggest 4 ft overlap for all materials.

Pipe “boots” are used to seal pipe penetrations through the liner. Boots can either be prefabricated by the liner fabricator or field fabricated by the contractor. The boot is then solvent cemented, heat welded or taped to the liner. A pipe clamp is normally used to seal the boot around the pipe. Seaming and sealing pipe boots at low temperatures (32° F minimum) requires preheating of the material.

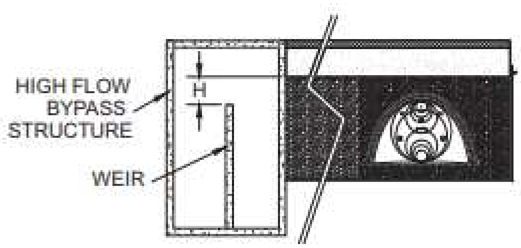
Design

General The design of a lined system must be performed by the consulting engineer and, at minimum, requires knowledge of design storage, peak flow rates and maximum seasonal high groundwater elevation. This information is used to design the peak flow control structure, maximum liner height and groundwater control (if necessary).

High Flow Bypass A high flow control is an important component for any lined system. The high flow control is designed to pass the peak flow while ensuring that the liner is not overtopped. The control structure can be an upstream high flow bypass or a downstream overflow structure. In both cases, a high flow weir, very similar to the high flow control in a pond outlet control structure, is normally used. The high flow weir should be sized such that the water surface elevation based on the maximum head on the weir is less than the top of the liner. Additional freeboard should be provided.

In a typical upstream bypass design, the calculated depth of flow over the weir (H) is subtracted from the maximum water surface elevation in the chamber system to establish the weir crest elevation. The storage in the chamber system associated with the weir crest elevation may be a design constraint. The designer may choose to increase the weir length and therefore decrease the flow depth to establish a higher weir crest.

The equation for a rectangular weir is:



$$H = (Q / (C_d \times L))^{2/3}$$

Where: Q = flow over the weir (cfs)

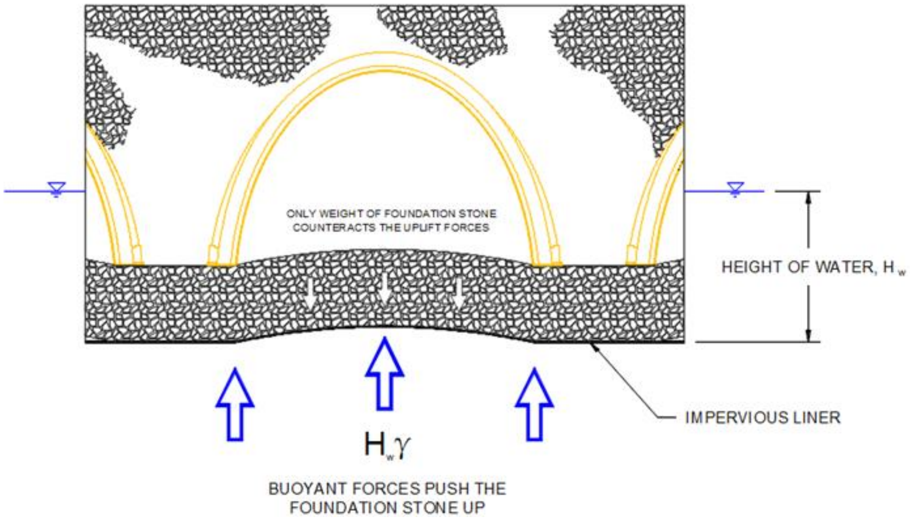
C_d = discharge coefficient = 3.3

H = Depth of flow over crest (ft)

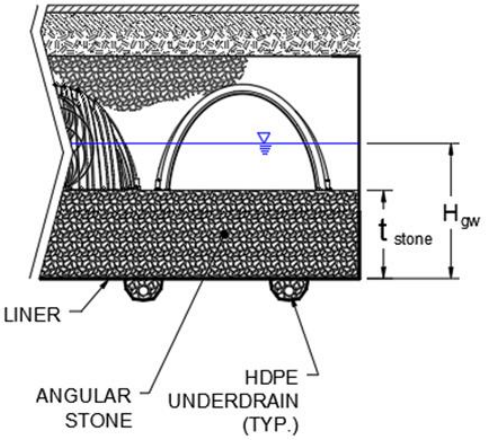
L = length of weir (ft)

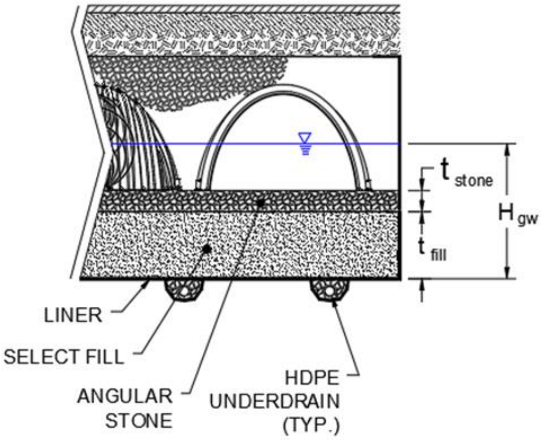
In a typical downstream overflow design, the designer may incorporate one or more low flow orifices into the high flow weir wall. The weir crest is established as described above but hydraulic losses from the inlet to chamber to the outlet structure may need to be considered. Losses may be factored in by lowering the weir crest or increasing the liner freeboard.

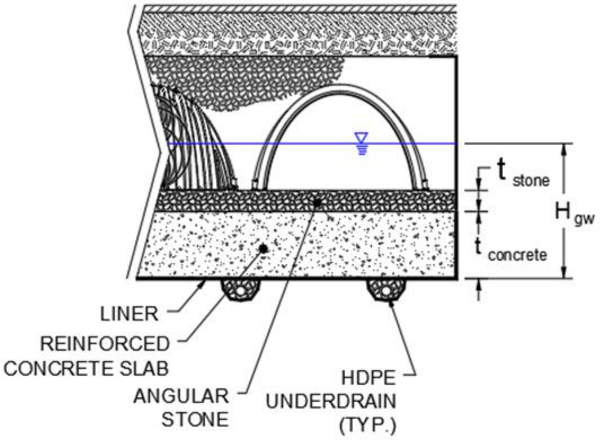
Buoyancy ADS recommends against installing lined chamber systems below groundwater. Although the total weight of a chamber system generally exceeds the buoyant force, a limiting stability condition may result when the buoyant pressure exceeds the resistance pressure directly under the chamber. This could result in a heave of the bedding under the chamber leading to instability.



To prevent adverse impacts from ground water, where gravity discharge is possible, ADS recommends the installation of an underdrain system under the liner. Where there is a potential buoyant force, ADS recommends a sufficient bedding thickness, such that the weight of bedding exceeds the maximum buoyant force. The additional bedding thickness can be either increasing the foundation stone or adding a combination of foundation stone and select fill beneath the feet of the chamber and inside the liner to counteract these uplift forces.



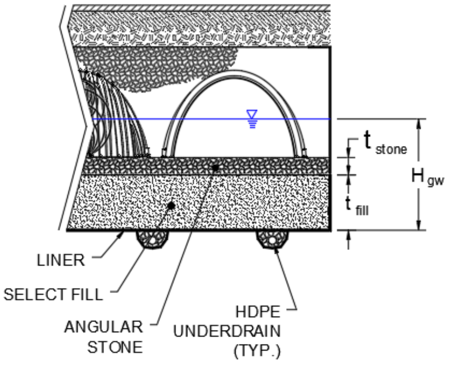




The bedding thickness calculation is simplified by ignoring any structural contribution from the liner and reinforcing material and considering only the weight of the stone or stone/fill in the thinnest area of the bedding, which is located under the chamber.

The relationship between bedding thickness and maximum allowable groundwater elevation is:

Select Fill and Foundation Stone Option



H_{gw} x (62.4 lb/ft^3) = [(γ_{stone} x t) + [(γ_{fill} x t)] / SF

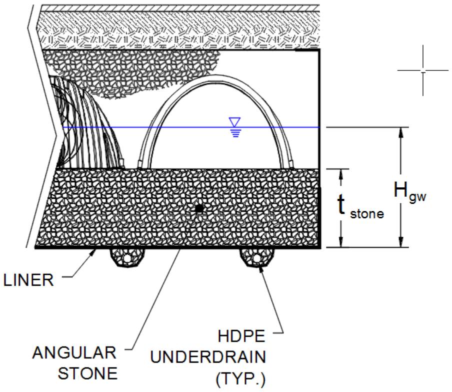
Where:

γH_{gw}= height of groundwater above liner bottom (in)
γ_{stone} = bulk density of bedding stone (lb/ft^3)
t_{fill} = thickness of fill bedding (in)
t_{stone} = thickness of stone bedding (in)
SF = safety factor (1.25 typical minimum)*

The bulk density of the open graded stone bedding materials varies from about 75 lbs/ft3 to over 100 lbs/ft^3. The bulk density of select fill materials varies from about 90 lbs/ft^3 to over 120 lbs/ft^3. Without specific bulk density information for the stone actually used, ADS recommends using not more than 75 lbs/ft^3.

* The consulting engineer may apply a lower or higher safety factor.

Increased Foundation Stone Option

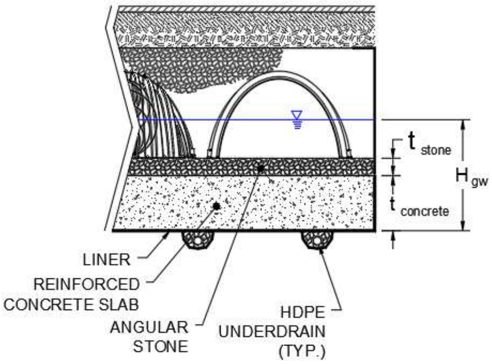


H_{gw} x (62.4 lb/ft^3) = [(γ_{stone} x t) / SF

Where:

γH_{gw}= height of groundwater above liner bottom (in)
γ_{stone} = bulk density of bedding stone (lb/ft^3)
t = thickness of stone bedding (in)
SF = safety factor (1.25 typical minimum)*

Reinforced Concrete Slab



Alternatively, an engineer can design a reinforced concrete slab. The slab should be designed to handle the uplift forces from groundwater while transferring live & dead loads to the underlying material.

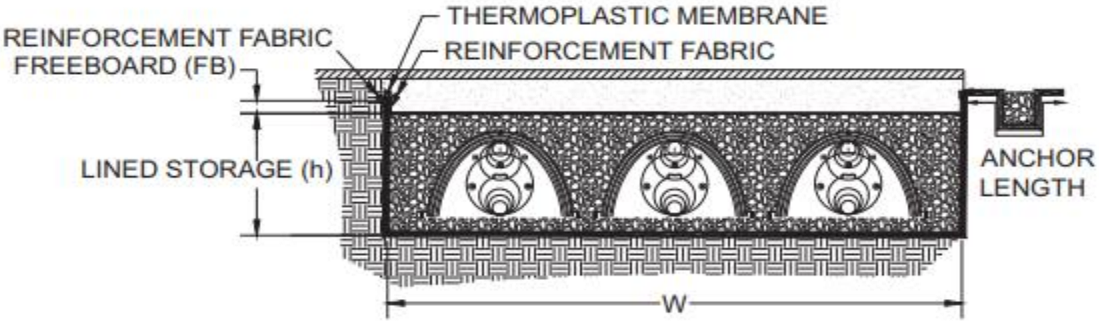
Installation

Installation should be in accordance with the liner manufacturer's instructions. Associations representing membrane materials have developed installation standards and other support documents for the respective lining materials. Visit their web sites for additional information.

- PVC Geomembrane Institute, University of Illinois, web: http://Pgi-tp.cee.uiuc.edu/forweb
- "HDPE Geomembrane Installation Specification" by the International Association of Geosynthetic Installers. Revised February 2000: http://www.iagi.org/specifications.htm

PVC and LLDPE liners should not be installed at temperatures less than 32° F or on windy days. Wind can catch the liner and be extremely dangerous to laborers. Stones and other protrusions should always be removed from the excavation. Rolling or compacting is recommended to knock down any remaining protrusions. The non-woven underlayment fabric is then placed in the excavation, the membrane placed, and a fabric reinforcement placed over the membrane. Liners are flapped by laborers to get air under the liner to enable easy drag across bed. Corners are generally formed by folding or "pleating" excess liner material.

An “anchor trench” about 12” deep by 12” wide may be dug around the top of the excavation to anchor the top of the reinforcement fabric and thermoplastic liner at the top of the excavation. Stone should be placed carefully to avoid puncture from long free falls. Similarly, additional care must be taken when spreading and compacting bedding stone to prevent stones from puncturing the liner during construction.



Estimating Liner Material

Liner fabricators require dimensional details to design panels and provide firm material quotations. The liner and reinforcing fabric quantities should include sidewalls and extra material for anchoring during installation. The excavation contractor should use care not to over excavate since a larger excavation would require additional liner materials.

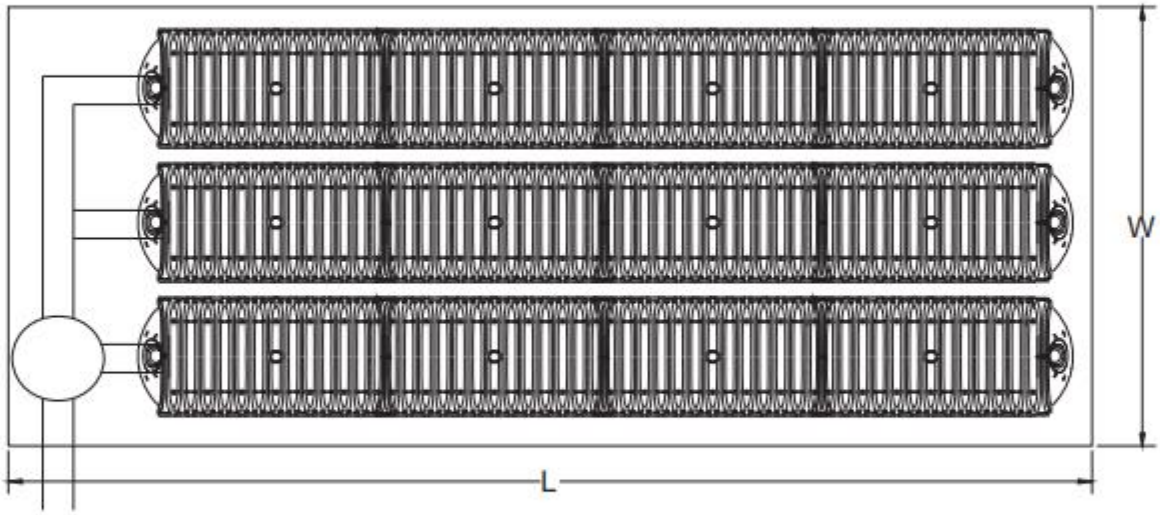
The fabricated sheet size for estimating purposes is calculated as follows:

Panel Size = [W + 2(h + FB + AL)] x [L + 2(h + FB + AL)]

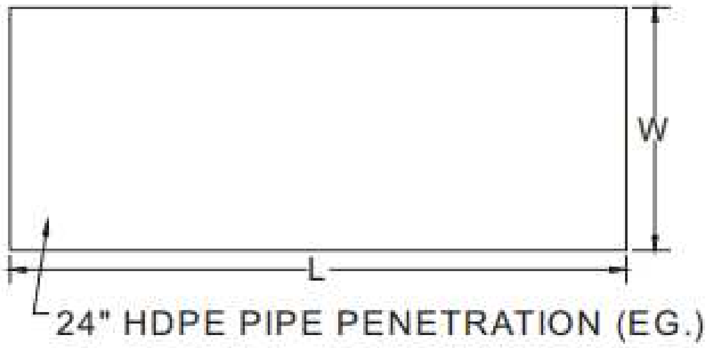
Where:

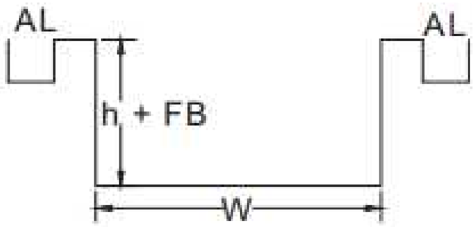
W = system width from StormTech layout drawing
L = system length from StormTech layout drawing
FB = freeboard based on engineer’s advice (0.5’ typical)
AL = anchor length of membrane and reinforcement to tie back sidewall material during installation and backfill of chambers (4’ typical)

The location and size of pipe penetrations should also be summarized for the fabricator.



Estimating Worksheet:





Panel Size = [W + 2(h + FB + AL)] x [L + 2(h + FB + AL)]

PATERSON GROUP

☒ APPROVED ☐ REJECTED
☐ REVISE AND RESUBMIT ☐ REVIEWED AS NOTED

THIS REVIEW AND SEAL ARE FROM A GEOTECHNICAL PERSPECTIVE ONLY. ALTERNATIVE PRODUCTS THAT MEET THE SAME TECHNICAL REQUIREMENTS CAN BE CONSIDERED AS LONG AS REVIEWED AND APPROVED BY PATERSON.

May 5, 2025
D. J. PETAHTEGOOSE
100568013

terrafix® Geomembrane

40mil LLDPE Smooth

Technical Data Sheet

PHYSICAL PROPERTIES

PROPERTY	TEST METHOD	FREQUENCY(1)	UNIT Metric	TERRAFIX 840-2000
Thickness (min. avg.)	ASTM D-5199	Every roll	mm	1.0
Thickness (min.)	ASTM D-5199	Every roll	mm	0.9
Resin Density	ASTM D-1505	1/Batch	g/cc	<0.926
Melt Index - 190/2.16 (max.)	ASTM D-1238	1/Batch	g/10 min.	1.0
Sheet Density	ASTM D-1505	Every 2 rolls	g/cc	<0.939
Carbon Black Content	ASTM D-4218	Every 2 rolls	%	>2.0 / <3.0
Carbon Black Dispersion	ASTM D-5596	Every 6 rolls	Category	Cat. 1 / Cat. 2
Oxidation Induction Time (min. ave)	ASTM D-3895	1/Batch	min.	100
Tensile Properties (min. avg) (2)	ASTM D-6693	Every 2 rolls		
Strength at Break			kN/m	28
Elongation at Break			%	800
2% Modulus (max.)	ASTM D-5323	Per formulation	kN/m	420
Tear Resistance (min. avg.)	ASTM D-1004	Every 6 rolls	N	100
Puncture Resistance (min. avg.)	ASTM D-4833	Every 6 rolls	N	276
Dimensional Stability	ASTM D-1204	Every 6 rolls	%	+/- 2
Multi-Axial Tensile (min.)	ASTM D-5617	Per formulation	%	30
Oven Aging - % retained after 90 days	ASTM D-5721	Per formulation		
STD OIT (min. avg.)	ASTM D-3895		%	35
HP OIT (min. avg.)	ASTM D-5885		%	60
UV Resistance - % retained after 1600 hrs	GRI-GM-11	Per formulation	%	
HP OIT (min. avg.)	ASTM D-5885			35

SUPPLY SPECIFICATIONS

(Roll dimensions may vary +/-1%)

Roll Dimension - Width	m	6.80
Roll Dimension - Length	m	237.8
Area (Surface/Roll)	m²	1617

NOTES:

1. Testing frequency based on standard roll dimensions and one batch is approximately 180,000 lbs (or one railcar).
2. Machine Direction (MD) and Cross Machine Direction (XMD or TD) average values should be on the basis of 5 specimens each directions.

* All Value are nominal test results, except when specified as minimum or maximum.

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terrafix®
environmental technology inc.

February 19th, 2025

Winston Yang, P.Eng., PMP
WSP Canada Inc.
2611 Queensview Drive, Suite 300
Ottawa, ON K2B 8K2
Winston.Yang@wsp.com

Subject: Structural Analysis Results for StormTech MC-3500 System – Lansdowne 2.0 in Ottawa, ON

To Whom it May Concern,

As requested, we are providing the results of the previously conducted structural analysis for the StormTech MC-3500 system installed at Lansdowne 2.0 in Ottawa, ON. This analysis evaluates whether the system can withstand additional dead loads, including heavy concrete blocks, the 120 RT crane, and the SAM575 Covered Wings stage. The analysis is based on the design layout (Revision 2), the plan set dated 2024-01-15, and the 120 RT crane specifications.

The system must be installed in accordance with all applicable Advanced Drainage Systems (ADS) specifications and construction installation procedures. If any of the specifications and assumptions are incorrect, the analysis is considered void, and updated information must be provided to ADS for a revised analysis.

See below for the findings and recommendations:

- Dead Load Capacity: **The system can support an additional 4,769 kg/m² of dead load** pressure before failure. While the concrete blocks can be placed long-term, **the engineer of record must verify that total additional loads do not exceed this threshold.**

- Crane Loading:
    - The 120-ton crane requires an effective outrigger area of at least 3.73 m² (5,776 in²). A possible configuration would be 1.93m × 1.93m (76 in × 76 in).

  - Under these conditions:
    - The crane can drive over the MC-3500 chambers for up to one (1) week. If the axle load remains in place for more than one week, it must be removed from the chambers.
    - The crane can operate on the outriggers for up to 8 hours and must be removed from the chambers past this period.
  - The engineer can reduce outrigger ground bearing pressure to 14,405 kg/m² (20.5 psi).

- Stage Load Distribution:
  - The 4 ft × 8 ft pad transfers 1,080 kg/m², and the 4 ft × 4 ft pad transfers double that amount.

Please note, **the 0.731m minimum cover over the system must meet or exceed the required values outlined in the design layout for safe crane operation.**

ADS strongly recommends minimizing load durations whenever possible and does not condone construction vehicle parking over StormTech systems.

Please let us know if you require any additional details or clarification.

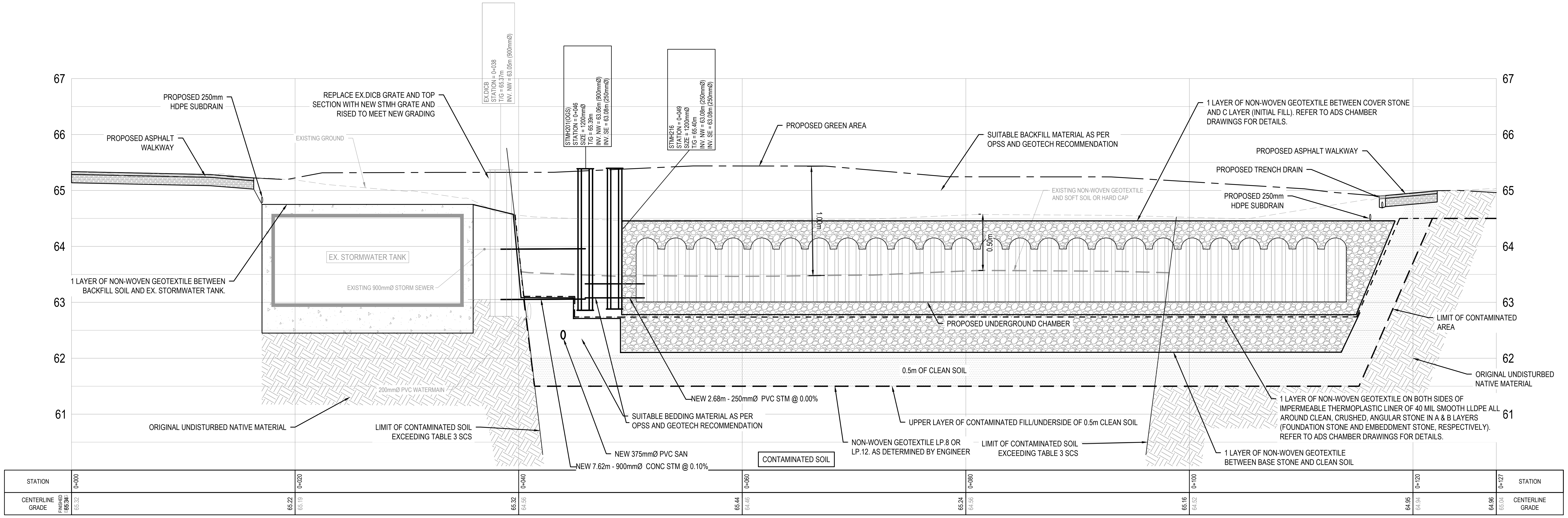
Best regards,

Advanced Drainage Systems, Inc

Graeme Caso

By: Rose Marie Nita Dorminie
Project Engineer I
Rosemarie.dorminie@adspipe.com
(514) 662-5663

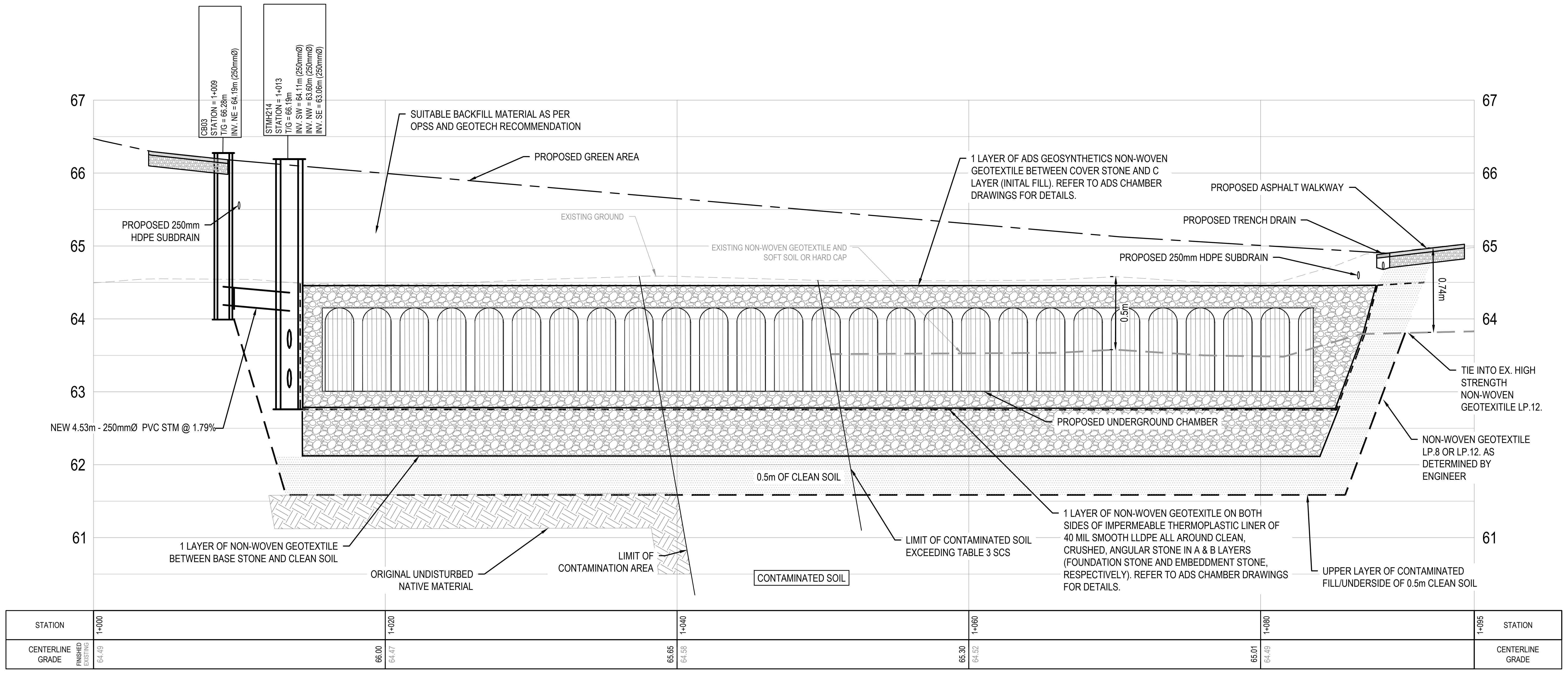
Verified by: Graeme Caso
Project Engineer II
Graeme.caso@adspipe.com
(860) 920-4362



UNDERGROUND CHAMBER CROSS SECTION B

B RM04

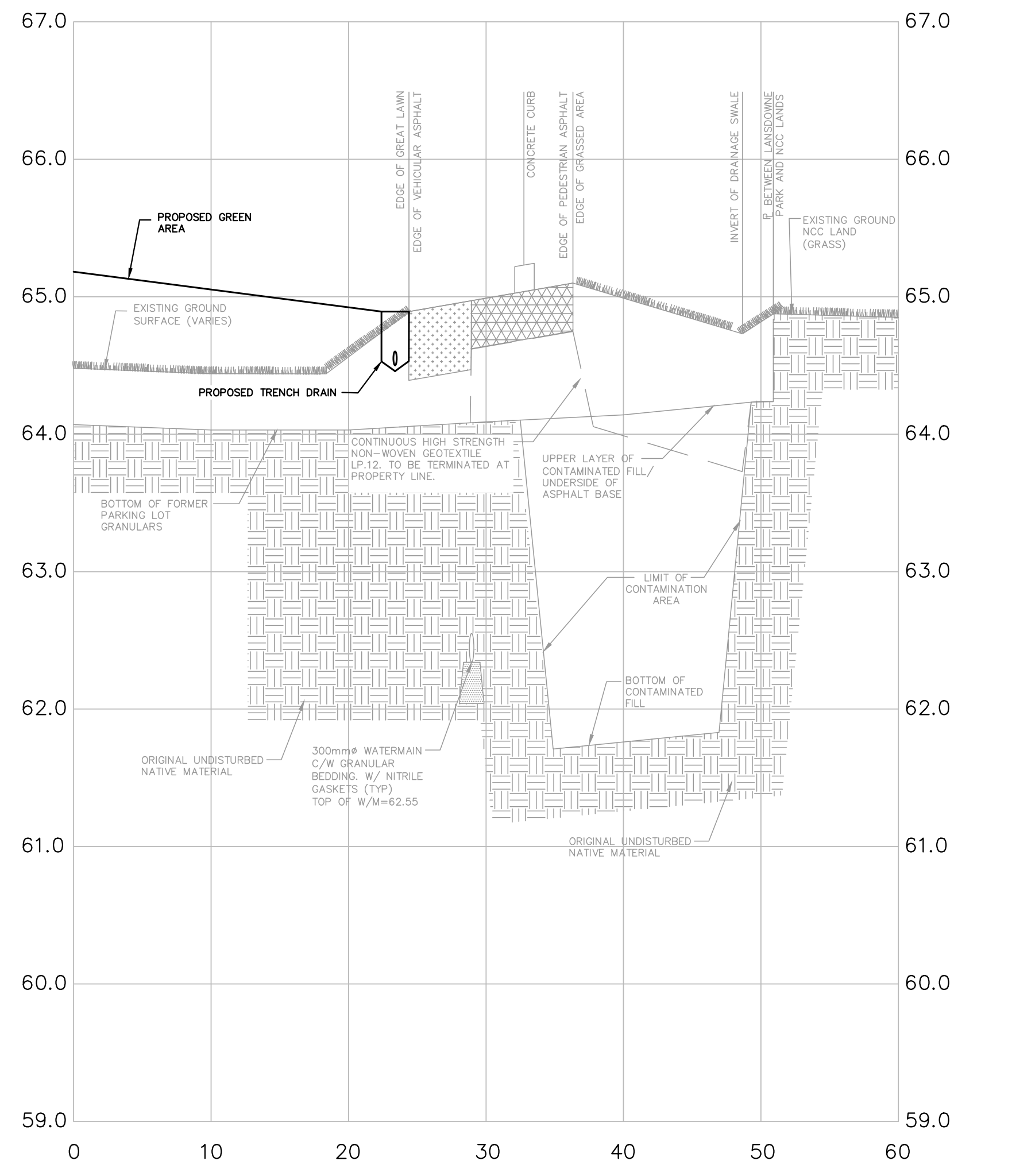
SCALE: H 1:200, V 1:40



UNDERGROUND CHAMBER CROSS SECTION C

C RM04

SCALE: H 1:200, V 1:40



CONTAMINATION ZONE FOR McElroy SITE CROSS SECTION F

F RM04

SCALE: H 1:300, V 1:30

DATE PLOTTED:

Ottawa

OSEG
Ottawa Sports and Entertainment Group

wsp
1931 ROBERTSON ROAD BUILDING A AND B.
OTTAWA, ONTARIO K2H 5B7
(613) 592-9600
ENVIRONMENTAL MANAGEMENT

wsp
2011 QUEENSVIEW DR.
OTTAWA, ONTARIO K2B 8K2
(613) 829-2800
CIVIL ENGINEER

NOT FOR CONSTRUCTION

NO.	DESCRIPTION	DATE
4	REVISED PER CITY COMMENTS	2025-06-20
3	ISSUED FOR PERMIT	2025-06-16
2	ISSUED FOR 100% CD FOR TENDER	2025-06-12
1	ISSUED FOR RISK MANAGEMENT PLAN	2025-05-20

REVISIONS/ ISSUES

CONTRACTOR SHALL CHECK AND VERIFY ALL DIMENSIONS AND REPORT ANY OMISSIONS OR DISCREPANCIES TO THE ARCHITECT BEFORE PROCEEDING WITH THE WORK.
DO NOT SCALE THE DRAWINGS

SEAL

D. B. YANG
100230568
2025-06-20

DRAWN J.T
DATE 2025/06/20
CHECKED D.Y/ P.H

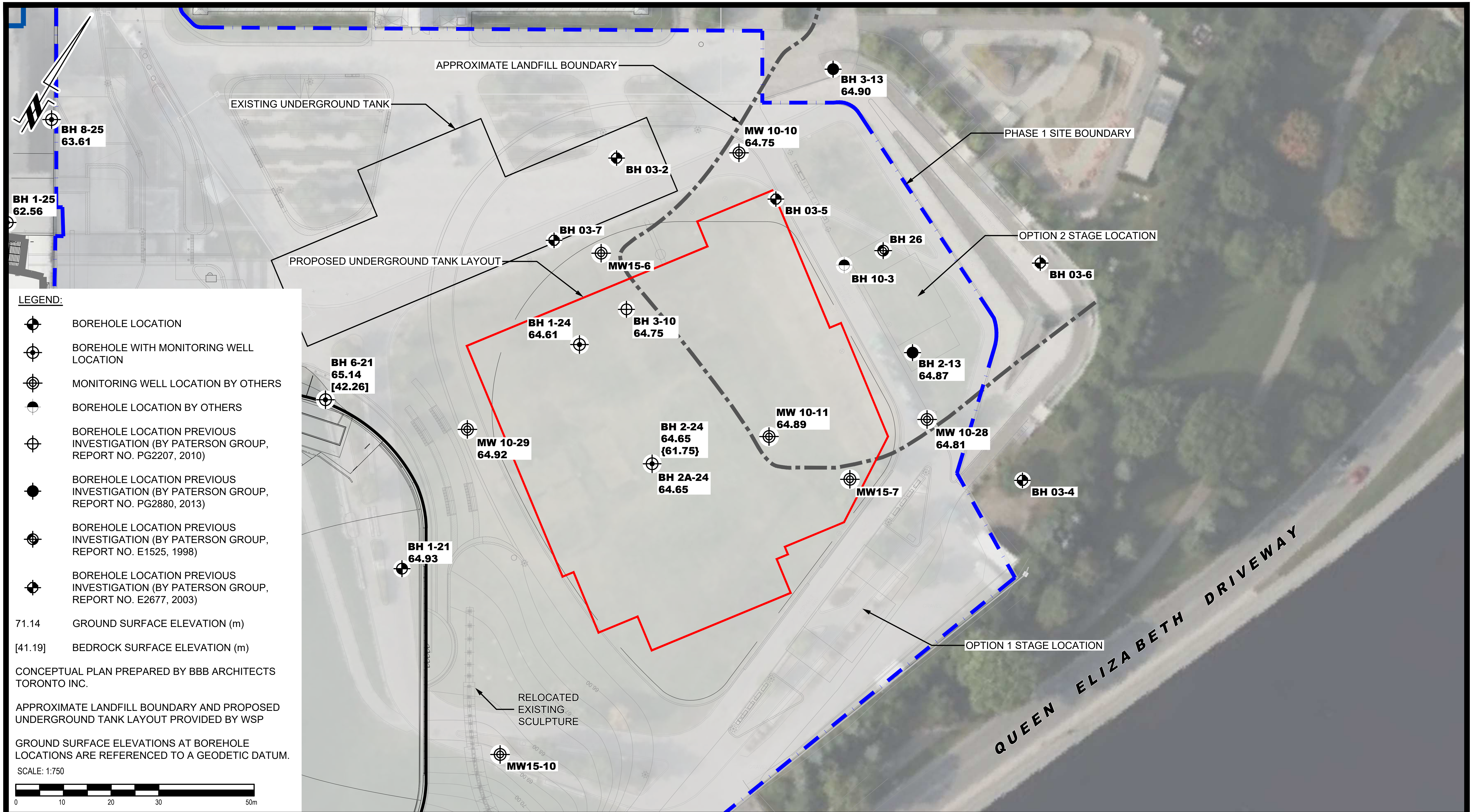
LANSDOWNE EVENT CENTRE
945 & 1015 BANK STREET

DWG. TITLE
LANSDOWNE 2.0
RISK MANAGEMENT PLAN
SECTIONS B & C

SCALE 1:200
DWG. NO. RM04
PROJ. NO. CA0033920.1056

#D07-12-24-0082

#19172



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
⊕	MONITORING WELL LOCATION BY OTHERS
◓	BOREHOLE LOCATION BY OTHERS
⊕	BOREHOLE LOCATION PREVIOUS INVESTIGATION (BY PATERSON GROUP, REPORT NO. PG2207, 2010)
●	BOREHOLE LOCATION PREVIOUS INVESTIGATION (BY PATERSON GROUP, REPORT NO. PG2880, 2013)
⊕	BOREHOLE LOCATION PREVIOUS INVESTIGATION (BY PATERSON GROUP, REPORT NO. E1525, 1998)
⊕	BOREHOLE LOCATION PREVIOUS INVESTIGATION (BY PATERSON GROUP, REPORT NO. E2677, 2003)
71.14	GROUND SURFACE ELEVATION (m)
[41.19]	BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PREPARED BY BBB ARCHITECTS TORONTO INC.

APPROXIMATE LANDFILL BOUNDARY AND PROPOSED UNDERGROUND TANK LAYOUT PROVIDED BY WSP

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

SCALE: 1:750

0 10 20 30 50m

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

NO.	REVISIONS	DATE	INITIAL

CITY OF OTTAWA
GEOTECHNICAL INVESTIGATION
LANDSDOWNE PARK REDEVELOPMENT
PROPOSED UNDERGROUND STORMWATER TANK SYSTEM
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750
Drawn by: ZS
Checked by: FC
Approved by: DP
Date: 05/2025
Report No.: PG6655-MEMO.09
Dwg. No.: PG6655-3
Revision No.:

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11x17