Geotechnical Investigation

Proposed High-Rise Buildings

2000 City Park Drive
Ottawa, Ontario

Prepared for Colonnade BridgePort

Report PG6552-2 dated July 17, 2026

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Colonnade BridgePort to undertake a geotechnical investigation for the proposed high-rise buildings to be located at 2000 City Park Drive in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

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

❑ Provide geotechnical recommendations for the 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.

2.0 Proposed Development

Based on the available conceptual drawings, it is understood that the proposed development at 2000 City Park Drive will consist of five (5) high-rise residential buildings. Block 4 (Tower A) is proposed as a 17-storey building, while Blocks 1, 2, and 3 are each proposed as 30-storey buildings. Block 5 is anticipated to be a high-rise building with fewer than 17 storeys; however, the final building height for Block 5 and design details were not available at the time of preparation of this report.

At finished grades, the proposed buildings will be surrounded by asphalt-paved access lanes and landscaped open spaces. A public park is proposed within the north-western portion of the 2000 City Park Drive parcel. The proposed development is expected to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out between January 17 and 19, 2023 (BH 1-23 to BH 5-23), and on November 3, 2023 (BH 7-23), and consisted of a total of six (6) boreholes advanced to a maximum depth of 12.4 m below existing grade, including coring bedrock. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground services and available access.

A previous geotechnical investigation was conducted at the subject site by others on July 30 and 31, 2007, consisting of five (5) boreholes advanced to a maximum depth of 4.8 m below the existing ground surface.

The locations of the boreholes are shown on Drawing PG6552-2 – Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low-clearance track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering and rock coring to the required depths at the selected borehole locations, and sampling and testing the soil and bedrock.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Bedrock samples were recovered from all boreholes using a core barrel and diamond drilling techniques. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the Soil Profile and Test Data Sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the bedrock quality.

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

A standpipe piezometer was installed in each borehole upon the completion of drilling and sampling, in order to permit monitoring of the groundwater levels. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a high-precision handheld GPS with respect to a geodetic datum. The locations of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG6552-2 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples and bedrock cores were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples from the November 2023 investigation will be stored in the laboratory for one (1) month after this report is completed. They will then be discarded unless we are otherwise directed.

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 for sulphate attacks against subsurface concrete structures. The samples were tested to determine the concentration of sulphate and chloride, and the resistivity and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and has a grass surface, with localized areas of mature trees.

The site is bordered by City Park Drive to the north, a commercial property to the west, the Confederation Line and Queensway (Highway 417) to the south, and a high-rise residential development to the east.

The site is relatively level and generally at grade with the adjacent roadways and surrounding properties. The existing ground surface across the subject site ranges from approximately geodetic Elevation 72.6 m to 74.3 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface condition at the subject site consists of a surficial layer of topsoil or asphalt which is underlain by fill and glacial till. The fill was generally observed to consist of silty sand to silty clay with gravel, crushed stone, brick, and organics, extending to approximate depths varying from 0.2 to 1.8 m below the existing ground surface.

A glacial till deposit was generally encountered underlying the fill, consisting of undisturbed, compact to very dense, brown silty sand with gravel and shale fragments.

Bedrock

Bedrock was encountered at depths ranging from 1.5 to 3.8 m and was cored at all borehole locations. Based on the recovered rock core, the bedrock was observed to consist of black shale, with the upper 1 to 2 m of the bedrock being generally weathered and very poor to poor quality, becoming good to excellent in quality with depth. The bedrock was cored to a maximum depth of about 12.4 m below the existing grade. In addition, based on available geological mapping, the bedrock consists of shale of the Billings Formation and is expected to be encountered at depths ranging from 2 to 5 m.

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

4.3 Groundwater

The groundwater levels were measured in piezometers on January 31, 2023 for boreholes BH 1-23 through BH 5-23, and on November 13, 2023, for borehole BH 7-23. The observed groundwater levels are summarized in Table 1 below.

Table 1 – Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1-23	73.20	2.39	70.81	Jan. 31, 2023
BH 2-23	73.95	5.15	68.80	Jan. 31, 2023
BH 3-23	73.06	3.83	69.23	Jan. 31, 2023
BH 4-23	74.26	5.00	69.26	Jan. 31, 2023
BH 5-23	72.56	3.43	69.13	Jan. 31, 2023
BH 7-23	73.89	3.81	70.08	Nov. 13, 2023

Note: Ground surface elevations at borehole locations were surveyed by Paterson and are referenced to a geodetic datum.

It should be noted that surface water can become trapped within a backfilled borehole, which can lead to higher than typical groundwater level observations.

The long-term groundwater level can also be estimated based on the observed colour, moisture content and consistency of the recovered samples. Based on these observations, the long-term groundwater level is expected to range between approximately **3.5** to **4.5 m** below ground surface.

However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed buildings are recommended to be founded on conventional spread footings bearing on the clean, surface sounded shale bedrock.

Bedrock removal will be required to complete the underground parking levels. Further, expansive shale bedrock could be present on site, and precautions should be provided during construction to reduce the risks associated with the potentially heaving shale bedrock.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious material, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Due to the anticipated founding level for the proposed buildings, all existing overburden material will be excavated from within the proposed building footprints. Bedrock removal will be required for the construction of the underground parking levels.

Fill Placement

Engineered fill placed for grading beneath the proposed buildings, where required, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

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

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

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Bedrock Excavation Face Reinforcement and Preparation

Bedrock excavation face reinforcement methods, such as the use of horizontal rock anchors and rock wedges/bolts in conjunction with shotcrete and/or chain link fencing with a layer of woven geotextile connected to the excavation face is expected to be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. Further, shotcrete and/or other material may be required to in-fill areas where bedrock pop-outs occur due to the nature of bedrock removal process throughout the excavation footprint and in advance of the placement of foundation waterproofing products.

The requirement for bedrock excavation face reinforcement should be evaluated by Paterson personnel during the excavation operations. As a preliminary recommendation, provisions should be carried out for providing a minimum 1 m wide bedrock face protection layer across building excavation footprint perimeters for all portions of the excavations that will extend below the bedrock surface. Throughout the building excavation and bedrock removal process, the vertical bedrock excavation perimeter surfaces should be hoe-rammed and grinded smooth to provide a relatively flat substrate surface for the placement of the drainage board. All loose bedrock fragments should be removed by grinding operations.

It is recommended that Paterson review the bedrock excavation program at the time of construction to evaluate and select the recommended locations for reinforcement, if any are present.

Overbreak in Bedrock

Sedimentary bedrock formations, such as sandstone, limestone, dolomite, and shale, contain bedding planes, joints and fractures, and mud seams which create natural planes of weakness within the rock mass. Although several factors of a blast may be controlled to reduce backbreak and overbreak, upon blasting, the rock mass will tend to break along natural planes of weakness that may be present beyond the designed blast profile. However, estimating the exact amount of backbreak and overbreak that may occur is not possible with conventional construction drill and blast methods.

Backbreak should be expected to occur along the perimeter of the building excavation footprint with conventional drill and blast bedrock removal methods. Further, overbreak is expected to occur throughout the lowest lifts of blasting due to the variable bedding planes and planes of weakness in the in-situ bedrock. It is very difficult to mitigate significant overblasting given the constraints posed by footing geometry and spacing with respect to the zone of influence of blasts and the bedrocks in-situ characteristics.

Depending on the methodology undertaken by the contractor, efforts taken to minimize backbreak and overbreak may add significant time and costs to the excavation operations and is not guaranteed to completely eliminate the potential for backbreak and overbreak. Overbreak below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

As such, volume estimates of bedrock to be removed may not be reflective of the actual volume of bedrock that may be required to be removed at the time of construction. This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss. It is recommended that the blasting operations be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

It is highly recommended that the contractor/owner engage Paterson early into the excavation activities to identify anomalies within the bedrock. Pre-blast surveys and vibration monitoring is also very essential at this stage, which can be completed by Paterson as well. Also, it is important to note that completing an as-built of bedrock removal, including overbreaks, is required to be completed by the contractor and reviewed by Paterson to avoid confusion on the reasons for overbreaking the rock and to ensure that all parties are aware of the site conditions. Paterson should receive all as-built surveys post removal of the bedrock.

Lean Concrete Filled Trenches

Where the proposed footings are to be founded on bedrock which is located below the underside of footing elevation, zero-entry vertical trenches should be excavated to the clean, surface sounded shale bedrock, and backfilled with lean concrete to the founding elevation (minimum **17 MPa** 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Footings supported directly on clean, surface sounded shale bedrock, or on lean concrete which is placed directly on clean, surface sounded shale bedrock, can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **3,000 kPa**. A reduced bearing resistance value at Ultimate Limit States (ULS) of **750 kPa** is recommended for areas where the bedrock is found to be weathered. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS. Strip and pad footings should have minimum plan dimensions of 0.6 and 1 m, respectively.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings supported on clean, surface-sounded bedrock designed for the bearing resistance values provided herein will be subjected to negligible post-construction total and differential settlements.

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 a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

A weathered bedrock bearing medium, will require a lateral support zone of 1H:1V (or shallower).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class X_C**. If a higher seismic site class (such as Class **X_B**) is required, then a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed buildings, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2024 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

With the removal of all topsoil and deleterious fill from within the footprints of the proposed buildings, the bedrock will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

It is anticipated that the underground levels for the proposed building will be mostly parking, and the recommended pavement structure noted in Section 5.8 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 300 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone under the lower basement floor of the proposed multi-storey building. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed buildings. 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 drained unit weight of 20 kN/m^3 (effective unit weight 13 kN/m^3).

However, the majority of the basement walls of the proposed buildings are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face, for which a nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). Further, a seismic earth pressure component will not be applicable for the foundation walls which are poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

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) and 0.05 for rock
> γ = unit weight of fill of the applicable retained soil (kN/m3)
> H = height of the wall (m)

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

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

Seismic Earth Pressures

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

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a \ \cdot H^2/g$ 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 0.364g according to OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o \cdot \gamma \cdot H^2$, where K = 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 2024.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in shale bedrock is based upon two possible failure modes. The rock anchor can fail either by shear failure along the grout/rock interface or by pullout at 60 to 90 degree cone of rock 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 individual anchor load capacity.

A third failure mode of shear failure along the grout/steel interface should be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

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 fluid does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not, prior to servicing. To resist seismic uplift pressures, a passive rock anchor system is adequate. However, a post-tensioned anchor will absorb the uplift load pressure with less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a fixed anchor length at the anchor base, which will provide the anchor capacity, and a free anchor length between the rock surface and the top of the bonded length. As the depth at which the apex of the shear failure cone develops midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a sleeve to act as a bond break, with the sleeve filled with grout. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International.

Grout to Rock Bond

Generally, the unconfined compressive strength of shale ranges between 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, should be provided. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

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

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2 in the following:

Table 2 – Parameters used in Rock Anchor Design

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Shale Bedrock Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Shale bedrock	40 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60°
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. The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 3 below for vertical grouted rock anchors used to resist uplift forces. The factored tensile resistance values provided are based on a single anchor with no group influence effects.

Table 3 – Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.0	1.0	3.0	450
	2.6	1.0	3.6	600
	3.2	1.3	4.5	750
	4.5	2.0	6.5	1000
125	1.6	1.0	2.6	600
	2.0	1.2	3.2	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

Temporary Horizontal/ Inclined Rock Anchors

Horizontal or inclined rock anchors may be required at specific locations to prevent pop-outs and slippage of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface. The requirement for horizontal or inclined rock anchors will be evaluated during the excavation operations to provide a sufficient bedrock stabilization for workers safety during construction.

Bedrock Stabilization

A weathered bedrock layer was noted within the upper 2 m of the bedrock on site. It is therefore expected that bedrock stabilization and protection measures will be required on site. Those measures should consist of anchored chain-link fencing and geotextile placed on the exposed excavation face to ensure that small rock fragments do not fall onto workers in close proximity to the excavation walls.

Paterson will need to complete a full review of the excavation face during rock removal to review the stabilization requirements. At this point, given the known information about the project, the following recommendations are made for budgetary purposes. Final field requirements will depend on the exact bedrock conditions encountered during construction.

Bedrock grinding is expected for this site to allow installation of the proposed drainage and waterproofing system. Once completed, a review of the upper portion of the bedrock layer should be completed by Paterson to establish the stabilization measures required.

Bedrock stabilization shall consist of chain link fencing with a woven geotextile liner, such as Terratrack 200 or equivalent, placed over the upper weathered bedrock in order to stabilize any loose rock present. The top of the chain link fencing should be secured by a series of bolts extended into the top of the bedrock bench. The weathered layer of rock was noted to range from 1 to 2 m in depth. The chain link fencing should be anchored below the weathered layer in good-quality rock using a series of minimum 150 mm length rock wedge bolts spaced no greater than 1.5 m apart.

Rock bolts, such as swellex bolts or equivalents, might be required to stabilize seams or large blocks of bedrock. The anchor spacing and length should be established on site following the bedrock review. Table 4 provides the recommended anchor lengths for a 75 and 125 mm diameter hole into the bedrock face at approximately 25 degrees below the horizontal, which will further secure the chainlink fence in place. If the DYWDAG threaded bars are chosen, it is recommended that a minimum 32 MPa grout be used.

Table 4 - Recommended Inclined Rock Anchor Lengths for Bedrock Stabilization - Grouted Rock Anchor *

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.9	2.9	4.8	450
	2.6	3.2	5.8	600
	3.2	3.3	6.5	750
	3.8	3.4	7.2	900
125	1.0	3.4	4.4	450
	1.3	3.8	5.1	600
	1.6	4.1	5.7	750
	1.9	4.4	6.3	900

* Rock anchors are inclined approximately 25 degrees below the horizontal for bedrock stabilization and protection measures.

Other Considerations

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

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

5.8 Pavement Design

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

Table 5 – Recommended Rigid Pavement Structure – Underground Parking Level	
Thickness (mm)	**Material Description**
150	**Exposure Class C2** – **32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

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

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

Table 6 – Recommended Asphalt Pavement Structure – Access Lanes and Heavy Loading Parking Area	
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
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type II overlying the concrete podium deck	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular (base and subbase) should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Groundwater Control for Construction

Foundation Drainage and Waterproofing

The following recommendations may be considered for the architectural design of the buildings' foundation drainage systems. It is recommended that Paterson be engaged at the design stage of the future building (and prior to tender) to review and provide supplemental information for the building foundation drainage system design.

Supplemental details, review of architectural design drawings, and additional information may be provided by Paterson for these items for incorporation in the building design packages and associated tender documents. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

Waterproofing System

It is recommended that a waterproofing system be provided for the proposed structures at the subject site. It is expected that insufficient room will be available for exterior backfill, and the foundation wall will be cast as a blind-sided pour against a shoring system and the bedrock surface. It is recommended that the waterproofing system consists of the following:

- ❑ A composite drainage membrane (Delta Terraxx, MiraDrain G100N, or equivalent) should be placed against the shoring system or bedrock face with the geotextile layer facing the shoring system or bedrock face from the finished ground surface to the top of the footing.
- ❑ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant. It is highly recommended that the drainage board rolls be installed horizontally rather than vertically to minimize the number of vertical joints forming between the rolls.
- ❑ A waterproofing membrane should be placed against composite drainage boards between the top of footings and at least 1 m above the long-term groundwater elevation.

- It is recommended that 150 mm diameter PVC sleeves, at least two (2) per wall on each side (to be confirmed at the time of construction), be cast into the foundation wall at the foundation wall/footing interface to allow infiltrating water to flow to the interior perimeter drainage pipe. The sleeves should be punched through the waterproofing membrane, but not through the drainage board. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe network.

The top end lap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and the foundation wall.

Interior Perimeter and Underfloor Drainage

Depending on the type of foundation wall pour time (double sided or blind-sided), an interior or exterior perimeter drainage along with an underfloor drainage system will be required to redirect water from the building's foundation drainage system to the building's sump pit(s) if it will not discharge to an exterior catch basin structure. For preliminary design purposes, it is recommended that the interior perimeter and underfloor drainage pipes should consist of 100 or 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock, placed at approximately 6 m centers. The spacing can be reassessed at the time of construction once the water infiltration through the excavation sidewalls is further reviewed.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves.

The spacing of the underfloor drainage should be confirmed by Paterson at the time of excavation, when water infiltration can be better assessed and once the foundation layout and sump system location have been finalized.

Foundation Backfill (If Applicable)

Where applicable, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Terraxx, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

Foundation backfill material should be compacted in maximum 300 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Podium Deck Waterproofing Tie-In (If Applicable)

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall as depicted in Figure 2 – Podium Deck to Foundation Wall Drainage System Tie-in Detail.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the building should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized structural and architectural drawings for the proposed buildings at the subject site to provide a building-specific waterproofing and drainage design, which includes the above-noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double-sided pours (if applicable), etc.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

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

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden and poor-quality bedrock materials should either be cut back at acceptable slopes or should be retained by a temporary shoring system from the start of the excavation until the structure is backfilled.

Unsupported Excavations

The excavation side slopes in the overburden and poor-quality bedrock, above the groundwater level, and 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 subsurface soil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

Temporary Shoring

Due to the anticipated depth of excavation of the building and the proximity of the proposed building to the site boundaries, temporary shoring may be required to support the overburden soils and poor quality bedrock of the adjacent properties.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the temporary shoring system or soils supported by the system. Any changes to the approved temporary shoring system design should be reported immediately to the owner's structural designer prior to implementation.

The temporary shoring system may consist of a soldier pipe and lagging system which could be cantilevered, anchored or braced.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressure acting on the shoring system may be calculated using the following parameters.

Table 7 – Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight , kN/m3	21
Submerged Unit Weight , kN/m3	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 table.

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.

Bedrock Stabilization

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Horizontal rock anchors and chainlink fencing or shotcrete are anticipated to be required over the upper 2 to 3 m of the vertical bedrock face, which generally consists of lower quality bedrock, in order to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

However, the specific requirements for bedrock stabilization measures should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade, or 300 mm of OPSS Granular A when placed on a bedrock subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

It is generally possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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.

Groundwater Control for Building Construction

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan, to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

Impacts on Neighbouring Properties

Given the relatively shallow bedrock present at and in the vicinity of the subject site, the neighbouring structures are expected to be founded on the bedrock surface. Therefore, no issues are expected with respect to groundwater lowering that would cause damage to adjacent structures surrounding the proposed development.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using 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 into 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.

6.7 Corrosion Potential and Sulphate

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

6.8 Protection of Potentially Expansive Shale Bedrock

Upon being exposed to air and moisture, the shale may decompose into thin flakes along the bedding planes. Previous studies have concluded shales containing pyrite are subject to volume changes upon exposure to air. As a result, the formation of jarosite crystals by aerobic bacteria occurs under certain ambient conditions.

It has been determined that the expansion process does not occur or can be retarded when air (i.e. oxygen) is prevented from contact with the shale and/or the ambient temperature is maintained below 20^{o}C, and/or the shale is confined by pressures in excess of 70 kPa. The latter restriction on the heaving process is probably the major reason why damage to structures has, for the greater part, been confined to slabs-on-grade rather than footings.

The presence of expansive shale may be encountered at the subject site. To reduce the long term deterioration of the shale, exposure of the bedrock bearing surface to oxygen should be kept as low as possible. The bedrock bearing surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air. A 50 to 75 mm thick concrete mud slab, consisting of minimum 15 MPa lean concrete, should be placed on the exposed bedrock bearing surface within a 48 hour period of being exposed. The excavated sides of the exposed bedrock should be sprayed with a bituminous emulsion or shotcrete to seal bedrock from exposure to air and dewatering.

Preventing the dewatering of the shale bedrock will also prevent the rapid deterioration and expansion of the shale bedrock. This can be accomplished by spraying bituminous emulsion as noted above.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ❑ Review preliminary and detailed grading, servicing, landscaping, and structural plan(s) from a geotechnical perspective.
- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.
- ❑ Review of architectural plans pertaining to waterproofing systems, underfloor drainage systems, and waterproofing details for elevator shafts.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ❑ Review and inspection of the installation of the foundation drainage systems.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Observation of driving and re-striking of all pile foundations.
- ❑ 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 and follow-up field density tests to determine the level of compaction achieved.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

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

8.0 Statement of Limitations

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

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

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

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

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
July 17, 2026
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Yashar Ziaeimehr, M.A.Sc., P.Eng.

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❑ Colonnade BridgePort (e-mail copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1900 & 2000 City Park Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE January 17, 2023

FILE NO. PG6552

HOLE NO. BH 1-23

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	73.20
FILL: Brown silty sand, some gravel, trace clay and topsoil (0.30)	AU	1				
FILL: Brown silty clay with sand, some gravel and topsoil	SS	2	83	16	1	72.20
(1.83)	SS	3	92	14	2	71.20
GLACIAL TILL: Brown silty sand with clay, some gravel and shale fragments	SS	4	100	50+		
(3.79)	SS	5	67	50+	3	70.20
BEDROCK: Poor to excellent quality, black shale	RC	1	100	42	4	69.20
	RC	2	100	74	5	68.20
					6	67.20
- vertical fracture from 7.8 to 8.8m depth	RC	3	100	95	7	66.20
	RC	4	100	75	8	65.20
					9	64.20
	RC	5	100	92	10	63.20
	RC	6	100	62	11	62.20
(12.37)					12	61.20
End of Borehole						
(GWL @ 2.39m - Jan. 31, 2023)						

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1900 & 2000 City Park Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE January 17, 2023

FILE NO. PG6552

HOLE NO. BH 2-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
FILL: Brown silty clay wtih sand, some topsoil, trace gravel	0.46	AU	1		
FILL: Brown silty sand, trace gravel and topsoil	1.68	SS	2	100	6
GLACIAL TILL: Compact to dense, brown silty sand with gravel and shale fragments	2.57	SS	3	100	23
		SS	4	91	50+
BEDROCK: Very poor to poor quality, black shale	4.30	RC	1	64	0
		RC	2	95	44
BEDROCK: Good to excellent quality, black shale - vertical fracture from 4.3 to 5.6m depth	11.84	RC	3	100	92
		RC	4	100	86
		RC	5	100	98
		RC	6	100	100
		RC	7	100	100
End of Borehole (GWL @ 5.15m - Jan. 31, 2023)					

DEPTH (m)	ELEV. (m)
0	73.95
1	72.95
2	71.95
3	70.95
4	69.95
5	68.95
6	67.95
7	66.95
8	65.95
9	64.95
10	63.95
11	62.95

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1900 & 2000 City Park Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE January 18, 2023

FILE NO. PG6552

HOLE NO. BH 3-23

SOIL DESCRIPTION	Depth to base of layer (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
TOPSOIL	0.05	AU	1		
FILL: Brown silty clay, some sand, topsoil and gravel	0.91				
GLACIAL TILL: Brown silty sand with gravel and shale fragments	2.11	SS	2	83	21
		SS	3	100	50+
BEDROCK: Very poor quality, black shale	3.00	SS	4	50	50+
		RC	1	100	0
BEDROCK: Good to excellent quality, black shale - vertical fracture from 7.1 to 7.6m depth - vertical fracture from 10.4 to 10.6m depth - vertical fracture from 11.25 to 12.1m depth	12.06	RC	2	100	81
		RC	3	100	92
		RC	4	100	76
		RC	5	100	98
		RC	6	100	80
		RC	7	100	96

End of Borehole

(GWL @ 3.83m - Jan. 31, 2023)

DEPTH (m)	ELEV. (m)
0	73.06
1	72.06
2	71.06
3	70.06
4	69.06
5	68.06
6	67.06
7	66.06
8	65.06
9	64.06
10	63.06
11	62.06
12	61.06

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1900 & 2000 City Park Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE January 18, 2023

FILE NO. PG6552

HOLE NO. BH 4-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
TOPSOIL	0.08				
FILL: Brown silty sand with clay, gravel and rock fragments, some topsoil	1.07	SS	1	62	9
Very stiff, brown **SILTY CLAY**	1.45	SS	2	83	9
GLACIAL TILL: Brown silty sand with gravel and shale fragments	1.68	SS	3	100	50+
BEDROCK: Very poor quality, black shale	3.33	RC	1	46	0
BEDROCK: Fair to excellent quality, black shale - vertical fracture from 6.6 to 6.8m depth - vertical fracture from 7.5 to 8.2m depth - vertical fracture from 8.7 to 9.0m depth	12.04	RC	2	96	67
		RC	3	100	91
		RC	4	100	85
		RC	5	100	90
		RC	6	100	100
		RC	7	100	100

DEPTH (m)	ELEV. (m)
0	74.26
1	73.26
2	72.26
3	71.26
4	70.26
5	69.26
6	68.26
7	67.26
8	66.26
9	65.26
10	64.26
11	63.26
12	62.26

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

End of Borehole

(GWL @ 5.00m - Jan. 31, 2023)

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1900 & 2000 City Park Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE January 19, 2023

FILE NO. PG6552

HOLE NO. BH 5-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	72.56
FILL: Brown silty clay with sand, organics, trace gravel — 0.23		SS	1	56	50+		
GLACIAL TILL: Very dense, brown silty sand with clay and shale fragments — 1.58		SS	2	100	50+	1	71.56
		SS	3	100	50+		
BEDROCK: Poor to fair quality, black shale		RC	1	100	38	2	70.56
- 75mm thick mud seam at 1.8m depth						3	69.56
- 140mm thick mud seam at 3.1m depth		RC	2	100	69	4	68.56
- 10mm thick mud seam at 5.8m depth		RC	3	100	93	5	67.56
- vertical fracture from 6.5 to 6.7m depth						6	66.56
- good to excellent quality by 4.5m depth		RC	4	100	69	7	65.56
- 40mm thick mud seam at 10.8m depth		RC	5	100	95	8	64.56
- 120mm thick mud seam at 10.9m depth						9	63.56
		RC	6	100	92	10	62.56
						11	61.56
		RC	7	100	72	12	60.56
12.27							
End of Borehole							
(GWL @ 3.43m - Jan. 31, 2023)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1900 & 2000 City Park Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

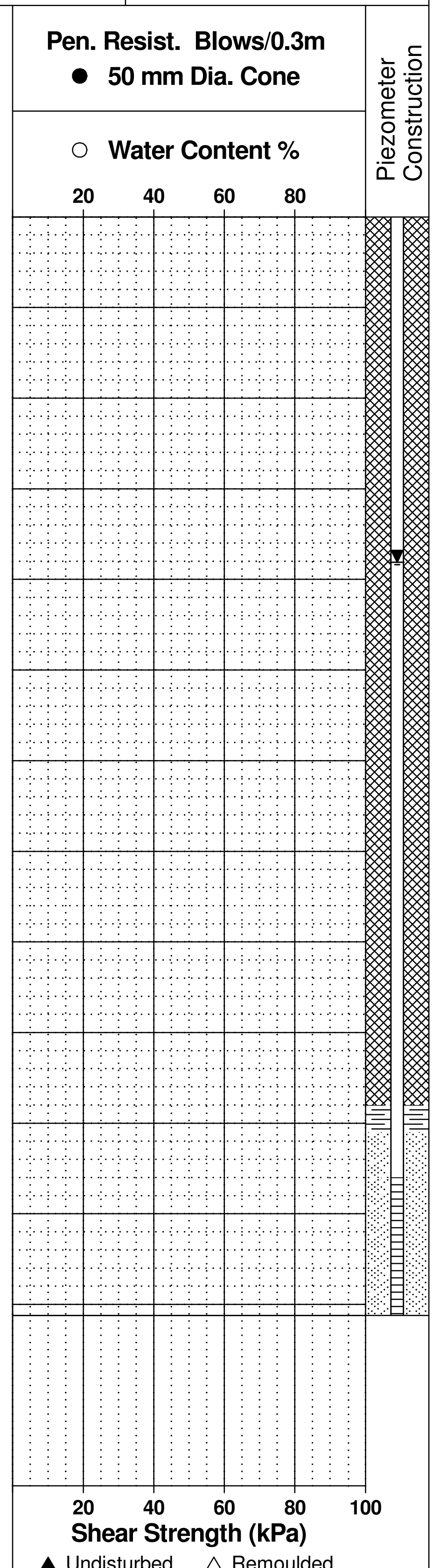
DATE November 3, 2023

FILE NO. PG6552

HOLE NO. BH 7-23

SOIL DESCRIPTION	Strata depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	73.89
Asphaltic concrete	0.05						
FILL: Brown silty sand with gravel and crushed stone		SS	1	50	30		
	1.47	SS	2	8	4	1	72.89
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, trace clay		SS	3	50	22	2	71.89
- grey by 2.6m depth	2.84	SS	4	100	40		
BEDROCK: Poor quality, black shale		RC	1	100	29	3	70.89
- good to excellent quality by 3.15m depth		RC	2	100	75	4	69.89
		RC	3	100	80	5	68.89
- 25mm thick mud seam at 6.2m depth						6	67.89
		RC	4	100	90	7	66.89
		RC	5	100	98	8	65.89
						9	64.89
		RC	6	100	98	10	63.89
		RC	7	100	91	11	62.89
	12.12					12	61.89
End of Borehole							

(GWL @ 3.81m - Nov. 13, 2023)



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

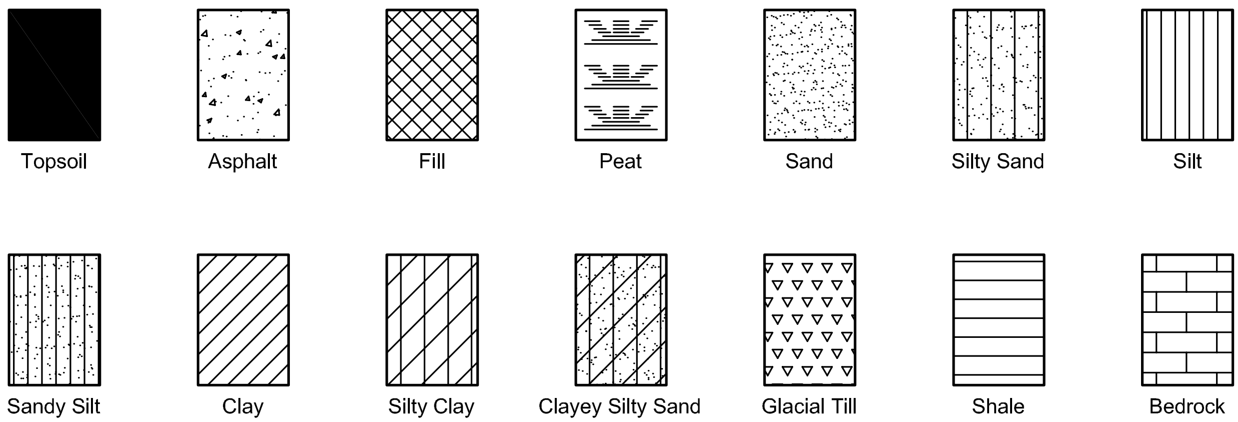
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

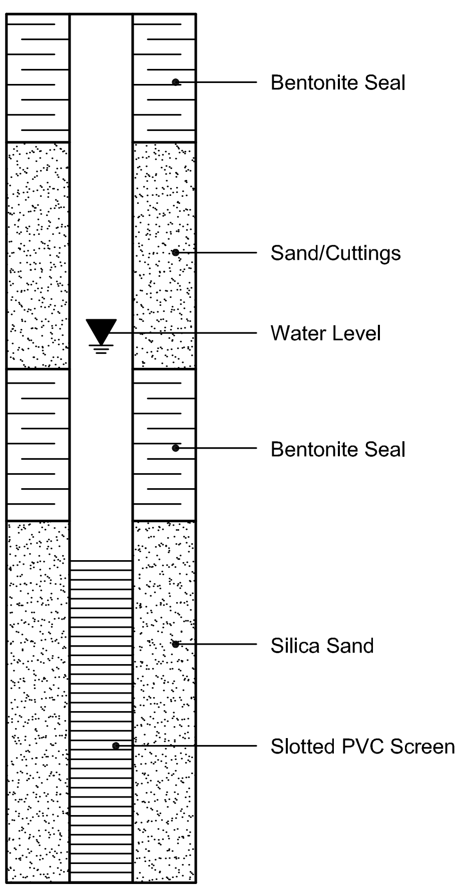
SYMBOLS AND TERMS (continued)

STRATA PLOT

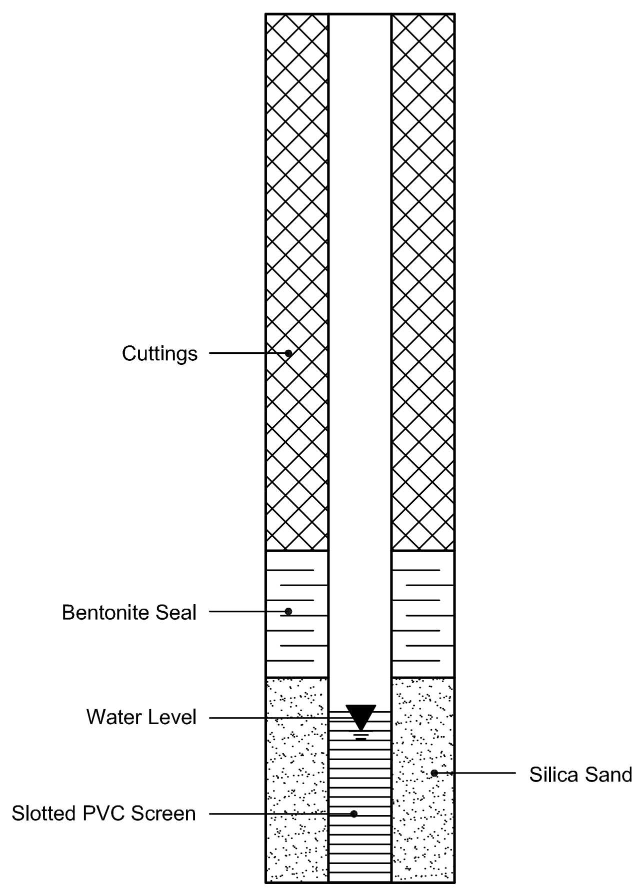


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



REFERENCE No.: T020297-A1

ENCLOSURE No.: 1

INSPEC-SOL

BOREHOLE No.: BH-01

ELEVATION: 73.19 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Bentall Real Estate Services

PROJECT: Geotechnical Investigation, Proposed Office Building

LOCATION: City Park Drive. Ottawa, Ontario

DESCRIBED BY: A. Solomon CHECKED BY: H. Krzywicki

DATE (START): July 30, 2007 DATE (FINISH): July 30, 2007

LEGEND

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

SCALE		STRATIGRAPHY		SAMPLE DATA				SCALE FOR TEST RESULTS
Depth BGS	Elevation (m)	Stratigraphy	DESCRIPTION OF SOIL AND BEDROCK	State / Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD	10 20 30 40 50 60 70 80 90 (50kPa 100kPa 150kPa 200kPa)
meters	73.19		GROUND SURFACE		%	ppm	N	
	73.04		TOPSOIL/ROOTMAT					
			SAND SILT FILL, compact, dark brown, some organic intrusions, occasional sand and clay layers, moist	SS1	46		15	●
0.5								
1.0				SS2	22		12	●
1.5				SS3	56		10	●
2.0	71.23		SHALE TILL, dense, black, frequent shale pieces, clayey, moist	SS4	59		50+	●
				RC5	100		0	
2.5	70.68		WEATHERED SHALE, thinly bedded	SS6	60		50+	●
3.0				SS7	100		50+	●
	69.94		SHALE BEDROCK -Billings formation -generally sound -occasional soft rock seams					
3.5								
4.0				RC8	90		28	
4.5	68.64		END OF BOREHOLE					
5.0								
5.5								
6.0								

NOTES:

BOREHOLE LOG T020297-A1-LOG1.GPJ INSPEC_SOL.GDT 8/23/07

INSPEC-SOL

BOREHOLE No.: BH-02

ELEVATION: 73.62 m

BOREHOLE LOG

CLIENT: Bentall Real Estate Services

PROJECT: Geotechnical Investigation, Proposed Office Building

LOCATION: City Park Drive. Ottawa, Ontario

DESCRIBED BY: A. Solomon CHECKED BY: H. Krzywicki

DATE (START): July 31, 2007 DATE (FINISH): July 31, 2007

LEGEND

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- o Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	Type and Number	Recovery (%)	Organic Vapour ppm or %LEL (ppm)	Penetration Index / RQD (N)
	73.62	GROUND SURFACE				
	73.59	TOPSOIL/ROOTMAT				
		SANDY SILT FILL, compact, dark brown, some gravel and shale pieces, moist	SS1	71		18
			SS2	56		16
		-becoming clayey by 1.5m depth	SS3	56		13
	71.64	SHALE TILL, dense, black, frequent shale pieces, moist	SS4	89		50+
	71.11	WEATHERED SHALE, thinly bedded				
	70.70	SHALE BEDROCK -Billings formation -generally sound	SS5	88		50+
		-occasional soft rock seams	RC6	88		37
	68.87	END OF BOREHOLE				

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0

SCALE FOR TEST RESULTS: 50kPa, 100kPa, 150kPa, 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:

REFERENCE No.: T020297-A1

ENCLOSURE No.: 3

INSPEC-SOL

BOREHOLE No.: BH-03

ELEVATION: 72.33 m

BOREHOLE LOG

CLIENT: Bentall Real Estate Services

PROJECT: Geotechnical Investigation, Proposed Office Building

LOCATION: City Park Drive. Ottawa, Ontario

DESCRIBED BY: A. Solomon CHECKED BY: H. Krzywicki

DATE (START): July 31, 2007 DATE (FINISH): July 31, 2007

LEGEND

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- o Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

SCALE FOR TEST RESULTS: 50kPa, 100kPa, 150kPa, 200kPa; 10, 20, 30, 40, 50, 60, 70, 80, 90

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	Type and Number	Recovery (%)	Organic Vapour ppm or %LEL (ppm)	Penetration Index / RQD (N)
	72.33	GROUND SURFACE				
		TOPSOIL/ROOTMAT				
	72.15	SANDY SILT FILL, compact, dark brown, moist -Becoming clayey with occasional gravel by 0.5m depth	SS1	50		7
			SS2	42		7
	71.11	SHALE TILL, dense, black, frequent shale pieces, moist	SS3	71		50+
	70.76	SHALE BEDROCK -Billings formation -highly weathered layer at surface -generally sound -occasional soft rock seams	RC4	50		0
			RC5	84		15
	67.66	END OF BOREHOLE				

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0

NOTES:

BOREHOLE LOG T020297-A1-LOG1.GPJ INSPEC_SOL.GDT 8/23/07

REFERENCE No.: T020297-A1 ENCLOSURE No.: 4

INSPEC-SOL

BOREHOLE No.: BH-04
ELEVATION: 72.41 m

BOREHOLE LOG

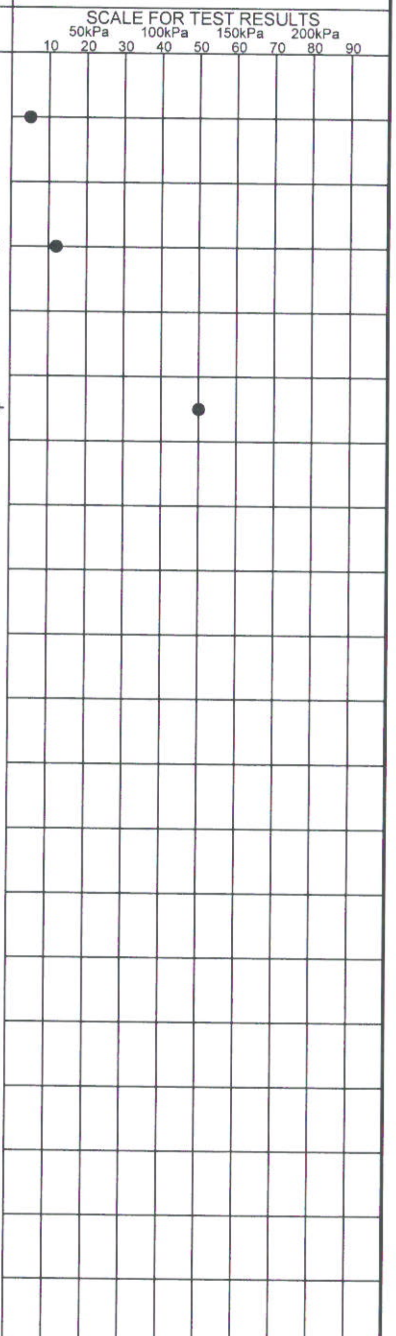
Page: 1 of 1

CLIENT: Bentall Real Estate Services
PROJECT: Geotechnical Investigation, Proposed Office Building
LOCATION: City Park Drive. Ottawa, Ontario
DESCRIBED BY: A. Solomon CHECKED BY: H. Krzywicki
DATE (START): July 31, 2007 DATE (FINISH): July 31, 2007

LEGEND

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- o Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (meters)	Elevation (m)	Description of Soil and Bedrock	Type and Number	Recovery (%)	Organic Vapour ppm or %LEL (ppm)	Penetration Index / RQD (N)
	72.41	GROUND SURFACE				
	72.38	TOPSOIL/ROOTMAT				
		SANDY SILT FILL, compact, dark brown, moist	SS1	33		5
1.0	71.50	SHALE TILL, frequent shale pieces, dense, black, moist	SS2	63		12
			SS3	77		50+
2.0	70.43	SHALE BEDROCK -Billings formation -Highly weathered layer at surface -Generally sound -Occasional soft rock seams	RC4	70		39
3.5	68.90	END OF BOREHOLE				



Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0

NOTES:

BOREHOLE LOG T020297-A1-LOG1.GPJ INSPEC_SOL.GDT 8/23/07

REFERENCE No.: T020297-A1

ENCLOSURE No.: 5

INSPEC·SOL

BOREHOLE No.: BH-05

ELEVATION: 72.19 m

BOREHOLE LOG

Page: 1 of 1

CLIENT: Bentall Real Estate Services

PROJECT: Geotechnical Investigation, Proposed Office Building

LOCATION: City Park Drive. Ottawa, Ontario

DESCRIBED BY: A. Solomon CHECKED BY: H. Krzywicki

DATE (START): July 31, 2007 DATE (FINISH): July 31, 2007

LEGEND

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

SCALE: Depth BGS	SCALE: Elevation (m)	STRATIGRAPHY: DESCRIPTION OF SOIL AND BEDROCK	SAMPLE DATA: Type and Number	Recovery	Organic Vapour ppm or %LEL	Penetration Index / RQD
meters	72.19	GROUND SURFACE		%	ppm	N
		TOPSOIL/ROOTMAT				
	71.99	SILTY CLAY FILL, stiff, dark brown, some organics, moist	SS1	67		6
0.5	71.61	SHALE TILL, dense, black, frequent shale pieces, moist				
1.0	71.25	WEATHERED SHALE, thinly bedded	SS2	76		50+
1.5	70.77	SHALE BEDROCK -Billings Formation -generally sound				
2.0		-occasional soft rock seams	RC3	85		18
2.5						
3.0	69.24	END OF BOREHOLE				
3.5						
4.0						
4.5						
5.0						
5.5						
6.0						

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 56641

Report Date: 25-Jan-2023

Order Date: 19-Jan-2023

Project Description: PG6552

	Client ID:	BH2-23 SS3 BOT	-	-	-
	Sample Date:	17-Jan-23 09:00	-	-	-
	Sample ID:	2303411-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	89.1	-	-	-
General Inorganics					
pH	0.05 pH Units	7.08	-	-	-
Resistivity	0.10 Ohm.m	97.4	-	-	-
Anions					
Chloride	10 ug/g dry	16	-	-	-
Sulphate	10 ug/g dry	38	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

DRAWING PG6552-2 – TEST HOLE LOCATION PLAN

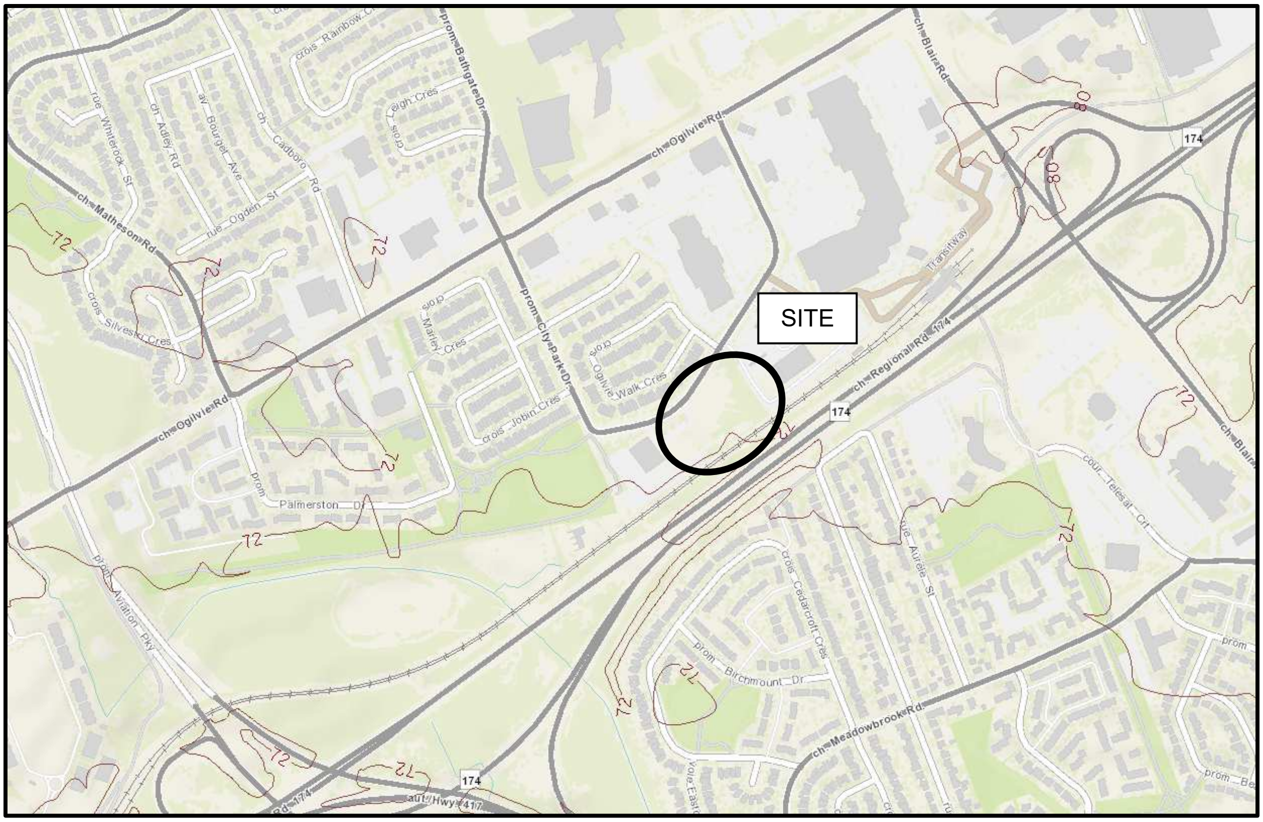
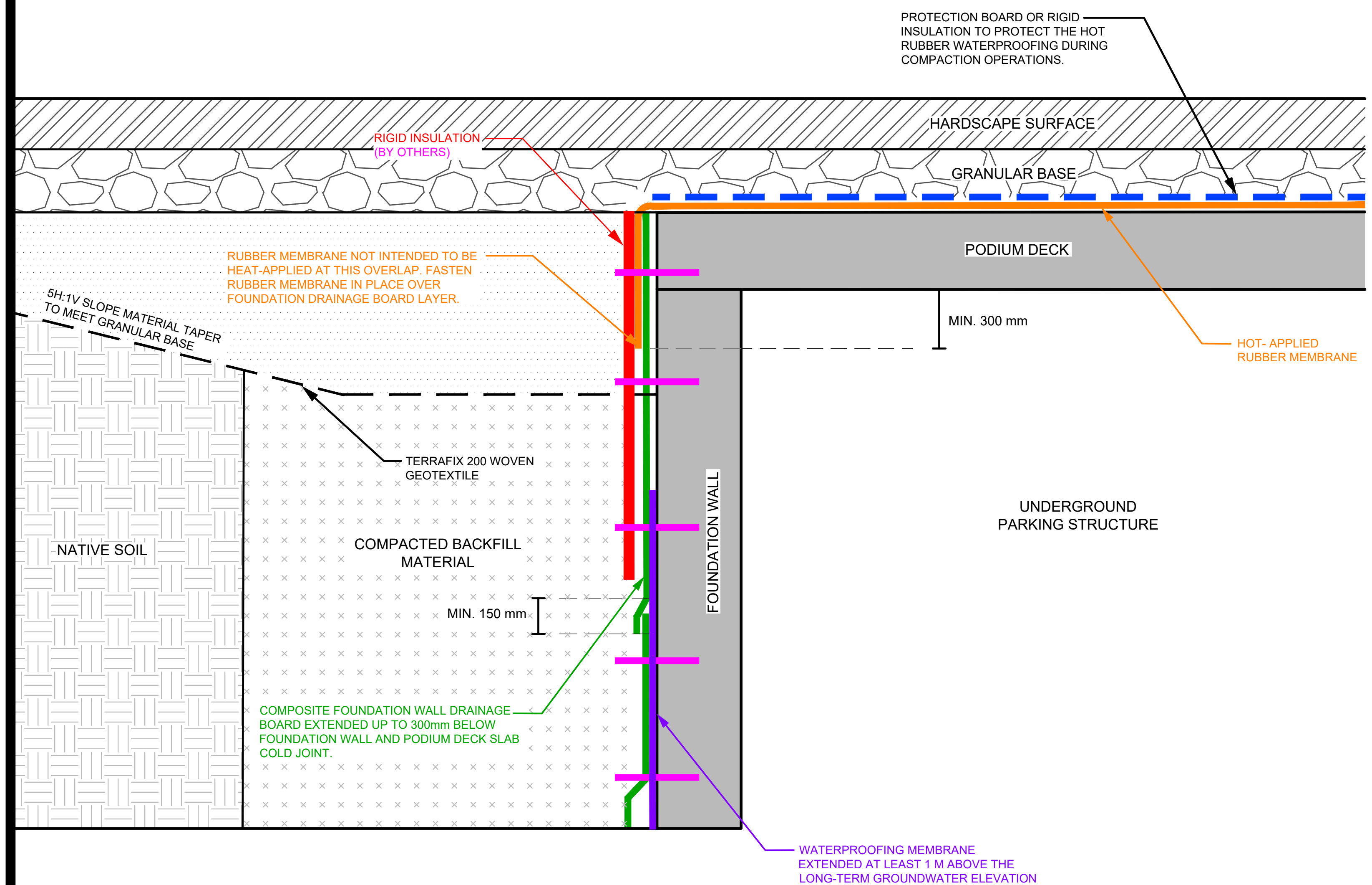


FIGURE 1

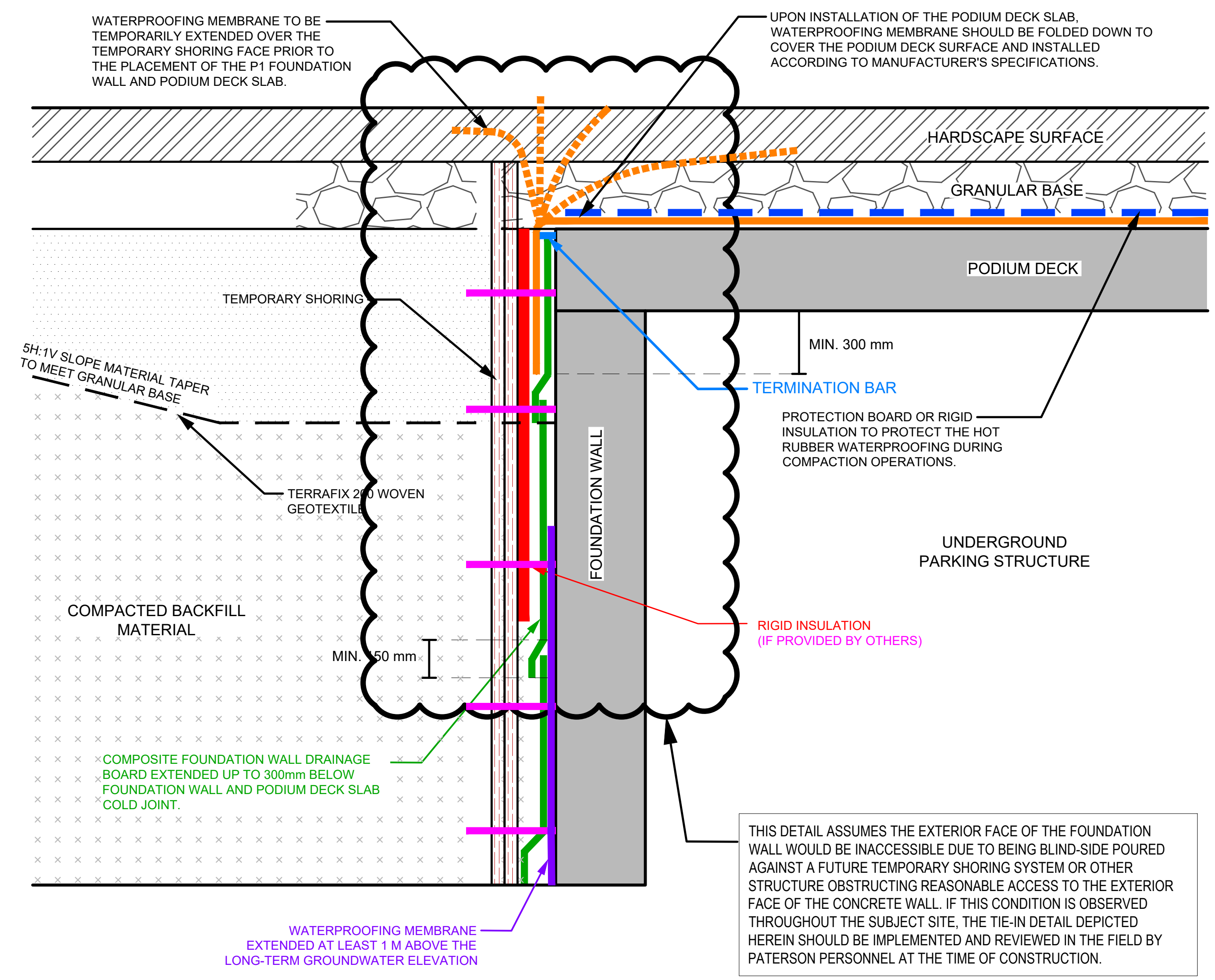
KEY PLAN

PATERSON GROUP

OPTION A - DOUBLE-SIDE POURED TOP OF FOUNDATION WALL



OPTION B - BLIND-SIDE POURED TOP OF FOUNDATION WALL



NOTES:

THE ABOVE DETAIL FOR HOT RUBBER AND DRAINAGE BOARD OVERLAP IS APPLICABLE TO ALL EDGE-PORTIONS OF THE PODIUM DECK AND/OR SUSPENDED GROUND FLOOR SLAB STRUCTURE.

APPLICABILITY THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS

WHERE THE GRADING SURFACE TERMINATES AGAINST THE BUILDING FACE AND PAVEMENT STRUCTURE IS NOT LOCATED ABOVE THE EDGE OF THE FOUNDATION WALL AND PODIUM DECK SLAB AS DEPICTED HEREIN, IT IS RECOMMENDED TO PROVIDE A SUITABLE TERMINATION BAR TO SEAL THE TOP ENDLAP OF THE HOT-APPLIED RUBBER MEMBRANE LAYER TO THE VERTICAL FACE OF THE STRUCTURE. THIS WOULD BE REQUIRED TO MITIGATE THE POTENTIAL FOR THE MIGRATION OF WATER BEHIND THE RUBBER MEMBRANE.

ALL PORTIONS OF THE ABOVE-NOTED DETAIL (INSULATION OF FOUNDATION DRAINAGE BOARD, TERMINATION BAR, HOT-RUBBER MEMBRANE OVER SLAB, FOUNDATION WALL CONSTRUCTION JOINT AND OVERLAPPING/SHINGLING OF DRAINAGE BOARD) SHOULD BE REVIEWED AT THE TIME OF CONSTRUCTION BY PATERSON PERSONNEL.

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

NO.	REVISIONS	DATE	INITIAL

COLONNADE BRIDGEPORT
GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE BUILDINGS
2000 CITY PARK DRIVE
OTTAWA, ONTARIO

Title:
PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

Scale:	N.T.S	Date:	07/2026
Drawn by:	GK	Report No.:	PG6552-2
Checked by:	YZ	Dwg. No.:	FIGURE 2
Approved by:	FA	Revision No.:	

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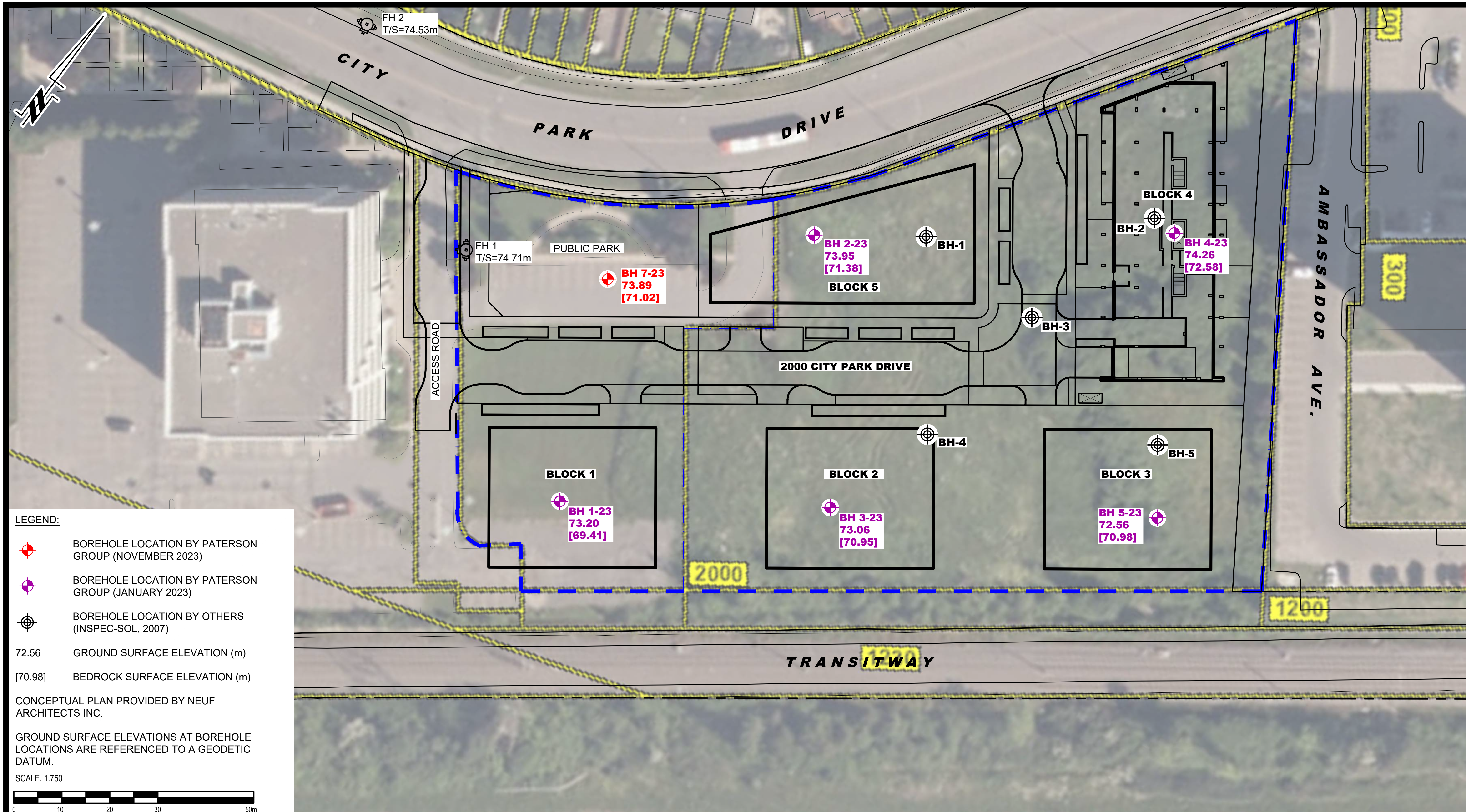

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DD/MM/YYYY	INITIAL

COLONNADE BRIDGEPORT
GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE BUILDINGS
2000 CITY PARK DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:750	Date:	07/2026
Drawn by:	GK	Report No.:	PG6552-2
Checked by:	YZ	Dwg. No.:	PG6552-2
Approved by:	FA	Revision No.:	

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