

Geotechnical Investigation

Proposed Multi-Use High Rise Complex

75 Breezehill Avenue North
Ottawa, Ontario

Prepared for 2729685 Ontario Inc. c/o Fotenn

Report PG6622-1 Revision 1 dated February 26, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by 2729685 Ontario Inc. c/o Fotenn to conduct a geotechnical investigation for the proposed multi-use high rise complex to be located at 75 Breezehill Avenue North in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was 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 development at 75 Breezehill Avenue North will consist of two multi-use high rise buildings with two levels of shared underground parking level occupying the majority of the footprint of the subject site. Building A is proposed to include 35 storeys while Building B will host 30 storeys. It is understood that the proposed Building B will include approximately 472 m² of commercial space. Associated access lanes, landscaped areas and a park are also anticipated as a part of the development. It is expected that the proposed buildings will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was conducted on April 18 and 19, 2023, and consisted of advancing four boreholes to a maximum depth of 13.65 m below the existing ground surface. The borehole locations were distributed across the site in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on Drawing PG6622-1 – 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, while the boreholes in the existing underground parking were completed using a portable drill rig operated by a two-person crew. The testing procedure consisted of advancing to the required depths at the selected locations and sampling the overburden. All fieldwork was reviewed in the field by Paterson personnel under the direction of a senior engineer from the geotechnical division.

Sampling and In Situ Testing

Soil samples from the drilling program were recovered with a 50 mm diameter split-spoon sampler or from the auger flights. The samples were classified on site, placed in sealed plastic bags and transported to Paterson's laboratory for further review. The depths at which the split-spoon samples were recovered from the boreholes are presented as SS on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery 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.

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils using a vane apparatus.

The thickness of the overburden was evaluated by a dynamic cone penetration test (DCPT) at boreholes BH 2-23 and BH 3-23.

The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Rock samples were recovered using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality. The subsurface conditions observed at the boreholes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The borehole locations and ground surface elevation at each borehole location were surveyed by Paterson at the time of the field investigation using a high precision GPS unit. The borehole locations are referenced to a geodetic datum. The borehole locations and ground surface elevation at the boreholes are presented on Drawing PG6622-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging. All soil samples were submitted for moisture content testing. All samples will be stored in the laboratory for a period of one month after the issuance of this report 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 of sulphate attacks against subsurface concrete structures by Paterson. The samples were submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a two-storey building with multiple commercial units and associated at grade parking and landscaped areas. The ground surface across the subject site is relatively flat with an approximate geodetic elevation of 64 m and is at grade with Breezehill Avenue, Laurel Street and the neighboring properties.

The subject site is bordered by a two-storey commercial building to the north, by Laurel Street to the south, by commercial/residential buildings and Breezehill Avenue to the west and by Light Rail Transit (LRT) tracks to the east.

4.2 Subsurface Profile

Building A

Overburden

The subsurface profile encountered at the borehole locations BH 1-23 and BH 2-23 generally consisted of a fill layer overlying hard to very stiff, brown silty clay transitioning to a very stiff to stiff grey silty clay, underlain by inferred glacial till extending to the bedrock surface. The fill material consisted of brown silty sand to silty clay with gravel, crushed stone and trace asphalt and was observed to extend to depths between 1.1 and 1.8 m below the existing ground surface.

Bedrock

Practical refusal to DCPT was encountered on inferred bedrock at a depth of 13.51 m below the existing ground surface at borehole BH 2-23. Based on available geological mapping, the subject site consists of interbedded shale and limestone of the Verulam formation and an approximate drift thickness of 0 to 10 m.

Building B

Overburden

The subsurface profile encountered at the borehole locations BH 3-23 and BH 4-23 generally consisted of a fill layer overlying hard to very stiff, brown silty clay transitioning to a very stiff to stiff grey silty clay. The silty clay was followed by very stiff to stiff and compact to dense glacial till, extending to the bedrock surface.

The fill material consisted of brown silty sand to silty clay with gravel, crushed stone and trace organics and was observed to extend to depths between 1.1 and 1.8 m below the existing ground surface.

Bedrock

Practical refusal to DCPT was encountered on inferred bedrock at a depth of 10.34 m below the existing ground surface at borehole BH 3-23. Practical refusal to augering was encountered on the bedrock surface at a depth of 9.73 m below the existing ground surface at BH 4-23. Based on available geological mapping, the bedrock in this area consists of interbedded limestone and shale of the Verulam Formation with an anticipated overburden drift thickness of 0 to 10 m.

Bedrock was encountered underlying the glacial till at borehole 4-23 at a depth of 9.83 m and consisted of good to excellent quality limestone. Reference should be made to the Soil Profile and Test Data sheets in the attachments for specific details of the soil profiles encountered at each borehole location.

4.3 Groundwater

Groundwater levels were measured on April 26, 2023 at the installed monitoring well locations. The groundwater level measurements are presented in Table 1 below.

Table 1 – Summary of Groundwater Levels					
Borehole Number	Observation Method	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
			Depth (m)	Elevation (m)	
BH 1-23	Monitoring Well	63.95	3.30	60.65	April 26, 2023
BH 2-23	Monitoring Well	64.08	3.97	60.11	
BH 3-23	Monitoring Well	63.76	3.00	60.76	
BH 4-23	Monitoring Well	63.86	4.00	59.86	

Groundwater levels can be influenced by surface water infiltrating the backfilled boreholes and perched water within the borehole backfill material. Long-term groundwater levels can also be estimated based on the observed colour, consistency and moisture levels of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 4 to 5 m below the existing ground surface. The groundwater levels are noted on the applicable Soil Profile and Test Data Sheets presented in Appendix 1. It should also be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended the proposed multi-storey buildings are founded on a raft foundation placed on an undisturbed, very stiff to stiff grey silty clay and/or compact to dense glacial till bearing surface.

Alternatively, For Building B, conventional spread footings may be extended to the bedrock surface via a lean concrete trench, placed over a clean, surface sounded, limestone bedrock surface.

End bearing piles are also recommended as a foundation option if the expected loads of the proposed buildings exceed the bearing resistance values provided in the previous options.

Due to the presence of a silty clay deposit within the subject site, any settlement-sensitive structures placed on a silty clay subgrade within the proposed development will be subjected to a permissible grade raise restriction.

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

5.2 Site Grading and Preparation

Stripping Depth

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

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

Vibration Considerations

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

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of a shoring system with soldier piles or sheet piling will require these pieces of equipment. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited. Two parameters determine the recommended vibration limit, 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). These guidelines are for current construction standards. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill used for grading beneath the building footprint, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It 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 building area should be compacted to at least 99% of its 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. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids.

If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Protection of Subgrade (Raft Foundation)

Since the subgrade material for the proposed foundations is expected to partially consist of very stiff to stiff silty clay, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the silty clay subgrade shortly after the completion of the excavation.

The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic or workers and equipment and to provide a more consistent bearing surface transition with the glacial till. The final excavation to the raft bearing surface level and the placing of the mud slab should be completed in smaller sections to avoid exposing large areas of the silty clay to potential disturbances due to drying.

5.3 Foundation Design

Raft Foundation

Based on the expected loads from the proposed structure, a raft foundation bearing on the undisturbed, very stiff to stiff, grey silty clay and/or compact to dense glacial till bearing surface may be considered for foundation support for the proposed building or a combination of both bearing mediums. For design purposes, it was assumed that the base of the raft foundation would be located at an approximate depth of 7 to 8 m since it would be provided with two level of underground parking.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

For the raft slab foundation, a bearing resistance value at SLS (contact pressure) of **275 kPa** will be considered acceptable for a raft supported on the undisturbed, very stiff silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **420 kPa**. For this case, the modulus of subgrade reaction was calculated to be **11 MPa/m** for a contact pressure of **275 kPa**.

For the raft slab foundation, a bearing resistance value at SLS (contact pressure) of **350 kPa** will be considered acceptable for a raft supported on the undisturbed, Dense Glacial till. The factored bearing resistance (contact pressure) at ULS can be taken as **500 kPa**. For this case, the modulus of subgrade reaction was calculated to be **15 MPa/m** for a contact pressure of **350 kPa**.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

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

Where the raft foundation is partly placed over silty clay and partly over glacial till, a minimum ground taper of 5H:1V is recommended to be placed at the transition zone. The silty clay should be subexcavated at a 5H:1V and replaced with OPSS Granular A or Granular B Type II and compacted to a 98% of the material's SPMDD. This will allow the foundation to smoothly transition from a silty clay bearing medium to a glacial till bearing medium. It is suggested that construction joints be considered within this area to withstand differential settlements, if occurred.

Conventional Foundation

Conventional spread footings placed on a clean, surface sounded limestone bedrock bearing surface and can be designed using a bearing resistance value of **1,500 kPa** at ULS incorporating a geotechnical resistance factor of 0.5.

Conventional Shallow Foundations (Auxiliary Structures)

If applicable, conventional spread footing bearing resistance values may be considered for isolated piers and other lightly loaded ancillary structures.

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over a hard to very stiff, brown silty clay bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

Settlement

The above-noted bearing resistance values at SLS for soil bearing surfaces will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

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.

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.

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.

Above the groundwater level, adequate lateral support is provided to silty clay and glacial till bearing surfaces when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:1V passes only through in situ soil or engineered fill.

Adequate lateral support is provided to bedrock bearing medium when a plane extending down and out from the bottom edges of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

A heavily fractured, weathered bedrock and/or overburden bearing medium will require a lateral support zone of 1H:1V (or flatter).

End Bearing Driven Piled Foundation

As an alternative to a raft foundation, or if the proposed raft bearing capacity is not sufficient to withstand the imposed structural loads, then consideration could be given to using concrete filled, steel pipe piles driven to refusal on the bedrock surface.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 2 below. 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 using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring.

Table 2 – Pile Foundation Design Data					
Outside Pile Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/12mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	910	1090	10	29
245	11	1050	1260	10	35
245	13	1250	1500	10	41

As a minimum, the pipe piles should be equipped with a base plate having a thickness of at least 20 mm to reduce potential damage to the pile tip during driving. Provision should be made for restriking all of the piles at least once, 48 hours after the initial driving, to confirm the design set and/or the permanence of the set, and to check for upward displacement due to driving adjacent piles.

It is recommended that a pile load test or dynamic monitoring and capacity testing be carried out at an early stage during the piling operations to verify the transferred energy from the piledriving equipment and determine the load carrying capacity of the piles. The recommended number of tests is dependent on the number of piles and pile sizes; as a guideline a minimum of 2 tests per pile size should be carried out. It is also recommended that the tested pile locations be spread out across the proposed building footprints.

The post construction settlement of structural elements which derive their support from piles bearing on bedrock should be negligible.

If piles are to be left exposed during winter months, some form of frost protection will be required to prevent frost adhesion and jacking of the piles. Further guidelines can be provided on these measures at the time of construction, if required.

Hybrid Foundation Design

A hybrid foundation design may be used where the underground parkade would be founded over a raft slab while the buildings can be supported by end bearing piles. A construction joint is expected to be provided between the two foundation systems. The bearing resistance values provided in the above sections would apply to each foundation system.

Permissible Grade Raise Restriction

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **2.0 m** is recommended for perimeter grading around foundations placed over a silty clay subgrade at the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. If the proposed footings will be founded within 3 m of the bedrock surface, a seismic site Class A or B may be achievable provided the site-specific shear wave velocity test is 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 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious materials within the footprint of the proposed building, a soil subgrade approved by Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab construction.

The recommended pavement structures noted in Subsection 5.7 will be applicable where the basement level underlying foundation support consists of piled foundations or conventional spread footings. The basement slab would be considered as a slab-on-grade where foundation support consists of piled foundations or conventional spread footings.

For buildings of slab-on-grade construction, it is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone. If storage or other uses of the lower level involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm of clear crushed stone.

Any soft areas should be removed and backfilled with appropriate backfill material such as OPSS Granular B Type II, with a maximum particle size of 50 mm. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone for a basement slab.

Where a raft slab is utilized, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD. An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lowest basement floor. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. This is discussed further in Section 6.1 of this report.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the proposed structure's underground parking and/or basement walls. 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 dry unit weight of 20 kN/m³.

Where undrained conditions are anticipated, the applicable effective unit weight of the retained soil can be designed with 13 kN/m³. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

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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 $K_o \cdot q$ 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 $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ 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.

5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the underground parking level of the proposed buildings 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 3 below:

Table 3 – 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 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

The flexible pavement structure presented in Table 4, presented below, should be used for access lanes and heavy loading parking areas.

Table 4 – 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	

Pavement Structure Over Podium Deck Area and Raft Foundations

It is anticipated that the podium deck structure will be provided for landscaping or to accommodate car only parking areas, access lanes, fire truck lanes and loading areas. Based on the concrete slab subgrade for this area and/or over basements located over a raft slab, the pavement structure indicated in the following tables may be considered for design purposes:

Table 5 - Recommended Pavement Structure - Car-Only Parking Areas (Podium Deck and Raft Slab)	
Thickness (mm)	Material Description
50	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
200**	Base - OPSS Granular A Crushed Stone
See Below*	Thermal Break* - Rigid insulation (See Paragraph Below)
n/a	Waterproofing Membrane and Protection Board
SUBGRADE – Reinforced Concrete Podium Deck or Raft Slab *If specified by others, not required from a geotechnical perspective. Also not required in basements over a raft slab. **Thickness is dependent on grade of insulation as noted in paragraphs below.	

Table 6 - Recommended Pavement Structure – Access Lane, Fire Truck Lane, Ramp and Heavy Truck Parking Areas (Podium Deck and Raft Slab)	
Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Wear Course - HL-8 or Superpave 19.0 Asphaltic Concrete
300**	Base - OPSS Granular A Crushed Stone
See Below*	Thermal Break* - Rigid insulation (See Paragraph Below)
n/a	Waterproofing Membrane and Protection Board
SUBGRADE – Reinforced Concrete Podium Deck or Raft Slab * If specified by others, not required from a geotechnical perspective. Also not required in basements over a raft slab. **Thickness is dependent on grade of insulation as noted in paragraphs below.	

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section. For this transition, a 5H:1V frost taper is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 600 mm below the top of the podium slab a minimum of 1.5 m horizontally from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified by others to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables.

The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 100 (HI-100), High-Load 60 (HI-60) or High Load (HI-40) extruded polystyrene. The pavement structures base layer thickness in Table 8 and Table 9 may be reduced by 25 mm if HI-100 is considered for this project. It should be noted that Styrofoam rigid insulation is not considered suitable for this application.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level, and the subgrade surface should be crowned to promote water flow to drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

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 buildings (and prior to tender) to review and provide supplemental information for the buildings 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.

Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed structures. 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 groundwater suppression system consist of the following:

- ☐ A waterproofing membrane should be placed against the shoring system between underside of footings and 3 m below existing ground surface. The height of the waterproofing layer should be confirmed on a per-building basis, however, is expected to vary between 3 and 4 m below existing ground surface. Where the membrane will extend against the shoring system, it is recommended to consist of a membrane with a bentonite-lined face for being placed against the shoring system. The membrane is recommended to overlap below the overlying perimeter foundation footprint by a minimum of 1 m inwards towards the building footprint and from the face of the overlying foundation. This will allow construction to proceed without imposing groundwater lowering within the surrounding area of the proposed buildings in the short and long term conditions.
- ☐ A composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the HDPE face of the waterproofing membrane with the geotextile layer facing the waterproofing layer from 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.
- ❑ It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer. 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 system.

The top endlap 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 foundation wall.

Interior Perimeter and Underfloor Drainage

An interior perimeter and 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.

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 has been finalized.

Foundation Backfill

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 Drain 6000, 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, sheepfoot roller for soil fill).

Podium Deck Waterproofing Tie-In

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 and as depicted in Figure 2 – Podium Deck to Foundation Wall Transition Detail.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings 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.

Foundation Raft Slab Construction Joints

If applicable, it is anticipated the raft slab will be poured in several pour segments. For the construction joint at each pour, a PVC water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a PVC water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized structural and architectural drawings for each building 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 required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover alone, or a combination of soil cover (minimum 0.6 m) in conjunction with foundation insulation should be provided in this regard.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings or footings near garage bay doors frequently opened in freezing conditions, should be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 600 mm m of soil cover, in conjunction with foundation insulation should be provided. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided for these areas.

6.3 Excavation Side Slopes

The side slopes of the excavations in soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Given the proximity of the underground parking levels to the property lines, it is expected that a temporary shoring will be required to support the excavation for this proposed development.

Temporary Side Slopes (Unsupported Excavations)

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a 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 maintain safe working distance from the excavation sides.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff if shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction. It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

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.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring is anticipated to be required to support the overburden soils for the underground parking levels. 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.

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. Inspections and approval of the temporary system will also be the responsibility of the designer. The geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system.

The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system.

Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 7.

Table 7 – Soil Parameters for Calculating Earth Pressures Acting on Shoring System	
Parameter	Value
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.

Excavation Base Stability (If Applicable)

The base of supported excavations can fail by three general modes:

- ❑ Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,
- ❑ Piping from water seepage through granular soils, and
- ❑ Heave of layered soils due to water pressures confined by intervening low permeability soils.

The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems. The factor of safety with respect to base heave, FS_b , is:

$$FS_b = N_b s_u / \sigma_z$$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.

s_u - undrained shear strength of the soil below the base level

σ_z - total overburden and surcharge pressures at the bottom of the excavation

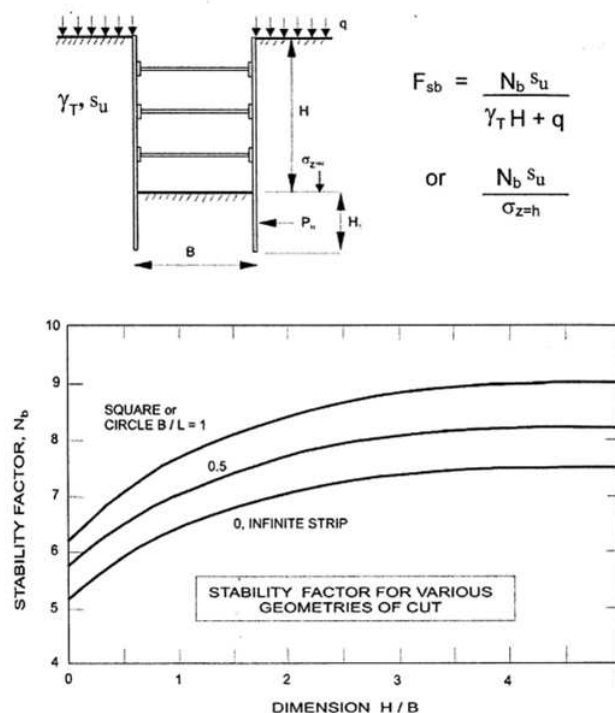


Figure 1 – Stability Factor for Various Geometries of Cut

In the case of stiff clays or compact sands, a factor of safety of 2 is recommended for base stability.

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.

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of grey silty clay. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 98% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 98% of its SPMDD.

It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated fill, such as the grey silty clay, will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

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 minimize 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.

Clay Seals

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches, where applicable. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub bedding and cover material.

The barriers should consist of relatively dry and compatible brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Due to the relatively impervious nature of the overlying silty clay within the upper portion of the soil profile, it is anticipated that groundwater infiltration into the excavations should be low and 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

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum of 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two 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.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which breaches the building's perimeter groundwater infiltration control system will be directed to the sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be very low to negligible. A more accurate estimate of groundwater flow can be provided at the time of construction, once the pressure relief chamber valve is closed and full hydrostatic pressure is applied to the structure.

Impact on Neighboring Properties

It is understood that between two levels of underground parking are planned for the proposed buildings. Based on the existing groundwater level and considering that the proposed buildings will be subject to the water suppression system provided herein, long-term groundwater lowering will be minimal and will take place within a limited range of the proposed buildings.

Based on the proximity of neighboring buildings and the minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

The subsoil conditions at this site mostly 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 where excavations are completed in proximity to existing structures which may be adversely affected due to the freezing conditions.

In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil.

Provisions should be made in the contract documents to protect the walls of the excavations from freezing, if and where applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and/or glycol lines and tarpaulins or other suitable means. 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 foundation is protected with sufficient soil cover to prevent freezing at founding level.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and/or glycol lines and tarpaulins or other suitable means. 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 foundation is protected with sufficient soil cover to prevent freezing at founding level. Trench excavations, foundation construction and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed during freezing conditions. Additional information could be provided, if required.

Under winter conditions, if snow and ice is present within imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Considerations

It is understood the proposed buildings will include two to six levels of underground parking and the structures will be founded on foundation located at a minimum of 7 m below finished grade. Given the depth of foundations proposed for the structures, it is expected that the support of the foundations derives from soil located below the depth that dewatering by tree roots. Therefore, foundation distress due to potential moisture depletion caused by trees is not expected to occur at the subject site. Since the structures are not anticipated to be founded upon silty clay soils affected by the depth of root penetration, City approved trees within the subject site will not be subject to planting restrictions as based on the *City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines)* from a geotechnical perspective.

7.0 Recommendations

It is recommended that the following be 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 design by Paterson, prior to construction, if applicable.
- ☐ Review of architectural plans pertaining to groundwater suppression system, 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 2729685 Ontario Inc. c/o Fotenn 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.



Owen R. Canton, B.Eng.



Faisal I. Abou-Seido, P.Eng.

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

DATUM Geodetic

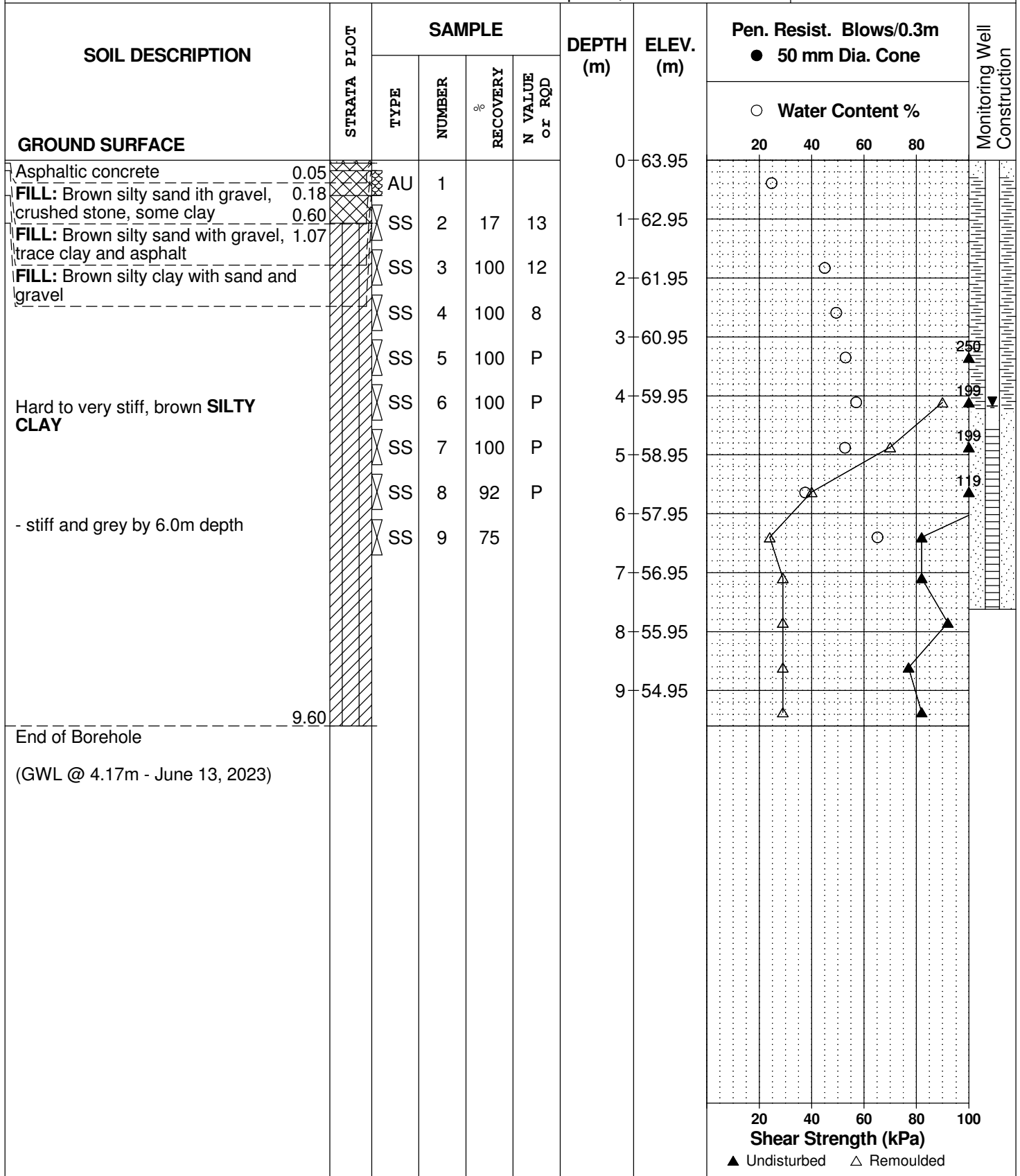
REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 18, 2023

FILE NO.
PG6622

HOLE NO.
BH 1-23



DATUM Geodetic

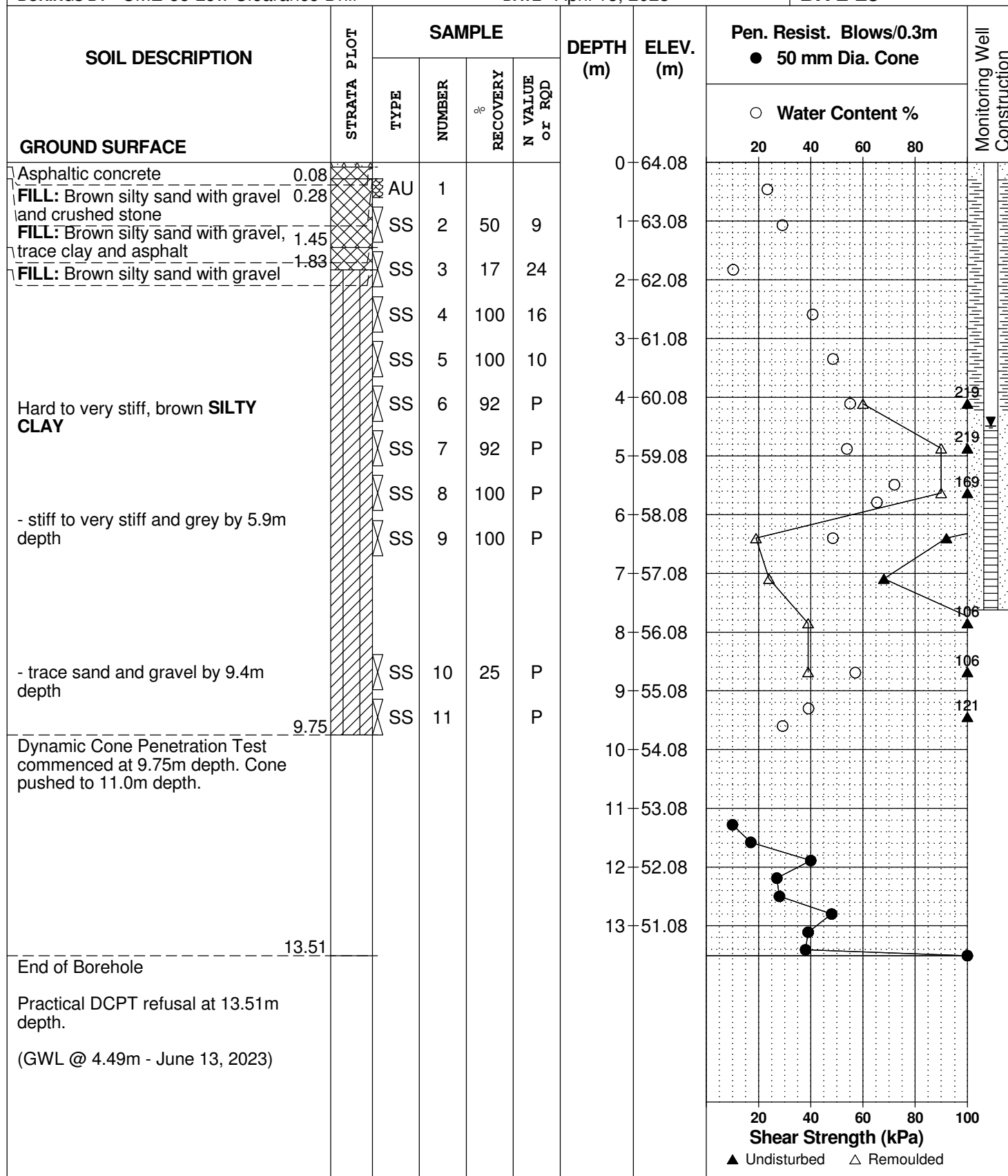
REMARKS

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DATE April 18, 2023

FILE NO.
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HOLE NO.
BH 2-23



DATUM Geodetic

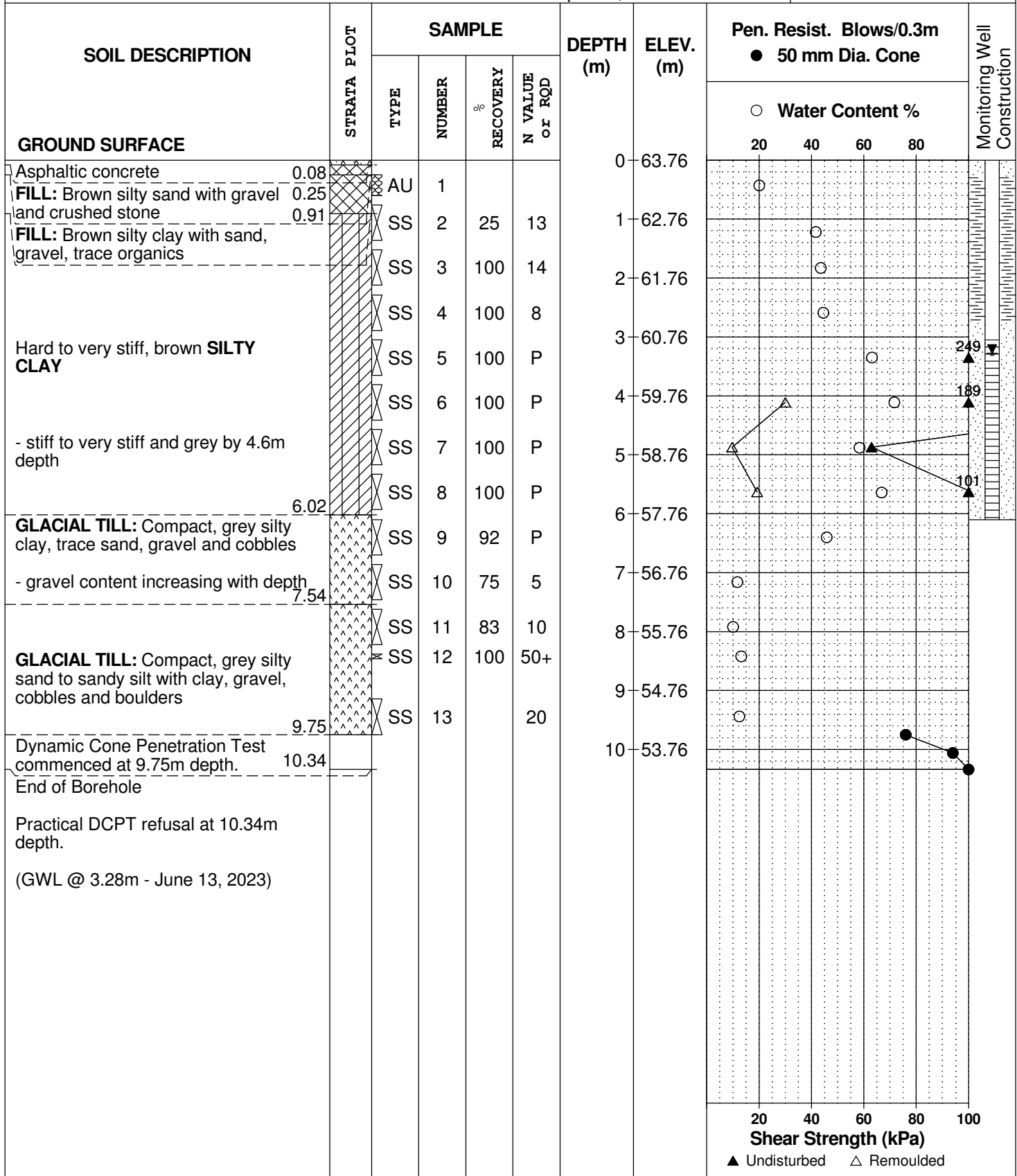
REMARKS

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DATE April 18, 2023

FILE NO.
PG6622

HOLE NO.
BH 3-23



DATUM Geodetic

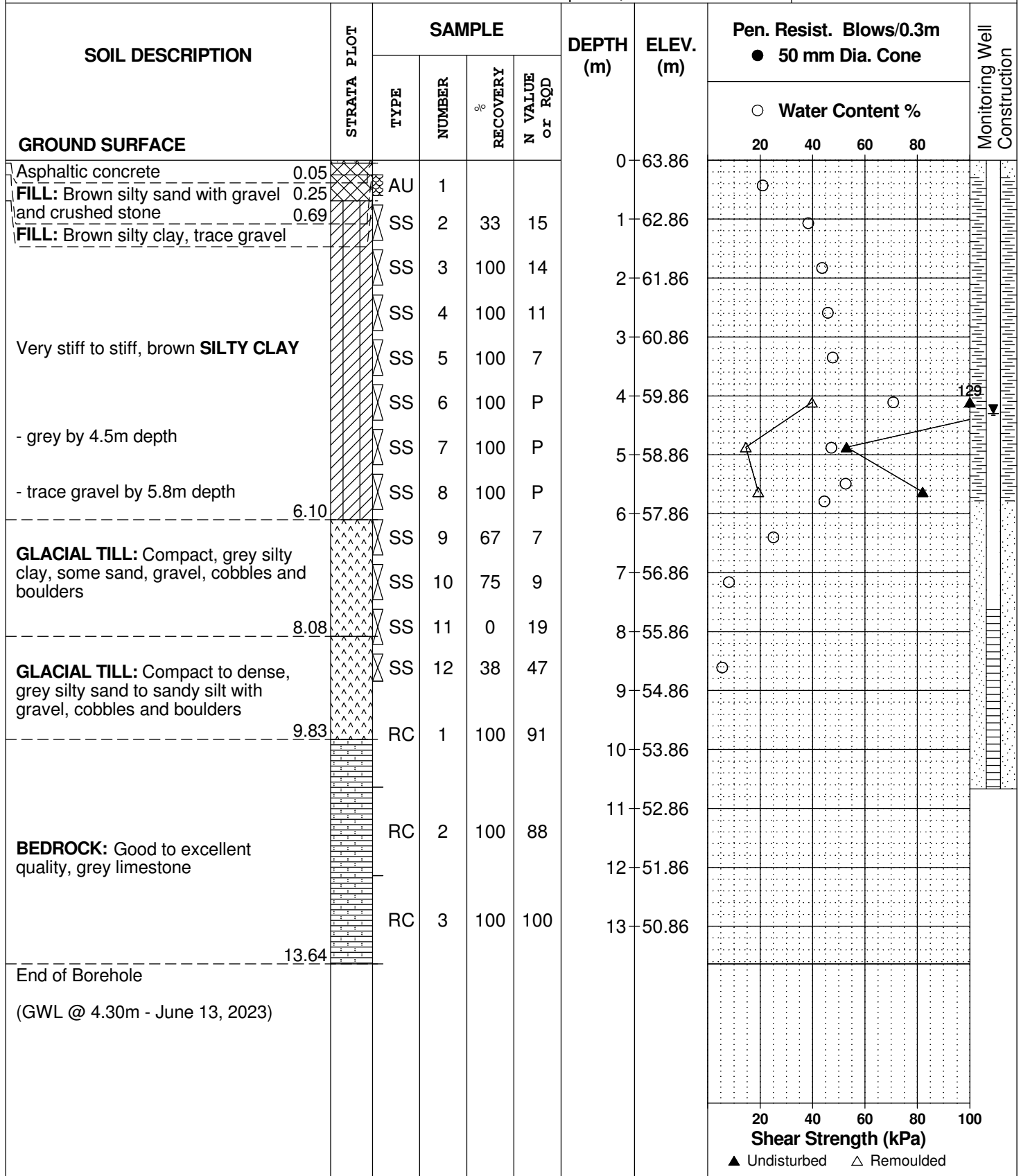
REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 19, 2023

FILE NO.
PG6622

HOLE NO.
BH 4-23



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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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, S_t , is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water 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)
D _{xx}	-	Grain size at 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
D ₁₀	-	Grain size at which 10% of the soil is finer (effective grain size)
D ₆₀	-	Grain size at which 60% of the soil is finer
C _c	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C _u	-	Uniformity coefficient = D_{60} / D_{10}

C_c and C_u are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < C_c < 3$ and $C_u > 4$

Well-graded sands have: $1 < C_c < 3$ and $C_u > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

C_c and C_u 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

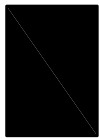
p' _o	-	Present effective overburden pressure at sample depth
p' _c	-	Preconsolidation pressure of (maximum past pressure on) sample
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
W _o	-	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.
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SYMBOLS AND TERMS (continued)

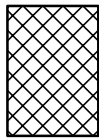
STRATA PLOT



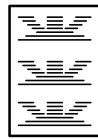
Topsoil



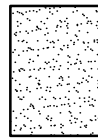
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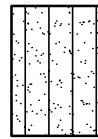
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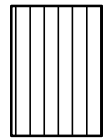
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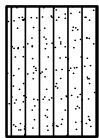
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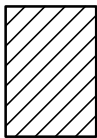
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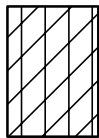
Silt



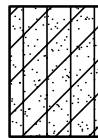
Sandy Silt



Clay



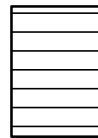
Silty Clay



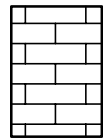
Clayey Silty Sand



Glacial Till



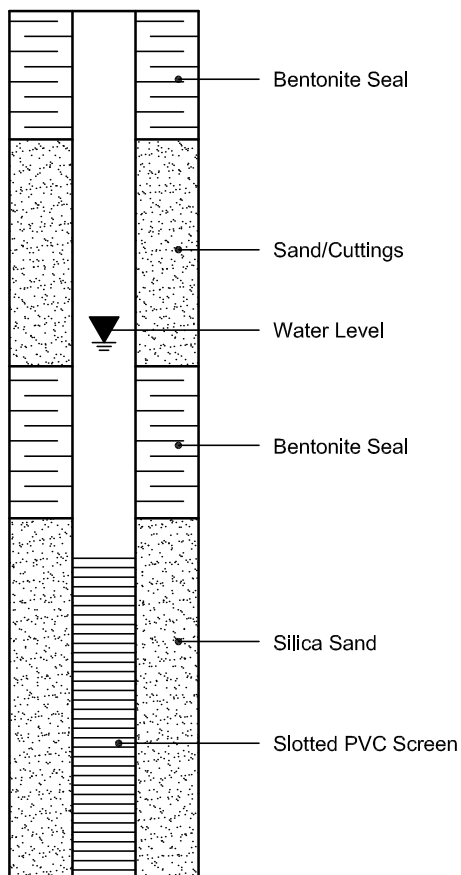
Shale



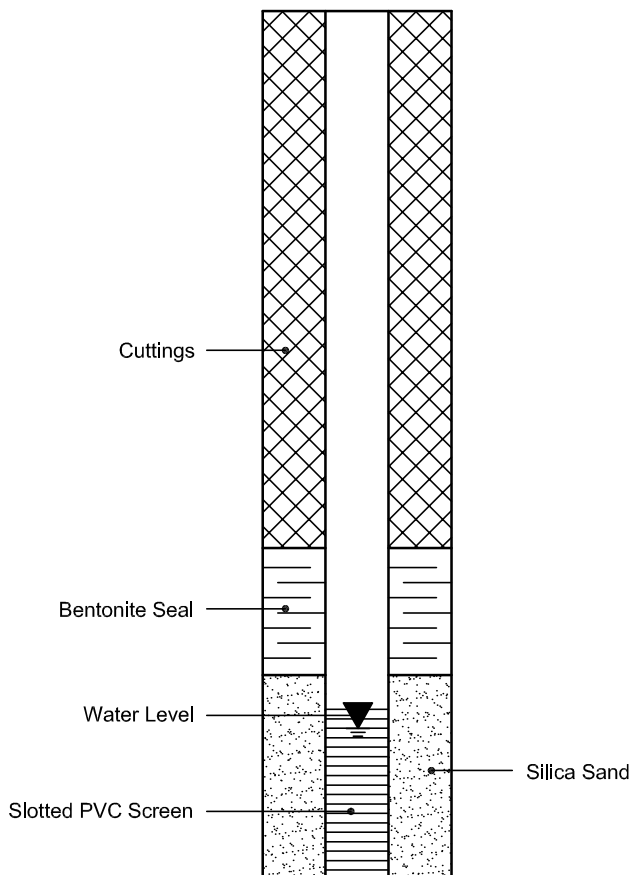
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 24-Apr-2023

Client: Paterson Group Consulting Engineers

Order Date: 19-Apr-2023

Client PO: 57262

Project Description: PG6622

Client ID:	BH2-23 SS6	-	-	-
Sample Date:	18-Apr-23 09:00	-	-	-
Sample ID:	2316337-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	66.5	-	-	-
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General Inorganics

pH	0.05 pH Units	7.67	-	-	-
Resistivity	0.1 Ohm.m	19.8	-	-	-

Anions

Chloride	10 ug/g dry	199	-	-	-
Sulphate	10 ug/g dry	155	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – PODIUM TO FOUNDATION TRANSITION DETAIL

DRAWING PG6622-1 – TEST HOLE LOCATION PLAN

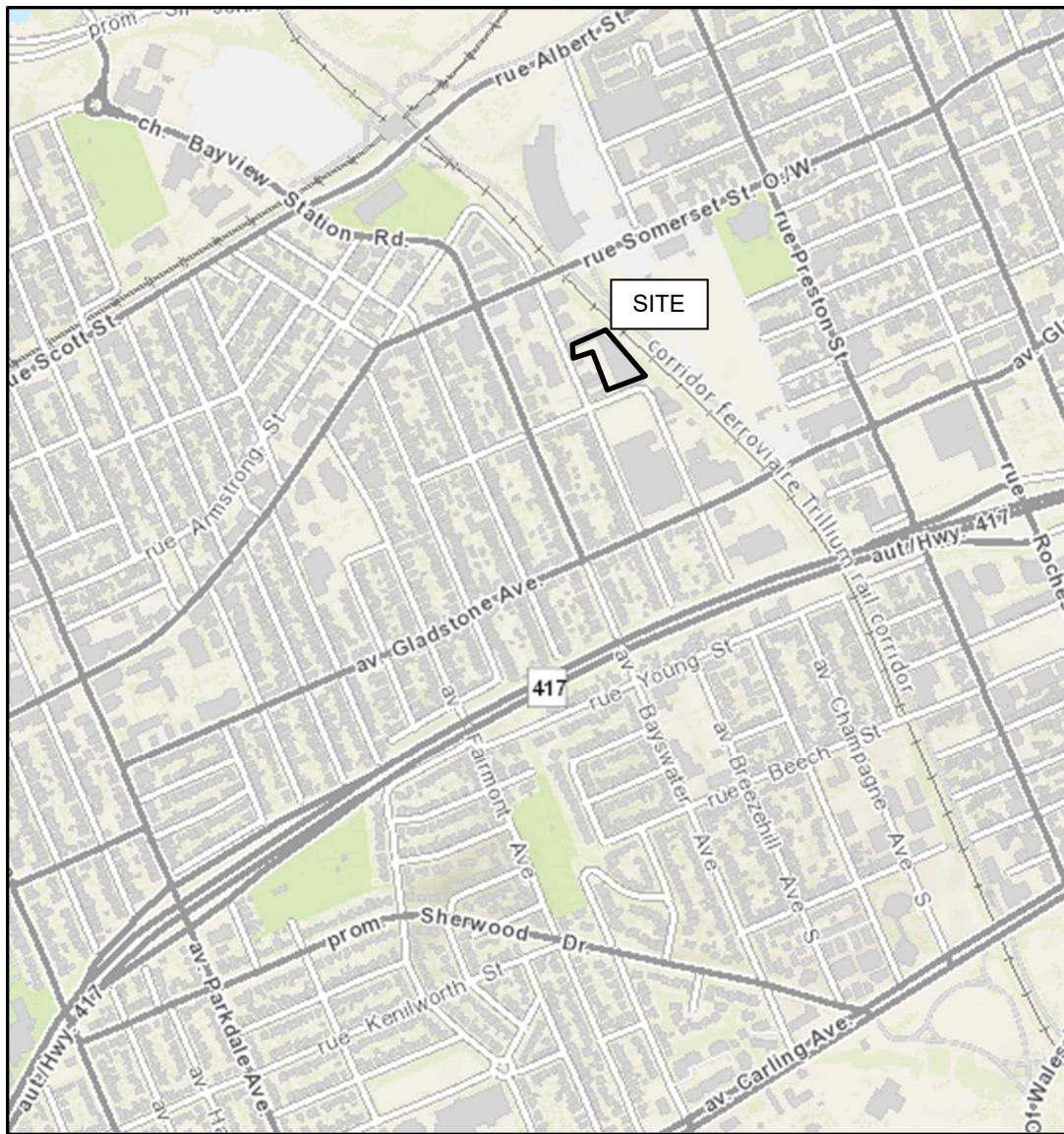


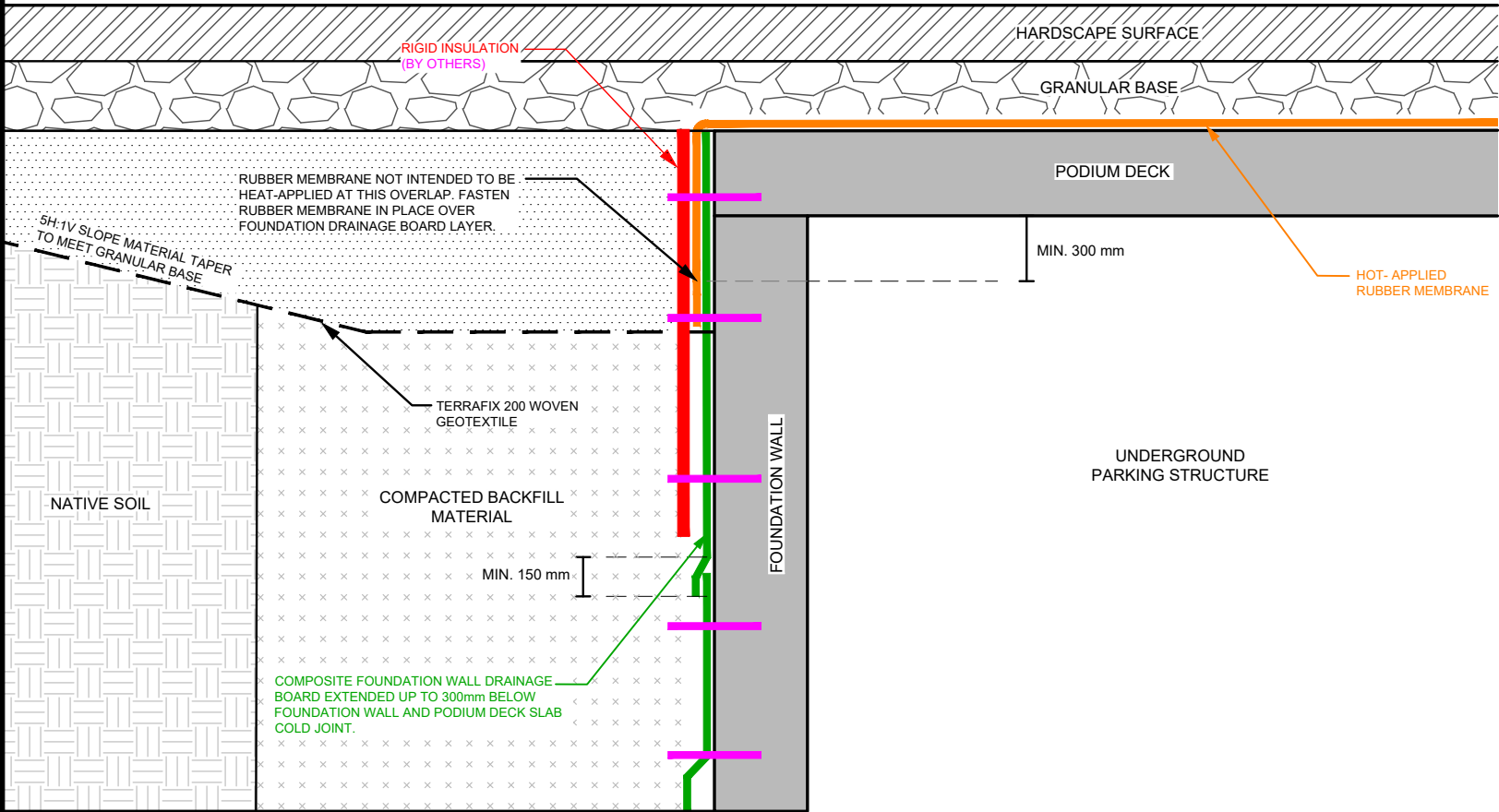
FIGURE 1

KEY PLAN

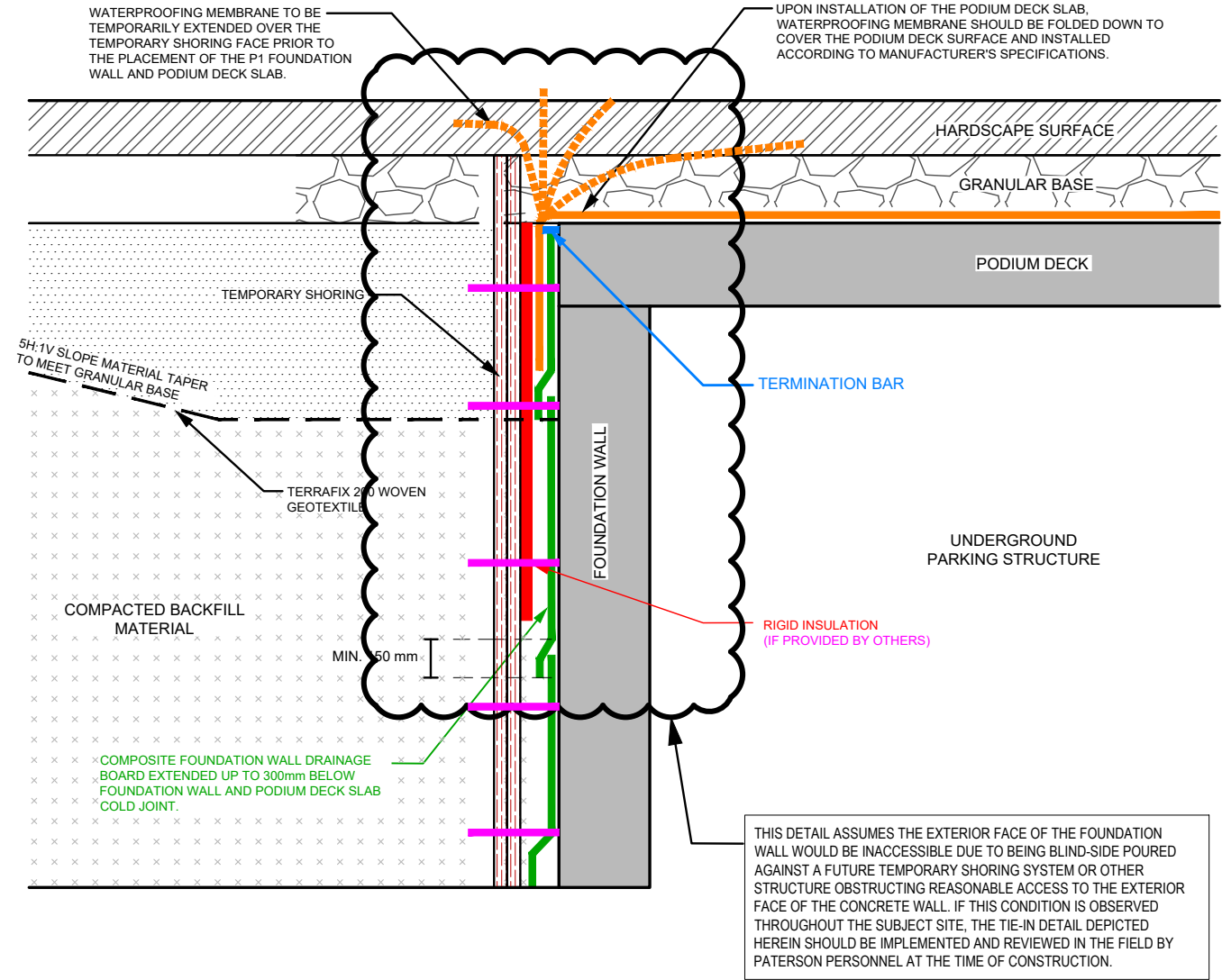


**PATERSON
GROUP**

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

0			
NO.	REVISIONS	DATE	INITIAL

SHOPPES AT FAIRWINDS C/O NA (GOULBORN) LIMITED PARTNERSHIP

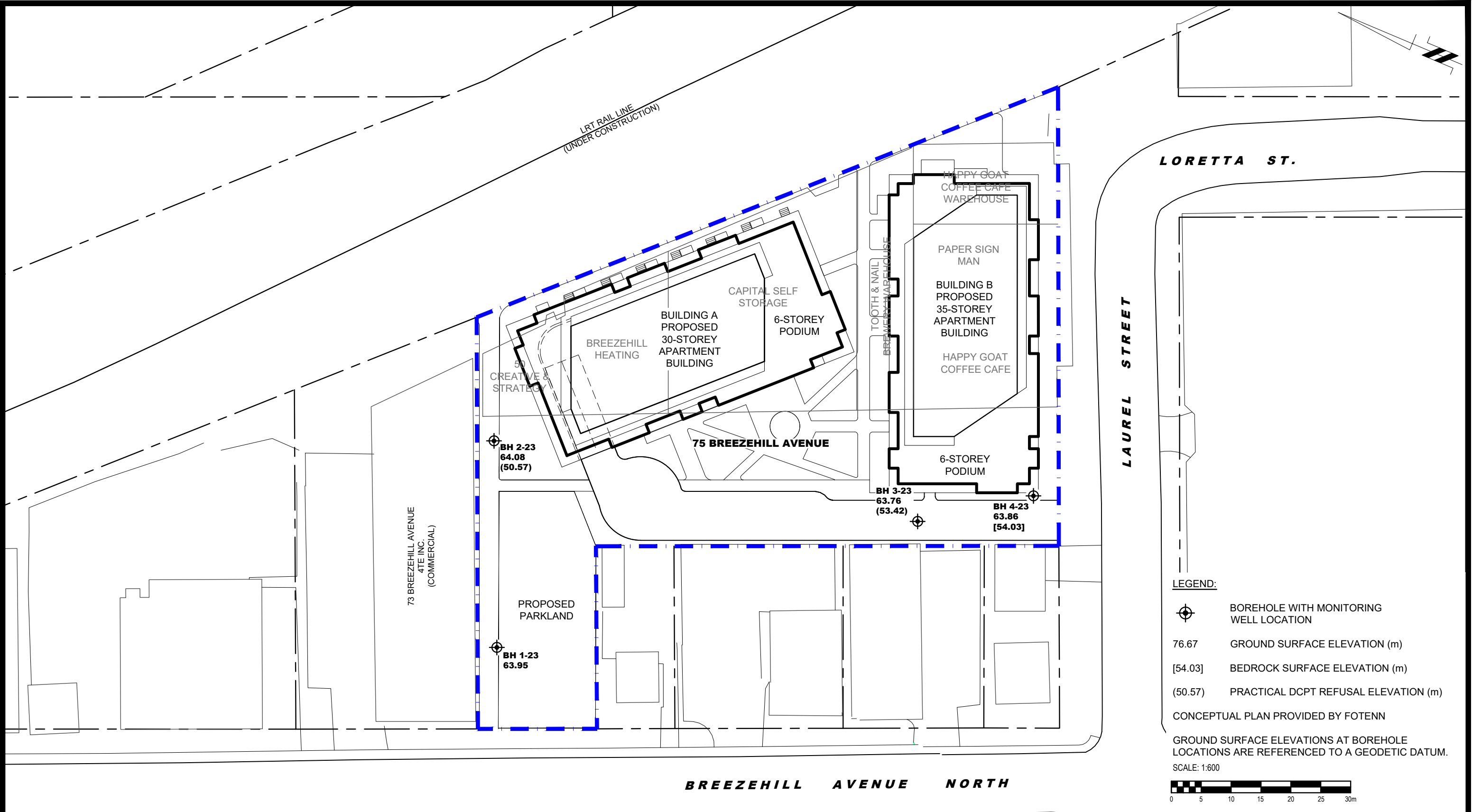
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-USE DEVELOPMENT
5705 HAZELDEAN ROAD

OTTAWA, ONTARIO

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

Scale:	N.T.S	Date:	06/2023
Drawn by:	RCG	Report No.:	PG6622-1
Checked by:	OC	Dwg. No.:	FIGURE 2
Approved by:	FA	Revision No.:	0

p:\aurocad\drawings\geotechnical\pg6622\fig 2.dwg





**PATERSON
GROUP**

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

1	UPDATED CONCEPTUAL PLAN	22/01/2024	PB
NO.	REVISIONS	DATE	INITIAL

2729685 ONTARIO INC. c/o FOTENN
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
75 BREEZEHILL AVENUE

OTTAWA,
Title:

ONTARIO

TEST HOLE LOCATION PLAN

Scale:	1:600	Date:	05/2023
Drawn by:	YA	Report No.:	PG6622-1
Checked by:	OC	Dwg. No.:	PG6622-1
Approved by:	FA	Revision No.:	1