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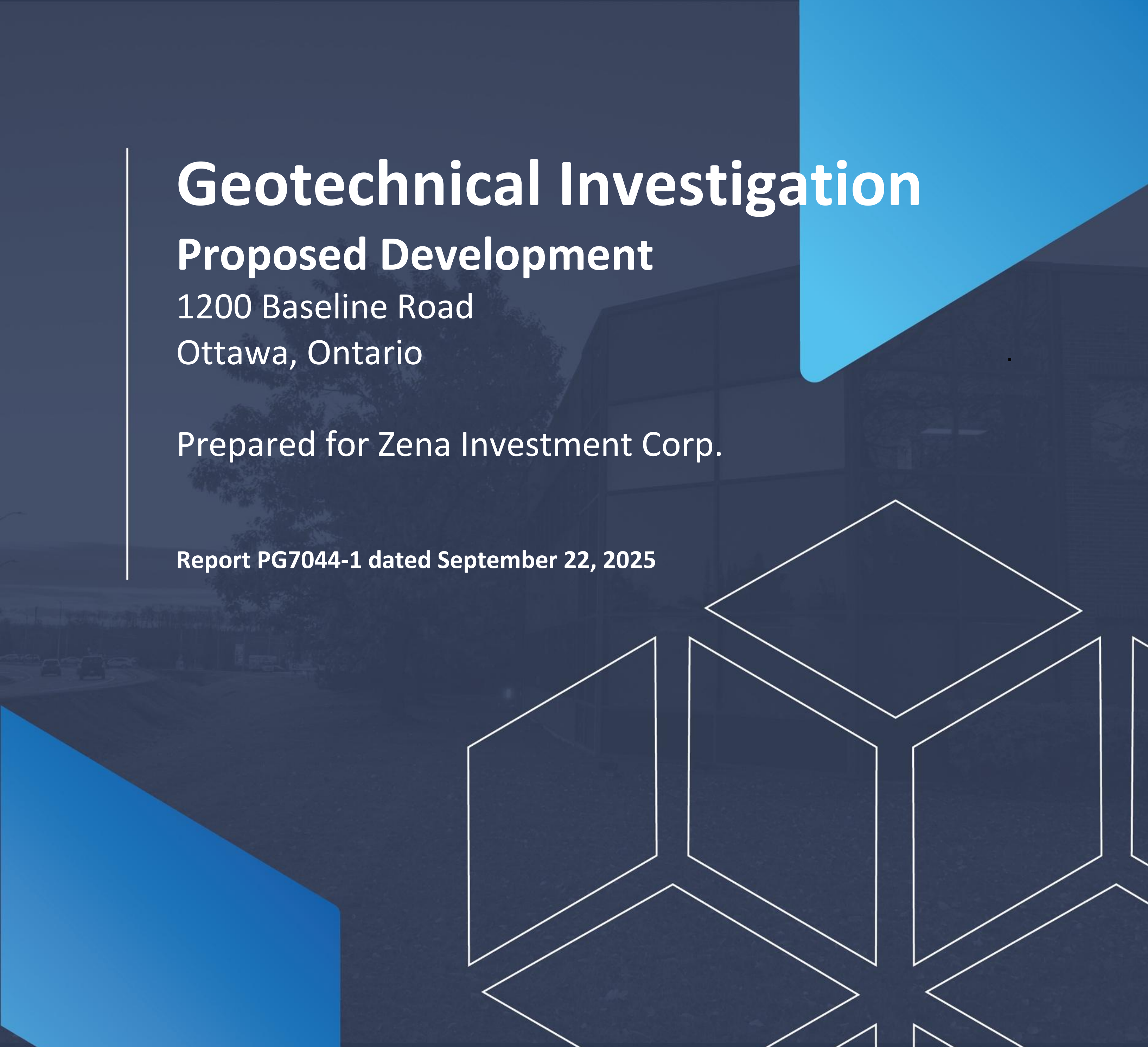

Geotechnical Investigation

Proposed Development

1200 Baseline Road
Ottawa, Ontario

Prepared for Zena Investment Corp.

Report PG7044-1 dated September 22, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Zena Investment Corp. to conduct a geotechnical investigation for the proposed development to be located at 1200 Baseline Road in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report 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 pertaining to design of the proposed development including construction considerations which may affect the design.

This 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 for 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 preliminary drawings, it is understood that the proposed development will consist of multiple mid-rise and high-rise mixed-use buildings with 2 or more levels of shared underground parking, which will occupy the majority of the subject site. Local access lanes, landscaped areas and surface parking are also proposed around the buildings.

Demolition of existing buildings will be required for development of the subject site. It is also expected that the proposed development will 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 March 5 and 16, 2025 and consisted of advancing a total of 16 boreholes to a maximum depth of 10.3 m below existing grade. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground utilities and site features. The approximate borehole locations are shown on Drawing PG7044-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a 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 bedrock coring to the required depths at the selected locations, and sampling and testing the overburden and bedrock.

Sampling and In Situ Testing

The soil samples were recovered 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 a 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 in Appendix 1.

A 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.

Bedrock samples were recovered from boreholes BH 1-25, BH 4-25 through BH 6-25, and BH 9-25 through BH 16-25 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. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. 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. The 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

Groundwater monitoring wells were installed in boreholes BH 1-25, BH 5-25, BH 6-25 and BH 9-25 through BH 16-25 to permit monitoring of the groundwater levels following the completion of drilling. Additionally, a standpipe piezometer was installed in borehole BH 4-25. The groundwater level readings were obtained after a suitable stabilization period subsequent to the completion of the field investigation.

3.2 Field Survey

The borehole locations, and ground surface elevation at each borehole location, were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The locations of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG7044-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. The results are discussed in Section 4.2 and are provided in Appendix 1 of this report.

Sample Storage

All samples will be stored in the laboratory for a period of 1 month after issuance of this report. 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 of sulphate attacks against subsurface concrete structures, one of which was collected from borehole BH 6-25. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing commercial development with buildings occupying the central portion of the western half of the site, and the majority of the eastern half of the subject site. The buildings are surrounded on all sides by asphalt paved access lanes and parking areas associated with the existing development.

The site is bordered to the northwest by Merivale Road, to the northeast by Baseline Road, to the east by an institutional property, to the south by existing residential buildings, and to the west by an existing commercial property. The ground surface across the subject site was observed to slope gently downward from the northwest to southeast at approximate geodetic elevations of 97 to 95 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of an approximate 60 to 100 mm thickness of asphaltic concrete underlain by fill material and glacial till. Boreholes BH 13-25 through BH 16-25 were completed within the southwest corner of the eastern building by coring through the concrete floor slab. At these locations, the concrete slab thickness ranged from approximately 150 to 180 mm.

The fill material encountered underlying the asphalt or concrete slab was observed to extend to maximum depths 0.3 to 2.6 m below the existing ground surface. The fill was generally observed to consist of brown silty sand with varying amounts of clay, gravel, crushed stone and cobbles.

Topsoil was encountered underlying the fill layer at boreholes BH 8A-25 and BH 13-25 and extended to maximum depths of 1.3 and 0.6 m, respectively.

A layer of very stiff brown clayey silt with sand was encountered below the fill layer at borehole BH 6-25 in the southeast corner of the site, becoming stiff and grey in colour at an approximate depth of 3.7 m. The clayey silt extended to an approximate depth of 4.4 m below ground surface.

A glacial till deposit was encountered underlying the clayey silt or fill layer at boreholes BH 6-25, BH 13-25 and BH 16-25, and extended to approximate depths of 5.8, 0.9 and 2.4 m, respectively. The glacial till deposit was observed to consist of brown to grey sandy silt to silty clay with gravel, cobbles and boulders.

Bedrock

Practical refusal to augering was encountered at approximate depths ranging from 0.6 to 5.8 m. The bedrock was cored at boreholes BH 1-25, BH 4-25 to BH 6-25, and BH 9-25 through BH 16-25 and cored to depths ranging from 4.4 m to 10.3 m below the existing ground surface. Based on the recovered rock core samples, the bedrock was observed to consist of limestone of very poor to poor in quality in the upper 0.3 to 2.0 m, becoming fair to excellent with depth.

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

Groundwater levels were measured within the installed monitoring wells and flexible standpipe on March 24, 2025. The measured groundwater are presented in Table 1 below and are shown on the Soil Profile and Test Data Sheets in Appendix 1.

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-25*	96.60	3.67	92.93	March 24, 2025
BH 4-25	97.13	3.82	93.31	March 24, 2025
BH 5-25*	96.46	2.93	93.53	March 24, 2025
BH 6-25*	94.17	2.92	91.25	March 24, 2025
BH 9-25*	95.39	1.85	93.54	March 24, 2025
BH 10-25*	95.32	2.20	93.12	March 24, 2025
BH 11-25*	96.94	1.95	94.99	March 24, 2025
BH 12-25*	95.30	3.16	92.14	March 24, 2025
BH 13-25*	95.46	1.93	93.53	March 24, 2025
BH 14-25*	95.46	1.31	94.15	March 24, 2025
BH 15-25*	95.46	2.25	93.21	March 24, 2025
BH 16-25*	95.46	3.14	92.32	March 24, 2025

Note: * - Denotes Monitoring Well

Based on the measured water levels, the groundwater table is estimated to range between approximately **2 to 4 m** below the existing ground surface. However, it should be noted that groundwater levels are subject to seasonal fluctuations, and could therefore 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. It is recommended that the proposed building be founded on conventional spread footings placed on clean, surface-sounded bedrock.

Bedrock removal will be required to complete the excavation for the underground parking levels. 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 of the proposed buildings, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed buildings.

Existing foundation walls and other demolition debris should completely be removed from the proposed building perimeter and within the lateral support zones of the foundation. Under paved area, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

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.
As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

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.

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 or blast rock fill approved by the geotechnical consultant. 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.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. Where the fill is open graded, a woven geotextile at the surface may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement.

Under winter conditions, if snow and ice is present within the blast rock 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. The geotechnical consultant should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

Lean Concrete Filled Trenches

Where rock overbreak occurs at the underside of footing (USF) elevation, lean concrete (minimum **17 MPa** 28-day compressive strength) can be used to reinstate the subgrade from the bedrock surface to the USF elevation. Typically, the excavation side walls will be used as the form to support the concrete. The lean concrete placement 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 will suffice in providing a direct transfer of the footing load to the underlying bedrock.

5.3 Foundation Design

Footings placed on a clean, surface-sounded bedrock, or on lean concrete placed directly over the clean, surface-sounded bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

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 prior to concrete placement for footings.

Footings supported directly on clean, surface sounded bedrock, designed for the bearing resistance values provided above, will be subject 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 through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock or soil bearing medium will require a lateral support zone of 1.5H: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 is required for the proposed buildings which are founded on, or within 3 m of, the bedrock surface, a site-specific shear wave velocity test should be completed in accordance with the Ontario Building Code (OBC) 2024 to accurately determine the applicable seismic site classification for foundation design.

Based on Paterson's review of the in-situ soil characteristics, the soils underlying the subject site are not considered susceptible to liquefaction.

5.5 Basement Floor Slab

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

It is anticipated that the underground levels for the proposed buildings will primarily consist of parking areas and the rigid pavement structure recommendation provided in Section 5.8 will be applicable. However, if storage or other uses of the lower level are considered where a concrete floor slab will be constructed, the upper 300 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

Any soft areas in the basement slab subgrade should be removed and backfilled with appropriate backfill material prior to placing 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. 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 98% of the SPMDD.

In consideration of the groundwater conditions at the site and the depth of excavation, a sub-slab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the lowest level floor slab. 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 multi-storey 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 bulk (drained) unit weight of 20 kN/m^3.

However, it is expected that the majority of the basement walls are to be poured against a composite drainage board, which will be placed against the exposed bedrock face. A nominal coefficient for 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 is to be 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.

Two distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ 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:

$a_c = (1.45 - a_{max}/g)amax$
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 subject site is 0.349g for a Site Class X_C 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 $K_o = 0.5$ 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 2024.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 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 load capacity of each individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

It should be further noted that the centre to centre spacing between bond lengths be at least four (4) times the diameter of the anchor holes and greater than one fifth (1/5) of the total anchor length or a minimum of 1.2 m to decrease the group influence effects. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and fluid grout 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 is provided with a fixed anchor length at the anchor base, 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, then 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, the entire drill hole should be filled with cementitious grout.

The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long-term performance of the foundation of the proposed building, if required, any rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of limestone ranges between about 40 and 100 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 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on bedrock 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 below:

Table 2 – Parameters Used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone	50 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

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 on the following page:

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	2.2	0.7	2.9	500
	3.4	0.5	3.9	750
	4.3	0.4	4.7	1000
125	1.4	1.1	2.5	500
	2.0	1.2	3.2	750
	2.6	1.2	3.8	1000

Other Considerations

The anchor drill holes should be a maximum of 1.5 to 2 times the rock anchor diameter, inspected by Paterson personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor hole. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction and reviewed at the time of testing by Paterson field personnel.

5.8 Pavement Design

Lowest Underground Parking Level

For design purposes, it is recommended that the rigid pavement structure for the lower underground parking levels of the proposed buildings consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The following rigid pavement structure is recommended:

Table 4 – Recommended Rigid Pavement Structure – Lowest 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 in bedrock.	

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

Podium Deck Area

It is anticipated that the podium deck structure will be provided for car only parking areas, access lanes, fire truck lanes and loading areas. Based on the concrete slab subgrade, the pavement structure indicated in the Tables 5 and 6 may be considered for design purposes:

Table 5 – Recommended Pavement Structure – Car-Only Parking Areas (Podium Deck)

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 *If specified by others, not required from a geotechnical perspective **Thickness is dependent on grade of insulation as noted in proceeding paragraph	

Table 6 – Recommended Pavement Structure – Access Lanes, Fire Truck Lanes, Ramp and Heavy Truck Parking Areas (Podium Deck)

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 IKO Protection Board**
SUBGRADE – Reinforced Concrete Podium Deck *If specified by others, not required from a geotechnical perspective **Thickness is dependent on grade of insulation as noted in proceeding paragraph	

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 is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 500 mm below the top of the podium slab a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified 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). The pavement structures base layer thickness will be dependent on the grade of insulation considered for this project and should be reassessed by the geotechnical consultant once pertinent design details have been prepared.

The higher grades of insulation have more resistance to deformation under wheel-loading and require less granular cover to avoid being crushing by vehicular loading. It should be noted that SM (Styrofoam) rigid insulation is not considered suitable for this application.

Pavement Structure On Overburden Soils

Beyond the podium deck, the following pavement structures may be considered for car only parking and heavy traffic areas. The proposed pavement structures are shown in Tables 7 and 8 on the following page.

Table 7 – Recommended Pavement Structure – Car Only Parking Areas

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

Table 8 – Recommended Pavement Structure – Access Lanes and Heavy Loading Area	
Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE** – OPSS Granular A Crushed Stone
400	**SUBBASE** – OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or fill.	

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 I or 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 material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Waterproofing

Foundation Drainage

It is recommended that the proposed building foundation walls be blind-poured and placed against a composite drainage board which is fastened to the temporary shoring system or vertical bedrock face.

For the installation of the composite drainage board against the vertical bedrock face, the following is recommended:

☐ Line drill the excavation perimeter (usually at 150 to 200 mm spacing).

☐ Mechanically remove bedrock along the foundation walls, up to approximately 150 mm from the finished vertical excavation face.

☐ Grind the bedrock surface up to the outer face of the line drilled holes to create a satisfactory surface for the composite drainage board.

☐ If bedrock overbreaks occur, shotcrete these areas to fill in cavities and to smooth out angular features of the bedrock surface, as required based on site inspection by Paterson.

☐ Place a composite drainage board, such as Delta Drain 6000 or equivalent, against the prepared vertical bedrock surface. The composite drainage layer should extend from finished grade to underside of footing level.

☐ Pour foundation wall against the composite drainage board.

It is recommended that 100 mm diameter sleeves at 3 m centres be cast at the foundation wall/footing interface to allow for the infiltration of water from the composite drainage board to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Elevators and any other pits located below the underslab drainage system should be waterproofed.

Perimeter and Underslab Drainage System

The perimeter and underslab drainage system is recommended to control water infiltration below the underground parking level slab and to re-direct water from the buildings foundation drainage system to the building's sump pit(s). For preliminary design purposes, it is recommended that 100 mm perforated pipes with a geosock, surrounded on all sides by a minimum 150 mm thick layer of 19 mm clear crushed stone, be placed at approximate 6 m centres underlying the underground parking level slab.

The perimeter drainage system should be mechanically connected to the 100 mm drainage sleeves and gravity connected to the underslab drainage system, which in turn is connected to the building's sump pit(s).

The spacing of the underslab drainage system should be confirmed by the geotechnical consultant at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Above the bedrock surface, where space may be available for backfilling, the backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials, such as clean sand or OPSS Granular B Type I granular material.

6.2 Protection of Footings Against Frost Action

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

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

However, the foundations 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 materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower 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.

A trench box is recommended 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.

Bedrock Stabilization

Where required, 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.

Due to the bedrock quality encountered at the site, horizontal rock anchors and chain link fencing connected to the excavation face will 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. It is expected that bedrock stabilization measures will be required for the upper weathered portion of the bedrock, extending to approximately 1 to 2 m below the bedrock surface. However, the specific requirements for bedrock stabilization measures will be evaluated during the excavation operations.

Temporary Shoring

Depending on the depth of excavation of the buildings and the proximity of the proposed building to the property boundaries, temporary shoring may be required to support the overburden soils 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 pile 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 parameters in Table 9 below:

Table 9 – 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/m$_3$	21
Submerged Unit Weight , 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 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.

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 or weathered bedrock subgrade. If the bedding is placed on clean, surface sounded bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. 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 should generally be possible to re-use 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

It is anticipated that groundwater infiltration into the excavations should 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.

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.

Impacts on Neighbouring Properties

Shallow bedrock was generally encountered at this site, which is not susceptible to settlement from dewatering. Therefore, no issues are expected with respect to groundwater lowering that could 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 by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. It should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above the frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable. 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 an aggressive to very aggressive corrosive environment.

7.0 Recommendations

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

- ☐ Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction, if required.
- ☐ Review of the bedrock stabilization and excavation requirements.
- ☐ Review of the proposed groundwater infiltration control system, foundation drainage and requirements.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should 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 Zena Investment Corporation, 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.

Kevin Pickard, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Sept. 22, 2025
K. A. PICKARD
100531344
Kevin Pickard
PROVINCE OF ONTARIO

Scott Dennis
Scott S. Dennis, P.Eng.

Report Distribution:

☐ Zena Investment Corporation (email copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364983.13 **NORTHING:** 5025178.22 **ELEVATION:** 96.60

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 5, 2025

FILE NO. : PG7044

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION

GROUND SURFACE

ASPHALT 0.10m [96.50m]

FILL: Compact, brown silty sand, some gravel and clay

- Rock fragments from 0.99 m depth

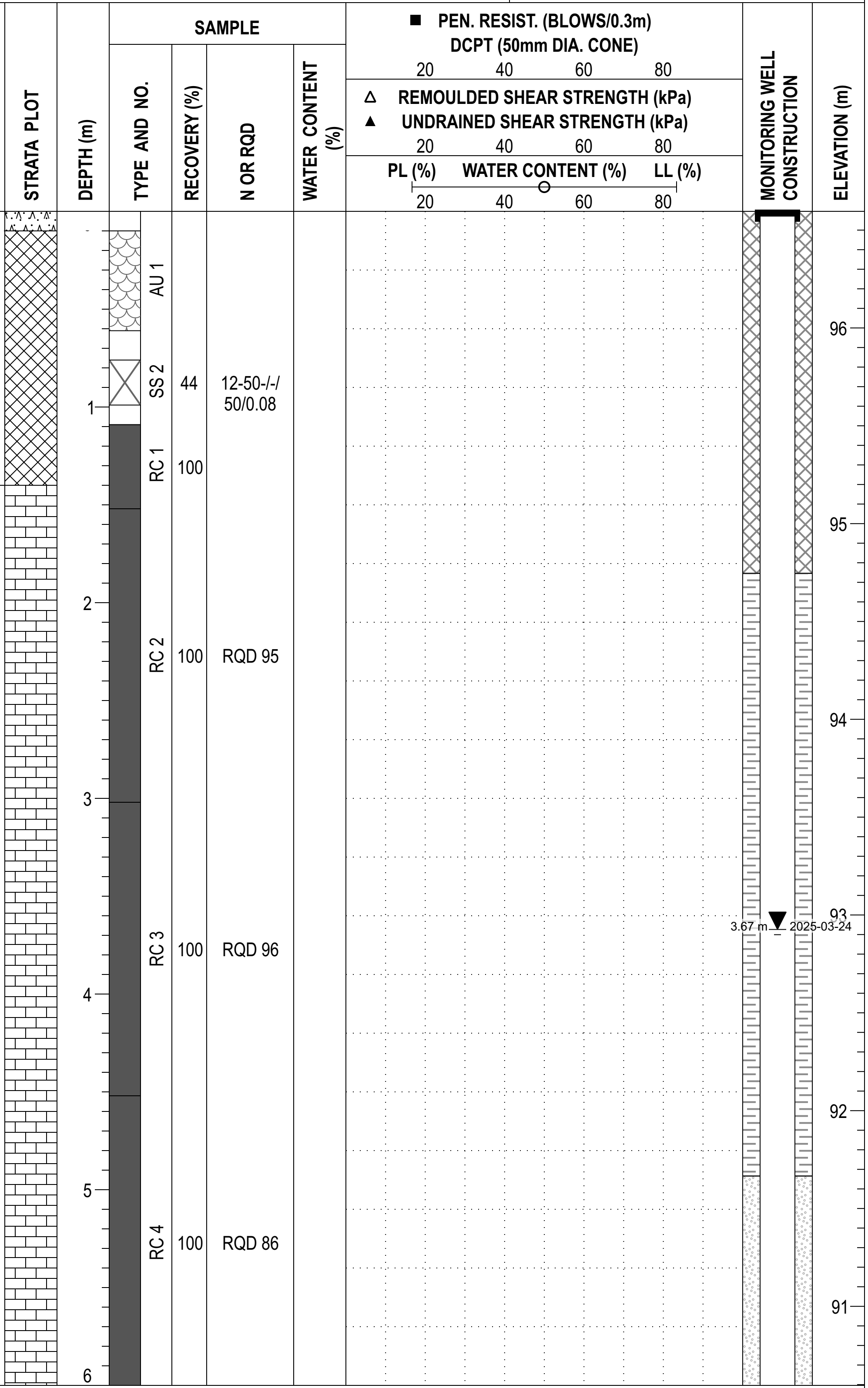
1.40m [95.20m]

BEDROCK: Excellent to good quality limestone bedrock

- Thin mud seams at 1.98 m depth

- Vertical fracture from 2.39 to 2.64 m depth

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			
SS 2	44	12-50-/-/ 50/0.08	
RC 1	100		
RC 2	100	RQD 95	
RC 3	100	RQD 96	
RC 4	100	RQD 86	



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364983.13 **NORTHING:** 5025178.22 **ELEVATION:** 96.60

PROJECT: Proposed Development

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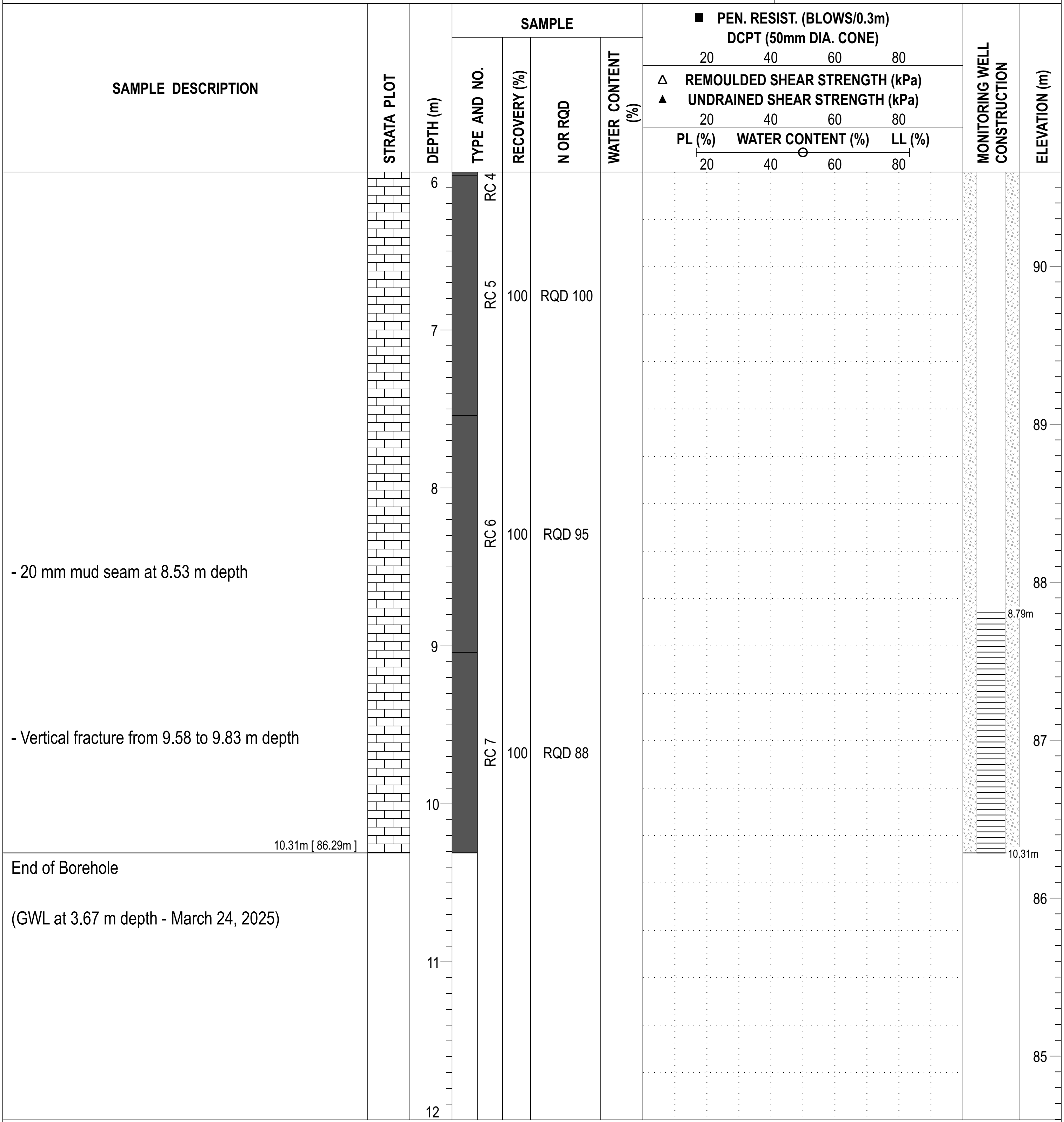


- 20 mm mud seam at 8.53 m depth

- Vertical fracture from 9.58 to 9.83 m depth

10.31m [86.29m]

End of Borehole

(GWL at 3.67 m depth - March 24, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD
RC 4		
RC 5	100	RQD 100
RC 6	100	RQD 95
RC 7	100	RQD 88

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365046.00 **NORTHING:** 5025180.80 **ELEVATION:** 95.26

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 5, 2025

FILE NO. : PG7044

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
ASPHALT 0.08m [95.18m]									95
FILL: Compact, brown silty sand with gravel, trace clay and crushed stone			AU 1						
			SS 2	28	21-50-/-/ 50/0.03				
1.07m [94.19m]		1							94
End of Borehole Practical refusal to augering at 1.07 m depth		2							93
		3							92
		4							91
		5							90
		6							

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365096.66 **NORTHING:** 5025211.49 **ELEVATION:** 95.42

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 5, 2025

FILE NO. : PG7044

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION

GROUND SURFACE

ASPHALT 0.08m [95.34m]

FILL: Compact to dense, brown silty sand with gravel, trace clay

- Rock fragments from 0.76 m depth

1.57m [93.86m]

End of Borehole

Practical refusal to augering at 1.57 m depth

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			
SS 2	4	20-21-15-13 36	

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

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

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

PIEZOMETER CONSTRUCTION

DEPTH (m): 1 2 3 4 5 6

ELEVATION (m): 95 94 93 92 91 90

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365046.12 **NORTHING:** 5025238.92 **ELEVATION:** 97.13

PROJECT: Proposed Development

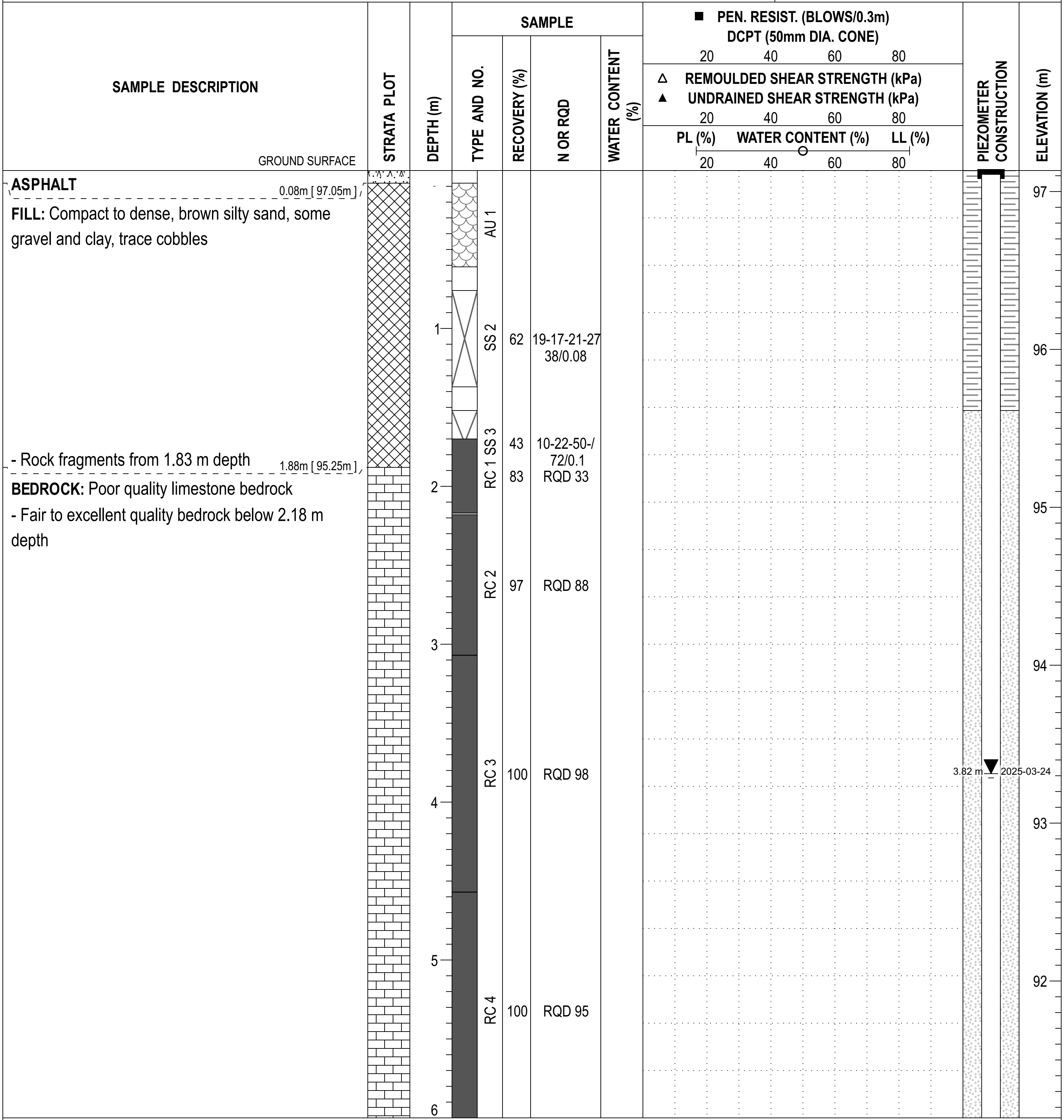
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 6, 2025

FILE NO. : PG7044

HOLE NO. : BH 4-25



ASPHALT 0.08m [97.05m]

FILL: Compact to dense, brown silty sand, some gravel and clay, trace cobbles

- Rock fragments from 1.83 m depth 1.88m [95.25m]

BEDROCK: Poor quality limestone bedrock

- Fair to excellent quality bedrock below 2.18 m depth

TYPE AND NO.	RECOVERY (%)	N OR RQD
AU 1		
SS 2	62	19-17-21-27 38/0.08
SS 3	43	10-22-50-/ 72/0.1
RC 1	83	RQD 33
RC 2	97	RQD 88
RC 3	100	RQD 98
RC 4	100	RQD 95

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SOIL PROFILE AND TEST DATA
Geotechnical Investigation
1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365046.12 **NORTHING:** 5025238.92 **ELEVATION:** 97.13

PROJECT: Proposed Development
ADVANCED BY: CME-55 Low Clearance Drill
REMARKS:
DATE: March 6, 2025

FILE NO. : PG7044
HOLE NO. : BH 4-25

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	ELEVATION (m)
	6	RC 4				91
	7	RC 5	100	RQD 95		90
	8	RC 6	100	RQD 98		89
- Fair quality bedrock below 9.18 m depth	9					88
- 10 mm mud seams at 9.5 m depth		RC 7	95	RQD 69		
10.16m [86.97m]	10					87
End of Borehole (GWL at 3.82 m depth - March 24, 2025)	11					86
	12					

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

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa)
20 40 60 80

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

STRATA PLOT

PIEZOMETER CONSTRUCTION

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365151.43 **NORTHING:** 5025307.14 **ELEVATION:** 96.46

PROJECT: Proposed Development

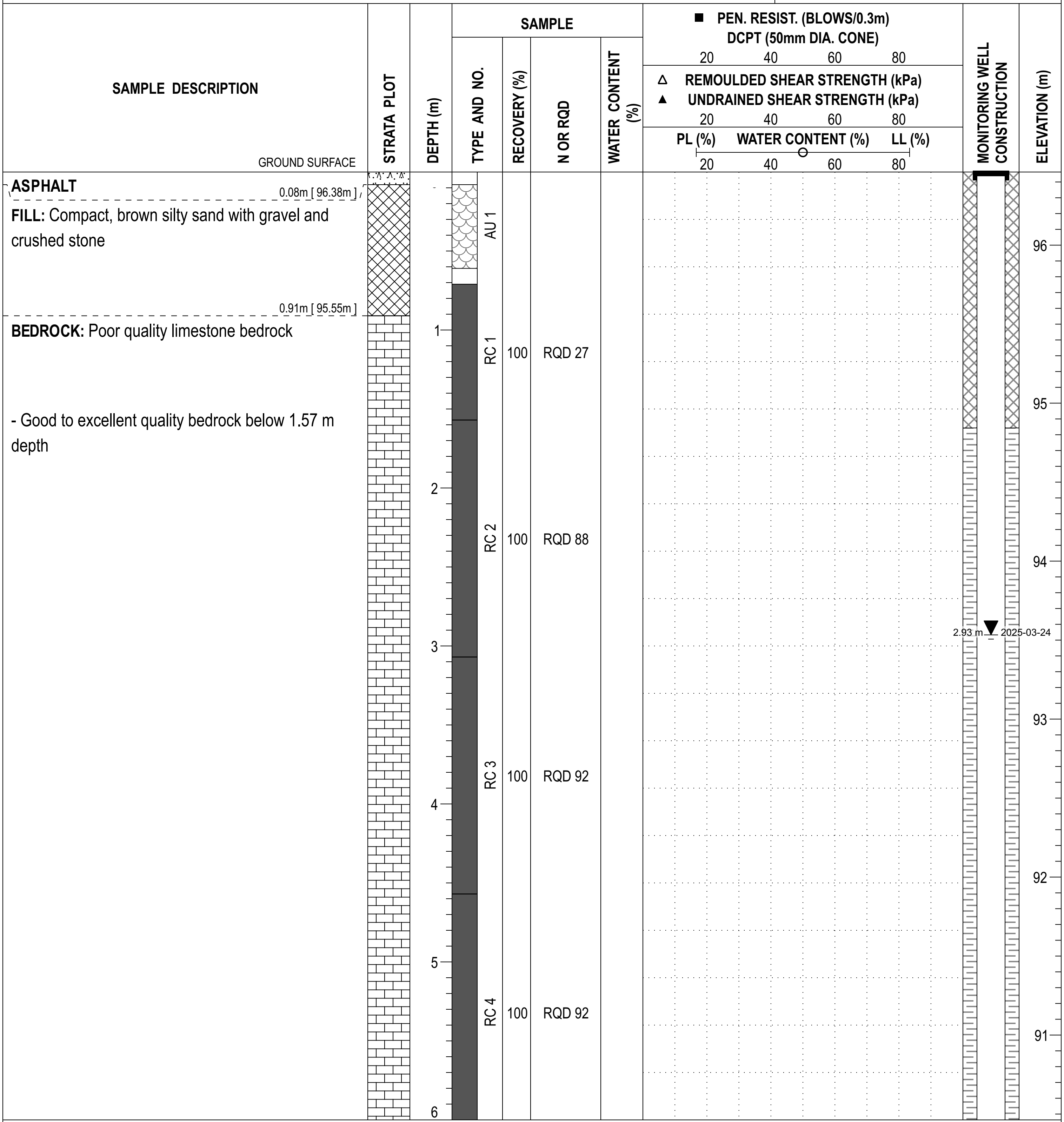
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 7, 2025

FILE NO. : PG7044

HOLE NO. : BH 5-25



ASPHALT

FILL: Compact, brown silty sand with gravel and crushed stone

BEDROCK: Poor quality limestone bedrock

- Good to excellent quality bedrock below 1.57 m depth

TYPE AND NO.	RECOVERY (%)	N OR RQD
AU 1		
RC 1	100	RQD 27
RC 2	100	RQD 88
RC 3	100	RQD 92
RC 4	100	RQD 92

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365151.43 **NORTHING:** 5025307.14 **ELEVATION:** 96.46

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 7, 2025

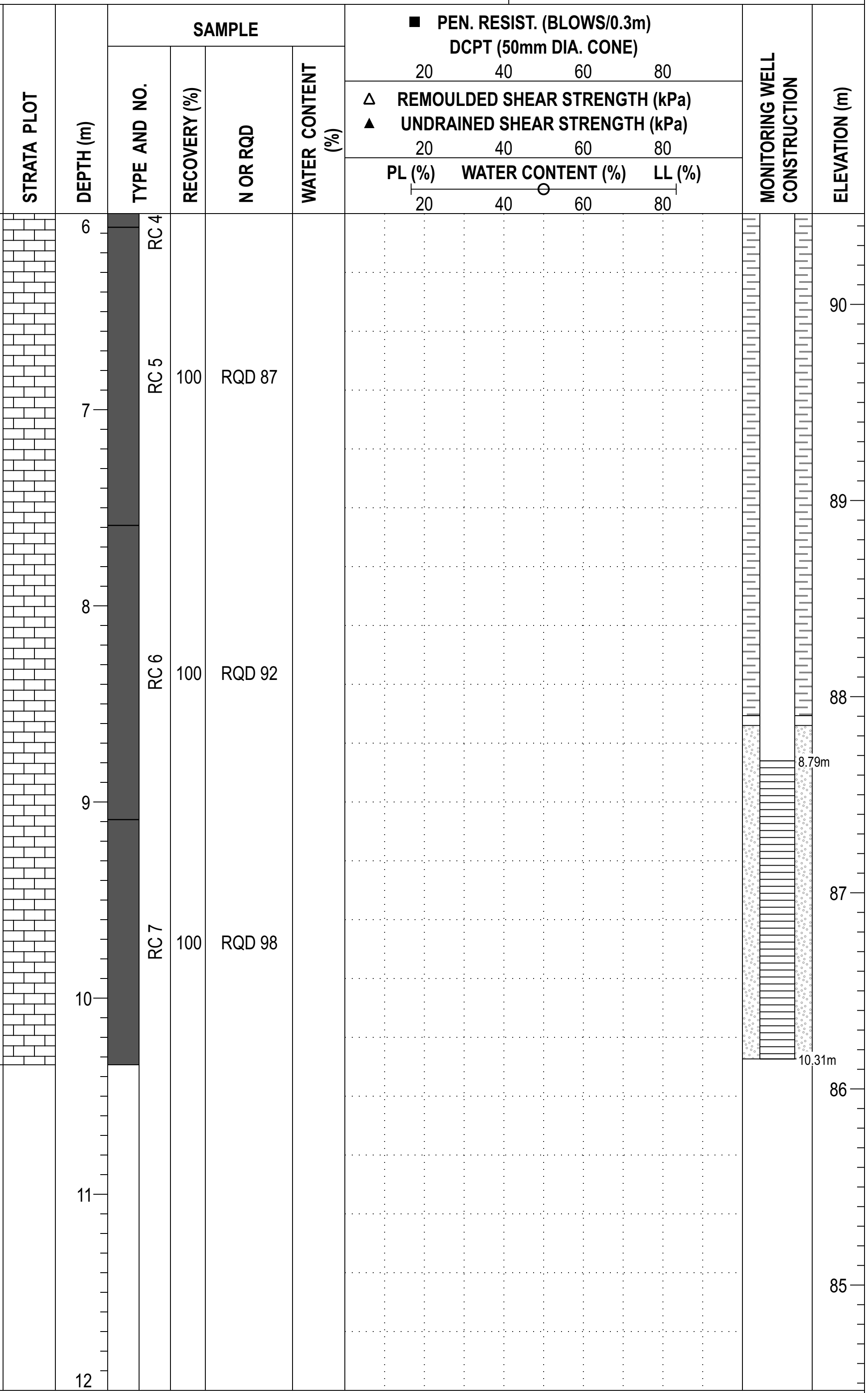
FILE NO. : PG7044

HOLE NO. : BH 5-25

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
- Vertical fracture from 6.1 to 6.55 m depth	6	RC 4			
		RC 5	100	RQD 87	
		RC 6	100	RQD 92	
10.34m [86.12m]		RC 7	100	RQD 98	

End of Borehole

(GWL at 2.93 m depth - March 24, 2025)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365207.11 **NORTHING:** 5025197.47 **ELEVATION:** 94.17

PROJECT: Proposed Development

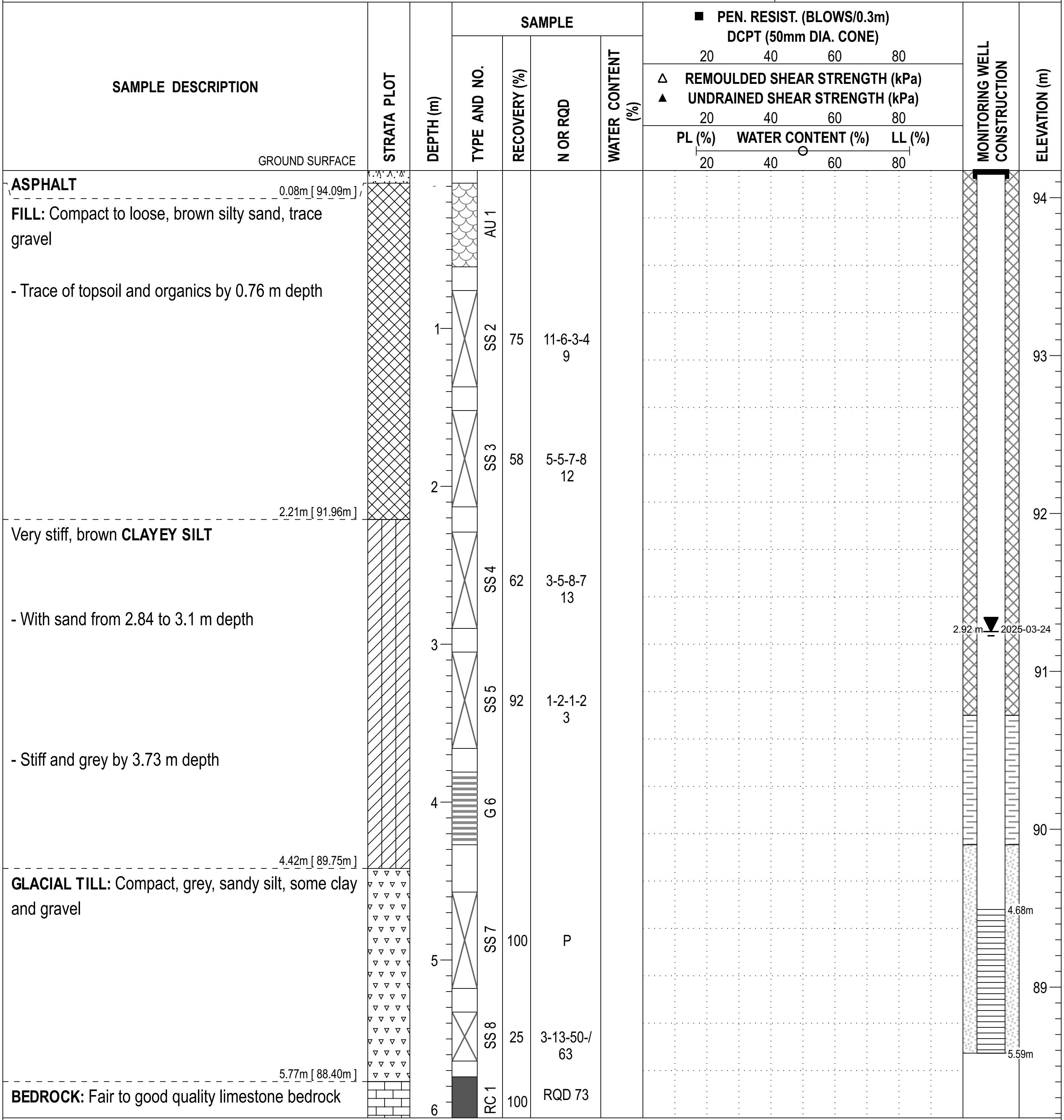
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 7, 2025

FILE NO. : PG7044

HOLE NO. : BH 6-25



SAMPLE DESCRIPTION		TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE					
ASPHALT	0.08m [94.09m]				
FILL: Compact to loose, brown silty sand, trace gravel - Trace of topsoil and organics by 0.76 m depth		AU 1			
		SS 2	75	11-6-3-4 9	
	2.21m [91.96m]	SS 3	58	5-5-7-8 12	
Very stiff, brown **CLAYEY SILT** - With sand from 2.84 to 3.1 m depth		SS 4	62	3-5-8-7 13	
		SS 5	92	1-2-1-2 3	
- Stiff and grey by 3.73 m depth	4.42m [89.75m]	G 6			
GLACIAL TILL: Compact, grey, sandy silt, some clay and gravel		SS 7	100	P	
	5.77m [88.40m]	SS 8	25	3-13-50-/ 63	
BEDROCK: Fair to good quality limestone bedrock		RC 1	100	RQD 73	

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365207.11 **NORTHING:** 5025197.47 **ELEVATION:** 94.17

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

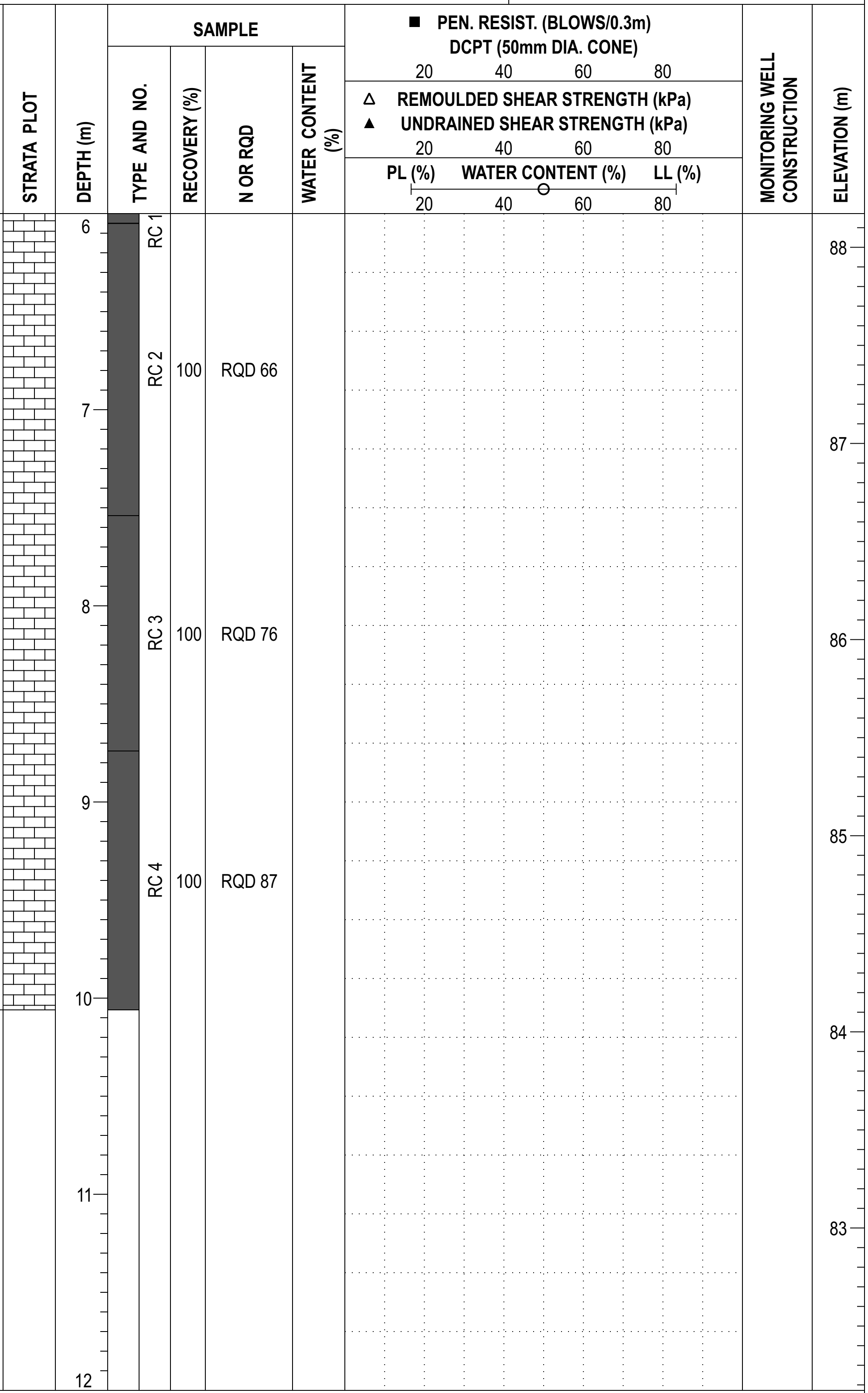
REMARKS:

DATE: March 7, 2025

FILE NO. : PG7044

HOLE NO. : BH 6-25

SAMPLE DESCRIPTION	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
	RC 1			
	RC 2	100	RQD 66	
- Vertical fracture from 7.19 to 7.39 m depth				
	RC 3	100	RQD 76	
- Thin mud seams at 8.36 m depth				
	RC 4	100	RQD 87	
10.06m [84.11m]				
End of Borehole (GWL at 2.92 m depth - March 24, 2025)				



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365155.71 **NORTHING:** 5025197.30 **ELEVATION:** 94.46

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

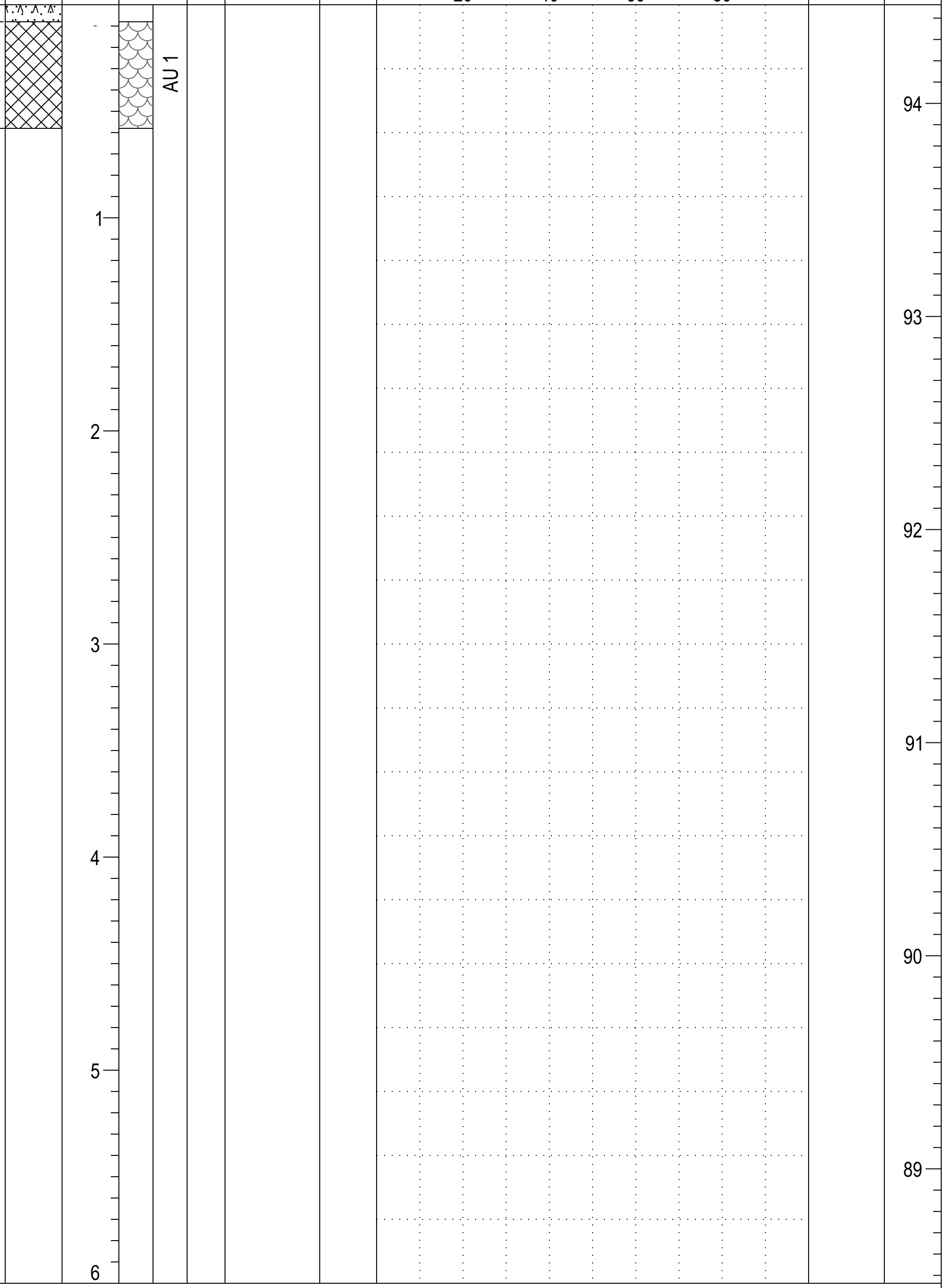
REMARKS:

DATE: March 7, 2025

FILE NO. : PG7044

HOLE NO. : BH 7-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
ASPHALT 0.08m [94.38m]									
FILL: Compact, brown silty sand, some gravel 0.58m [93.88m]			AU 1						
End of Borehole Practical refusal to augering at 0.58 m depth									



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 365153.71 **NORTHING:** 5025197.04 **ELEVATION:** 94.48

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 7, 2025

FILE NO. : PG7044

HOLE NO. : BH 7A-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE									
OVERBURDEN 0.64m [93.84m]									94
End of Borehole Practical refusal to augering at 0.64 m depth		1							93
		2							92
		3							91
		4							90
		5							89
		6							

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365127.88 **NORTHING:** 5025329.42 **ELEVATION:** 96.89

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 7, 2025

FILE NO. : PG7044

HOLE NO. : BH 8-25

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE						
ASPHALT	0.08m					[96.81m]
FILL: Compact, brown silty sand with gravel and cobbles		AU 1				
		SS 2	0	50-/-/-/ 50/0.02		
	0.81m					[96.08m]
End of Borehole Practical refusal to augering at 0.81 m depth						

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

△ REMOULDED SHEAR STRENGTH (kPa)

▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

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

PIEZOMETER CONSTRUCTION

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365126.12 **NORTHING:** 5025328.66 **ELEVATION:** 96.89

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 7, 2025

FILE NO. : PG7044

HOLE NO. : BH 8A-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	PIEZOMETER CONSTRUCTION	ELEVATION (m)
GROUND SURFACE								
ASPHALT 0.08m [96.81m]								
FILL: Compact, brown silty sand with gravel 0.69m [96.20m]								96
TOPSOIL 1.32m [95.57m]		1	SS 1	50	4-2-1-1 3			
End of Borehole Practical refusal to augering at 1.32 m depth		2						95
		3						94
		4						93
		5						92
		6						91

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

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa)
20 40 60 80

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365120.82 **NORTHING:** 5025244.83 **ELEVATION:** 95.39

PROJECT: Proposed Development

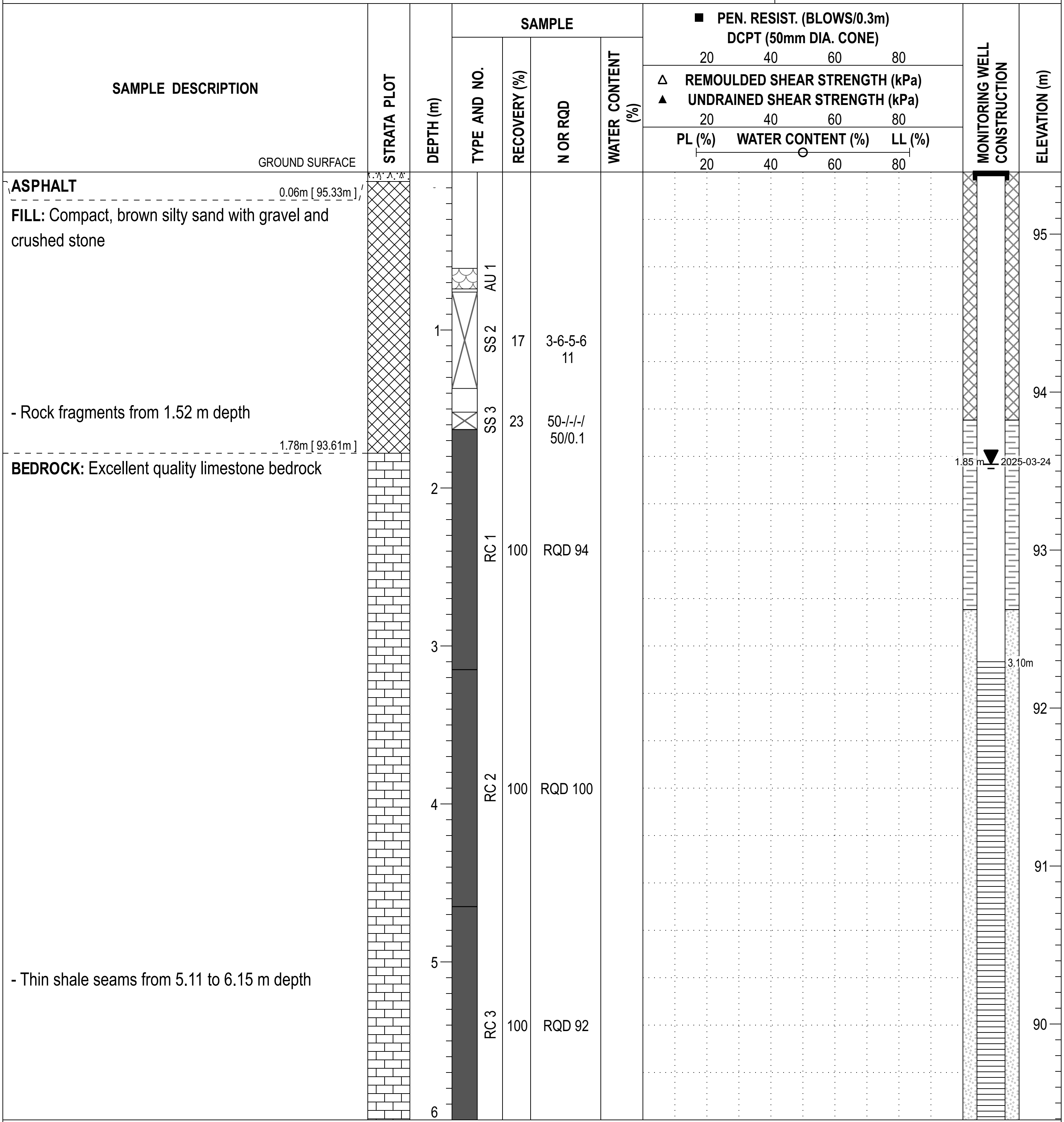
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 8, 2025

FILE NO. : PG7044

HOLE NO. : BH 9-25



ASPHALT 0.06m [95.33m]

FILL: Compact, brown silty sand with gravel and crushed stone

- Rock fragments from 1.52 m depth

1.78m [93.61m]

BEDROCK: Excellent quality limestone bedrock

- Thin shale seams from 5.11 to 6.15 m depth

TYPE AND NO.	RECOVERY (%)	N OR RQD
AU 1		
SS 2	17	3-6-5-6 11
SS 3	23	50-/-/- 50/0.1
RC 1	100	RQD 94
RC 2	100	RQD 100
RC 3	100	RQD 92

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365120.82 **NORTHING:** 5025244.83 **ELEVATION:** 95.39

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

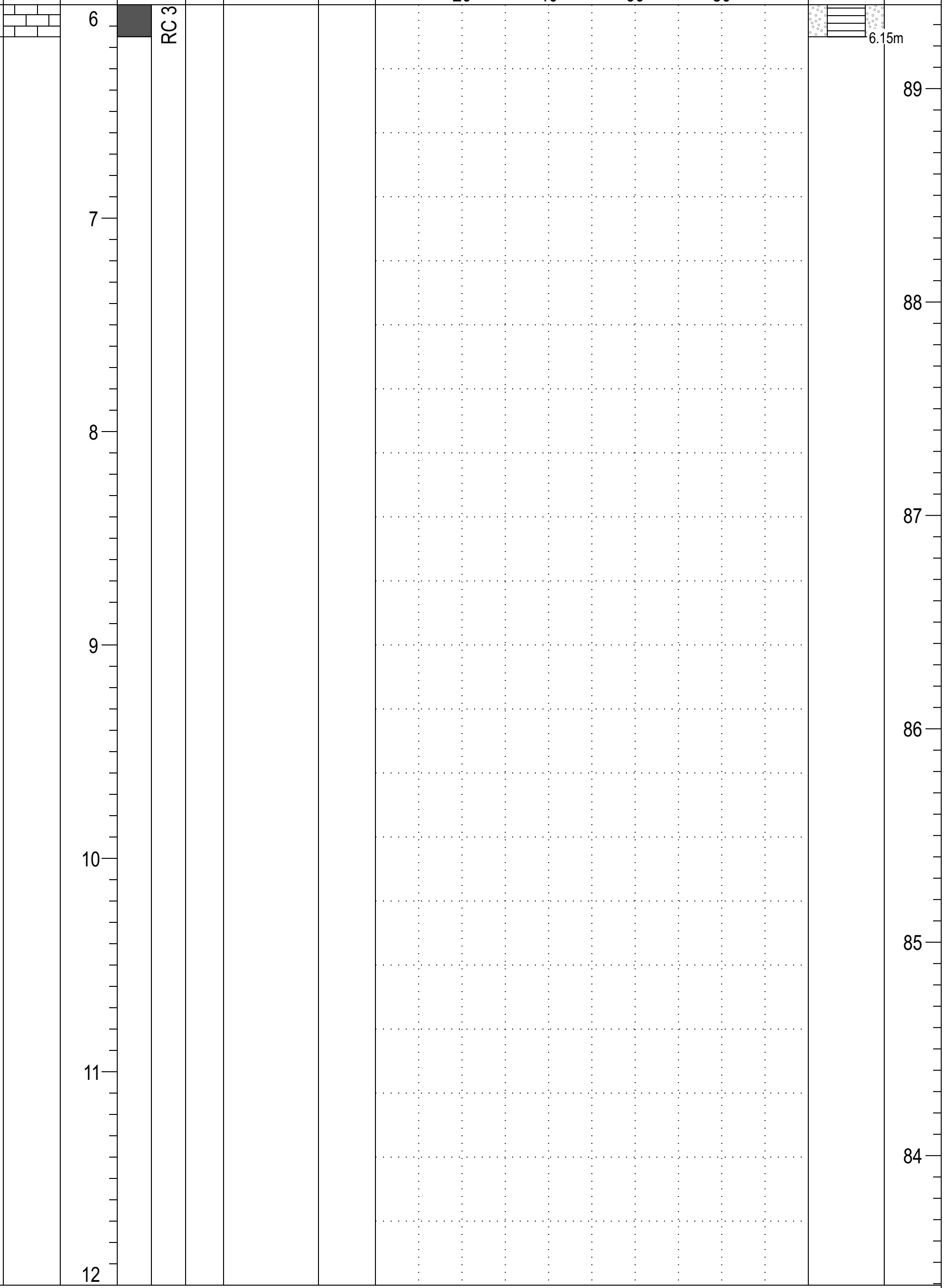
REMARKS:

DATE: March 8, 2025

FILE NO. : PG7044

HOLE NO. : BH 9-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
6.15m [89.24m]		6	RC 3					6.15m	
End of Borehole (GWL at 1.85 m depth - March 24, 2025)									



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365146.36 **NORTHING:** 5025219.02 **ELEVATION:** 95.32

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

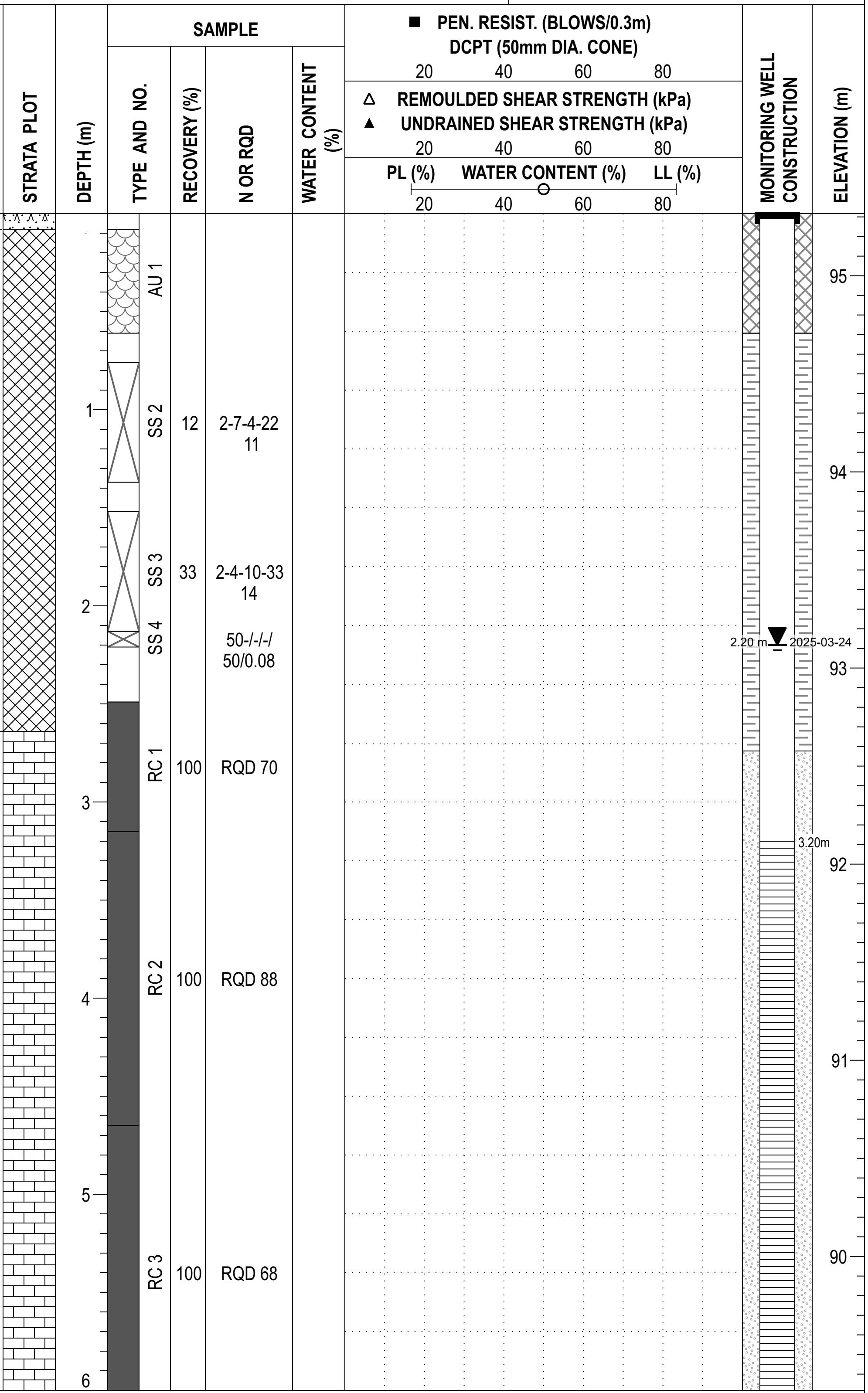
REMARKS:

DATE: March 8, 2025

FILE NO. : PG7044

HOLE NO. : BH10-25

SAMPLE DESCRIPTION	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE				
ASPHALT 0.08m [95.24m]				
FILL: Compact, brown silty sand with gravel and crushed stone	AU 1			
	SS 2	12	2-7-4-22 11	
- Trace clay and organics by 1.52 m depth	SS 3	33	2-4-10-33 14	
	SS 4		50-/-/-/ 50/0.08	
2.64m [92.68m]				
BEDROCK: Fair to good quality limestone bedrock	RC 1	100	RQD 70	
	RC 2	100	RQD 88	
	RC 3	100	RQD 68	
- Poor quality bedrock between 5.66 and 6.29 m depth				



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365146.36 **NORTHING:** 5025219.02 **ELEVATION:** 95.32

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 8, 2025

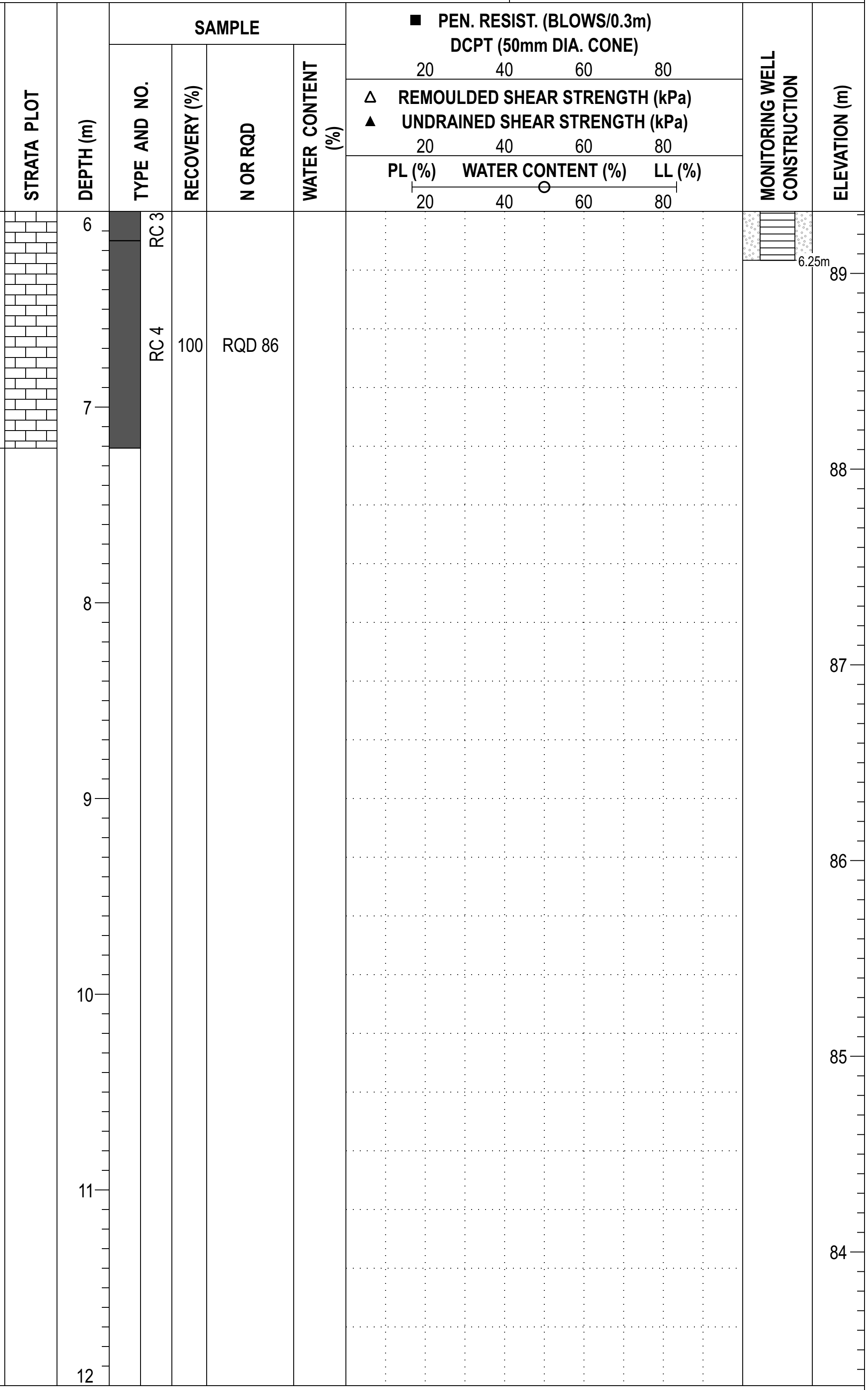
FILE NO. : PG7044

HOLE NO. : BH10-25

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
	6	RC 3			
		RC 4	100	RQD 86	
7.21m [88.11m]					

End of Borehole

(GWL at 2.20 m depth - March 24, 2025)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365097.00 **NORTHING:** 5025310.71 **ELEVATION:** 96.94

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

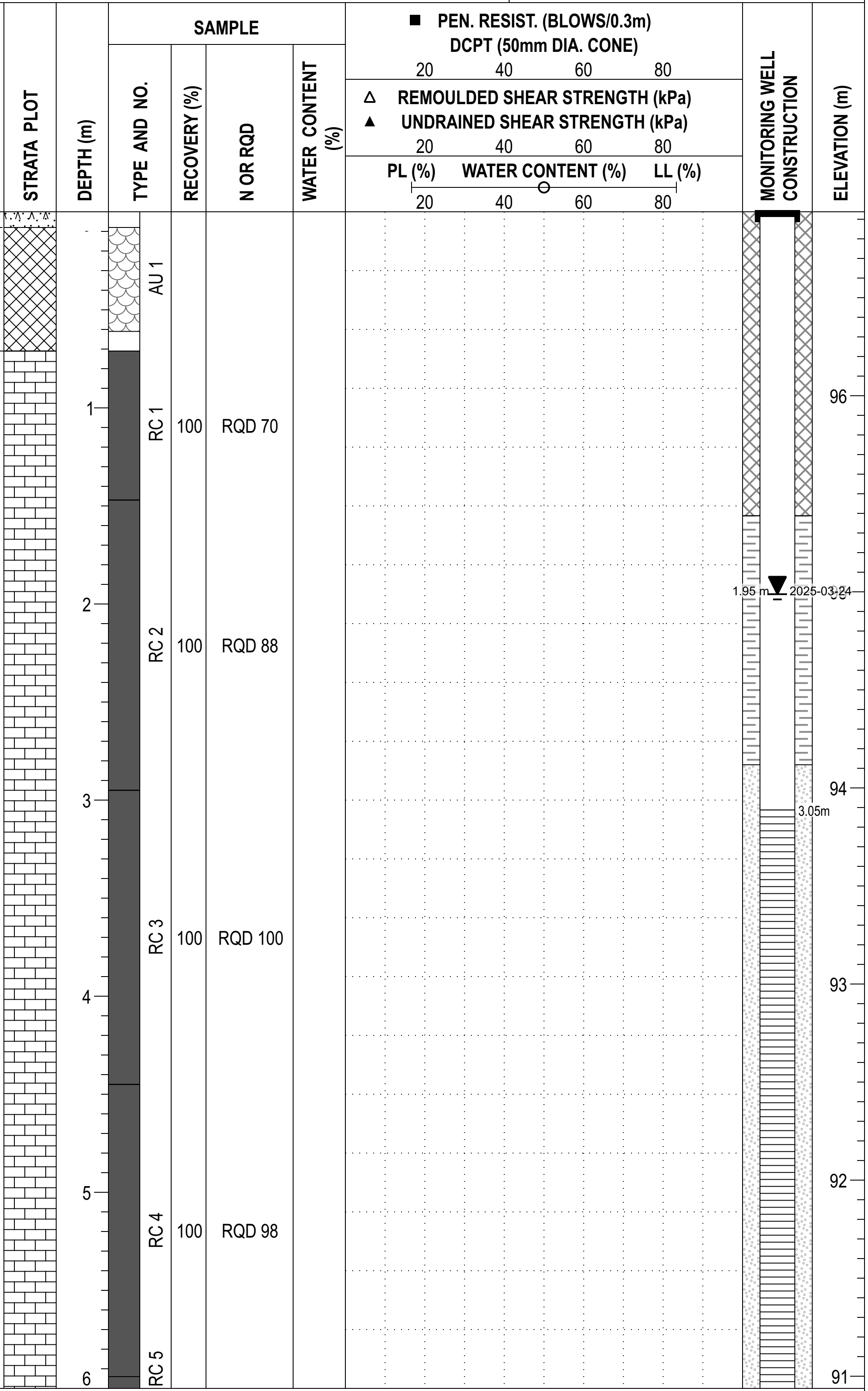
REMARKS:

DATE: March 9, 2025

FILE NO. : PG7044

HOLE NO. : BH11-25

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE					
ASPHALT 0.08m [96.86m]					
FILL: Compact, brown silty sand with gravel and crushed stone 0.71m [96.23m]		AU 1			
BEDROCK: Fair to excellent quality limestone bedrock	1	RC 1	100	RQD 70	
	2	RC 2	100	RQD 88	
	3	RC 3	100	RQD 100	
	4				
	5	RC 4	100	RQD 98	
	6	RC 5			



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365097.00 **NORTHING:** 5025310.71 **ELEVATION:** 96.94

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 9, 2025

FILE NO. : PG7044

HOLE NO. : BH11-25

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	MONITORING WELL CONSTRUCTION	ELEVATION (m)
- Thin shale seams from 6.25 to 6.55 m depth	6	RC 5	100	RQD 92		6.10m	
7.21m [89.73m]	7						90
End of Borehole (GWL at 1.95 m depth - March 24, 2025)	8						89
	9						88
	10						87
	11						86
	12						85

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

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa)
20 40 60 80

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

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365182.38 **NORTHING:** 5025253.25 **ELEVATION:** 95.30

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 9, 2025

FILE NO. : PG7044

HOLE NO. : BH12-25

SAMPLE DESCRIPTION

GROUND SURFACE

ASPHALT 0.08m [95.22m]

FILL: Compact, brown silty sand with gravel and crushed stone, trace clay and topsoil

1.52m [93.78m]

BEDROCK: Fair to excellent quality limestone bedrock

- Thin mud seams at 1.73 m and 2.9 m depths

DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
0–0.53	AU 1			
0.76–1.32	SS 2	47	7-7-12-50 19/0.08	
1.32–3.05	RC 1	100	RQD 72	
3.05–4.57	RC 2	100	RQD 87	
4.57–6	RC 3	100	RQD 93	

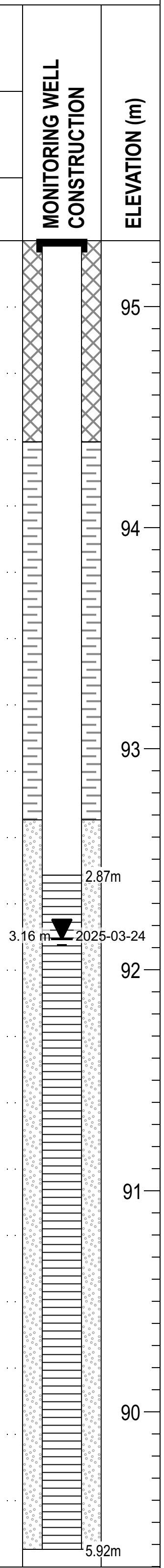
DEPTH (m): 1, 2, 3, 4, 5, 6

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

△ REMOULDED SHEAR STRENGTH (kPa)

▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

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



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COORD. SYS.: MTM ZONE 9 **EASTING:** 365182.38 **NORTHING:** 5025253.25 **ELEVATION:** 95.30

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 9, 2025

FILE NO. : PG7044

HOLE NO. : BH12-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	MONITORING WELL CONSTRUCTION	ELEVATION (m)
		6	RC 3						
- Thin shale seams from 6.58 to 6.86 m depth			RC 4	100	RQD 86				89
7.16m [88.14m]		7							
End of Borehole (GWL at 3.16 m depth - March 24, 2025)									88
		8							87
		9							86
		10							85
		11							84
		12							

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365133.67 **NORTHING:** 5025253.37 **ELEVATION:** 95.46

PROJECT: Proposed Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 15, 2025

FILE NO. : PG7044

HOLE NO. : BH13-25

SAMPLE DESCRIPTION

GROUND SURFACE

CONCRETE 0.18m [95.28m]

FILL: Gravel, trace silty sand 0.28m [95.18m]

TOPSOIL: with brown, silty sand, trace gravel 0.61m [94.85m]

GLACIAL TILL: Grey, silty clay with sand, gravel, cobbles and boulders 0.89m [94.57m]

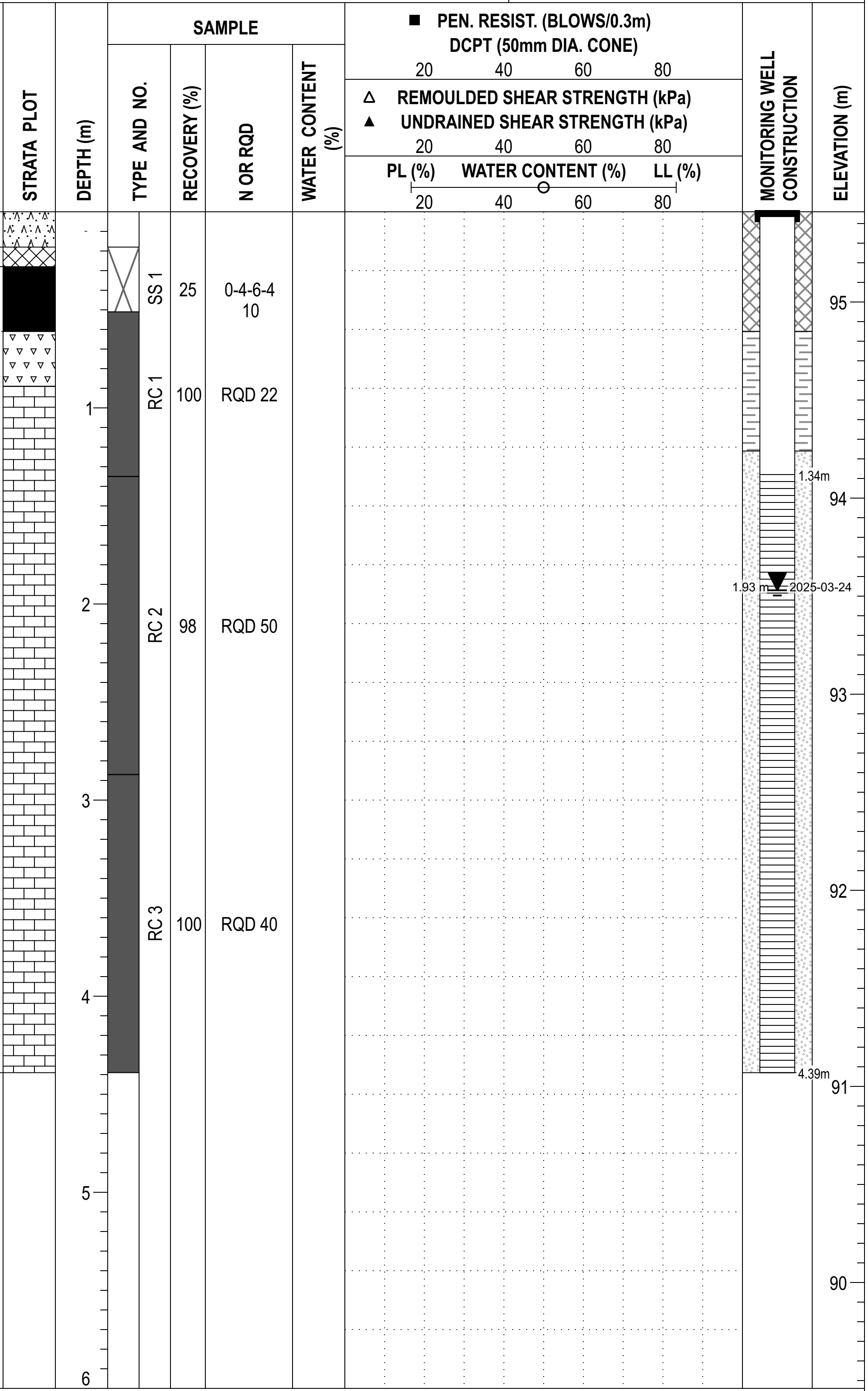
BEDROCK: Very poor to poor quality grey limestone bedrock

- 40 mm mud seams at 2.41 m depth

4.39m [91.07m]

End of Borehole

(GWL at 1.93 m depth - March 24, 2025)



TYPE AND NO.	RECOVERY (%)	N OR RQD
SS 1	25	0-4-6-4 10
RC 1	100	RQD 22
RC 2	98	RQD 50
RC 3	100	RQD 40

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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365136.99 **NORTHING:** 5025236.49 **ELEVATION:** 95.46

PROJECT: Proposed Development

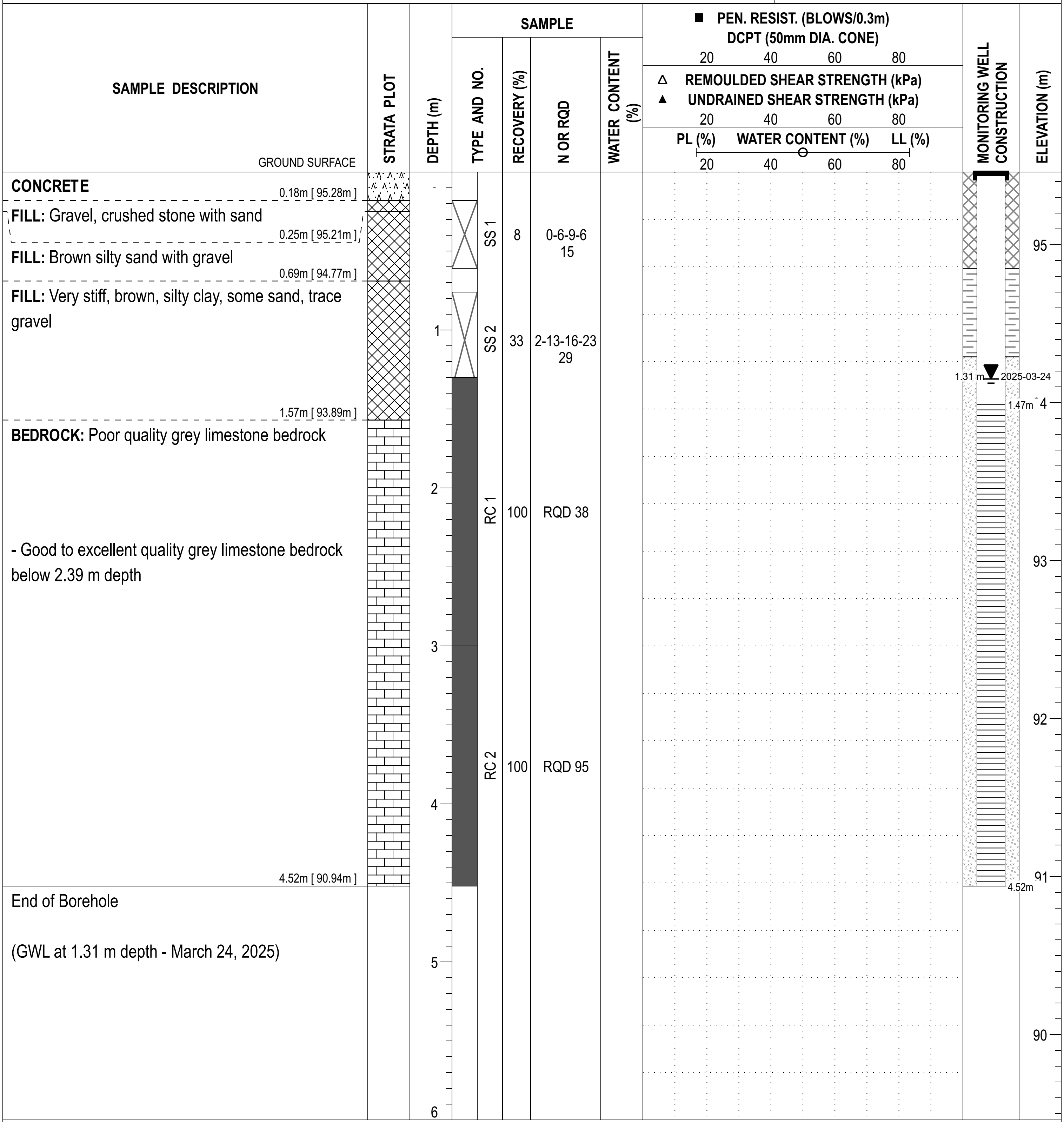
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 16, 2025

FILE NO. : PG7044

HOLE NO. : BH14-25



SAMPLE DESCRIPTION	Depth [Elevation]
GROUND SURFACE	
CONCRETE	0.18m [95.28m]
FILL: Gravel, crushed stone with sand	0.25m [95.21m]
FILL: Brown silty sand with gravel	0.69m [94.77m]
FILL: Very stiff, brown, silty clay, some sand, trace gravel	1.57m [93.89m]
BEDROCK: Poor quality grey limestone bedrock - Good to excellent quality grey limestone bedrock below 2.39 m depth	4.52m [90.94m]

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	8	0-6-9-6 15	
SS 2	33	2-13-16-23 29	
RC 1	100	RQD 38	
RC 2	100	RQD 95	

End of Borehole

(GWL at 1.31 m depth - March 24, 2025)

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365155.91 **NORTHING:** 5025252.30 **ELEVATION:** 95.46

PROJECT: Proposed Development

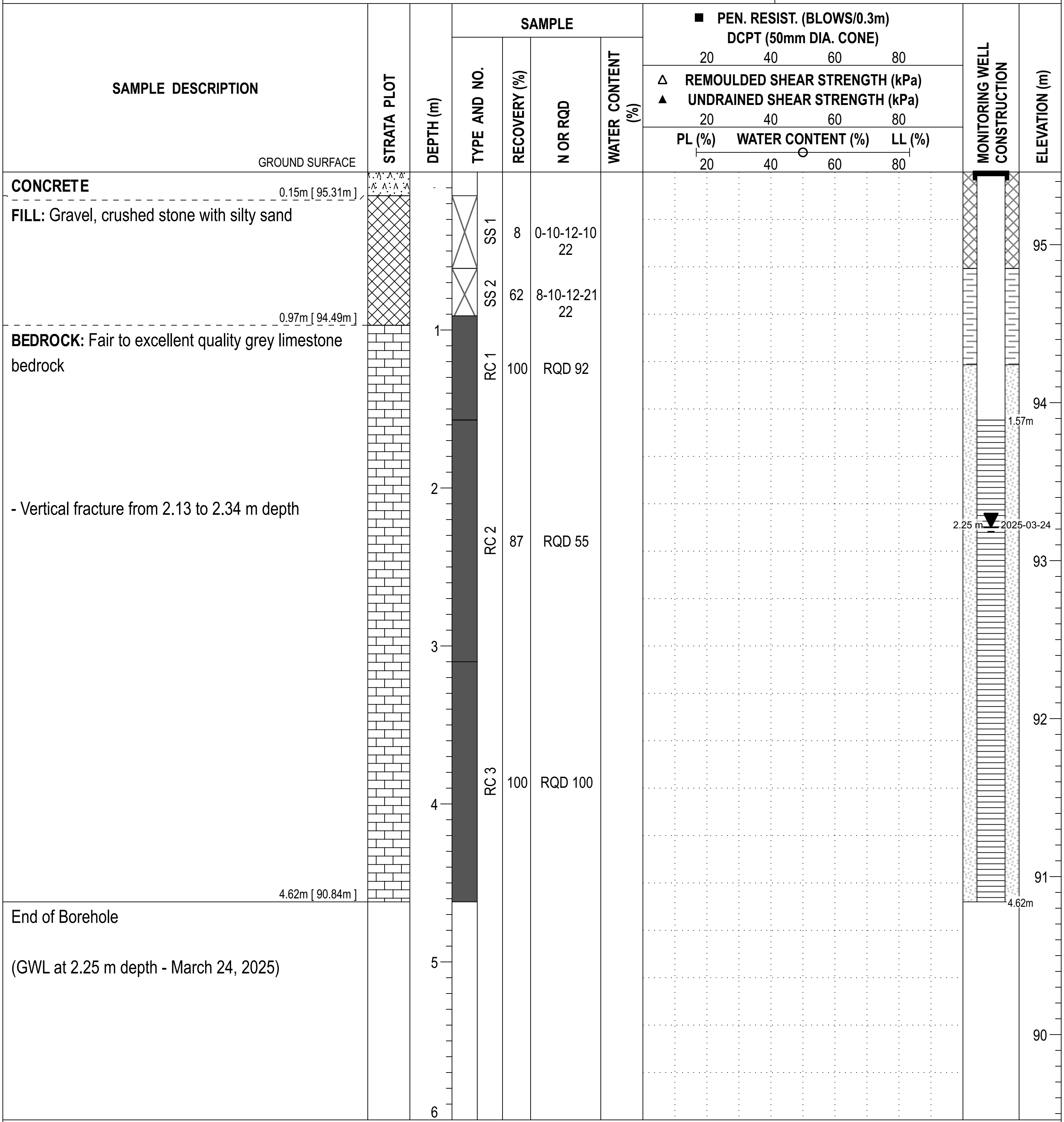
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 16, 2025

FILE NO. : PG7044

HOLE NO. : BH15-25



SAMPLE DESCRIPTION

GROUND SURFACE

CONCRETE 0.15m [95.31m]

FILL: Gravel, crushed stone with silty sand 0.97m [94.49m]

BEDROCK: Fair to excellent quality grey limestone bedrock

- Vertical fracture from 2.13 to 2.34 m depth

4.62m [90.84m]

End of Borehole

(GWL at 2.25 m depth - March 24, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	8	0-10-12-10 22	
SS 2	62	8-10-12-21 22	
RC 1	100	RQD 92	
RC 2	87	RQD 55	
RC 3	100	RQD 100	

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1200 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 365167.51 **NORTHING:** 5025233.03 **ELEVATION:** 95.46

PROJECT: Proposed Development

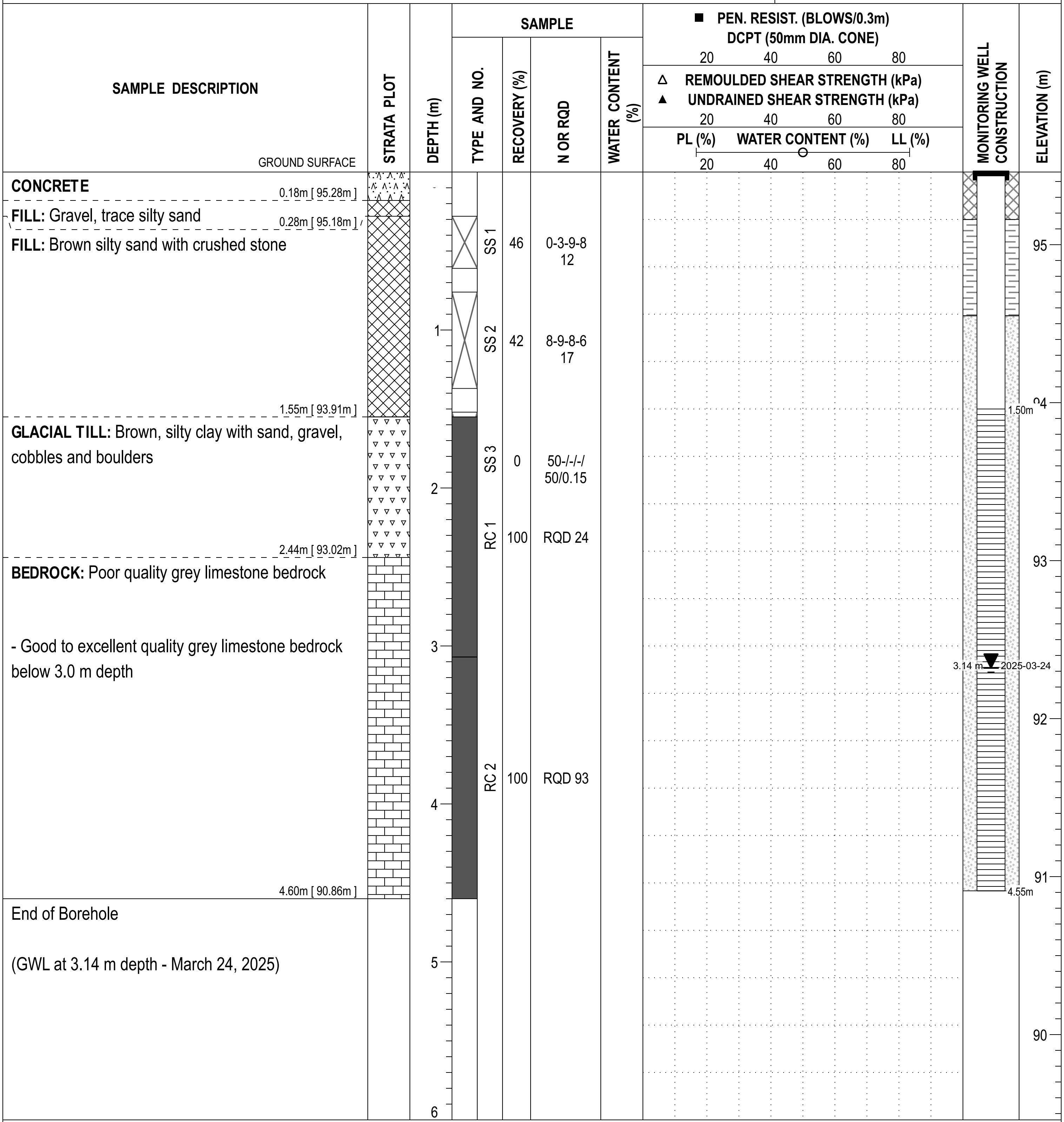
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 16, 2025

FILE NO. : PG7044

HOLE NO. : BH16-25



SAMPLE DESCRIPTION	
GROUND SURFACE	
CONCRETE	0.18m [95.28m]
FILL: Gravel, trace silty sand	0.28m [95.18m]
FILL: Brown silty sand with crushed stone	1.55m [93.91m]
GLACIAL TILL: Brown, silty clay with sand, gravel, cobbles and boulders	2.44m [93.02m]
BEDROCK: Poor quality grey limestone bedrock - Good to excellent quality grey limestone bedrock below 3.0 m depth	4.60m [90.86m]
End of Borehole (GWL at 3.14 m depth - March 24, 2025)	

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	46	0-3-9-8 12	
SS 2	42	8-9-8-6 17	
SS 3	0	50-/-/-/ 50/0.15	
RC 1	100	RQD 24	
RC 2	100	RQD 93	

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

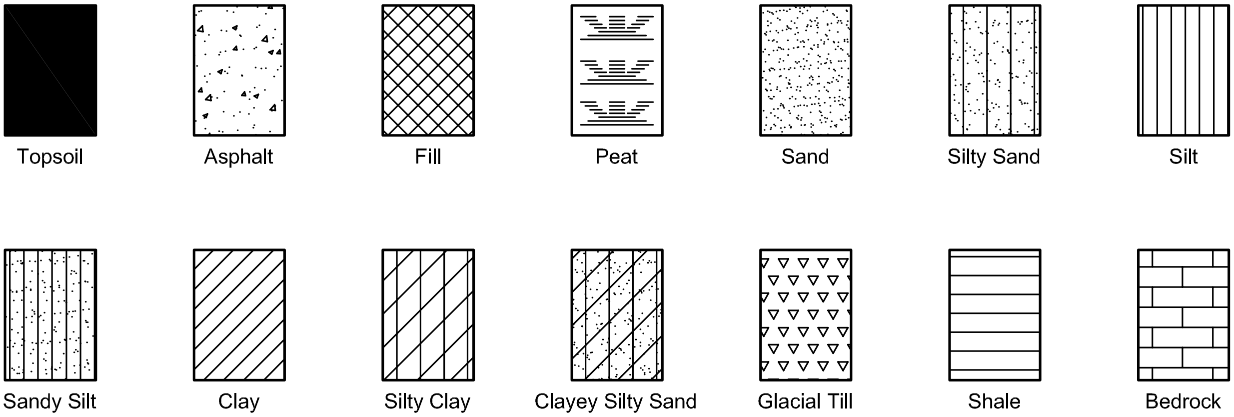
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

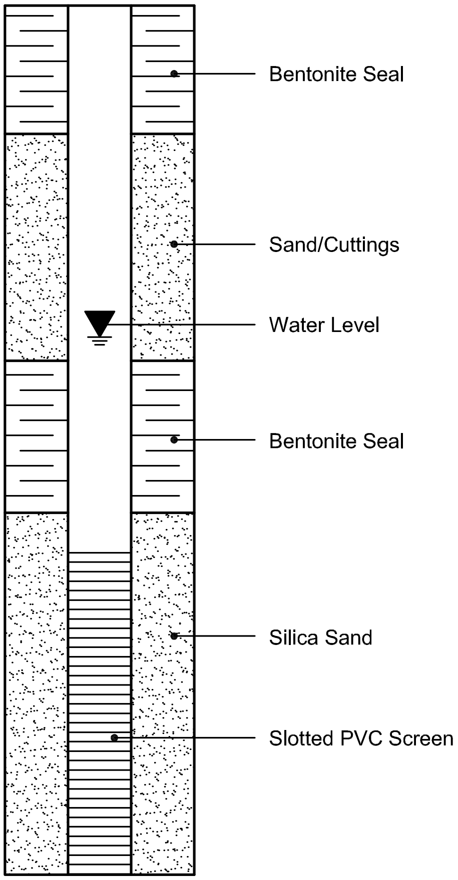
SYMBOLS AND TERMS (continued)

STRATA PLOT

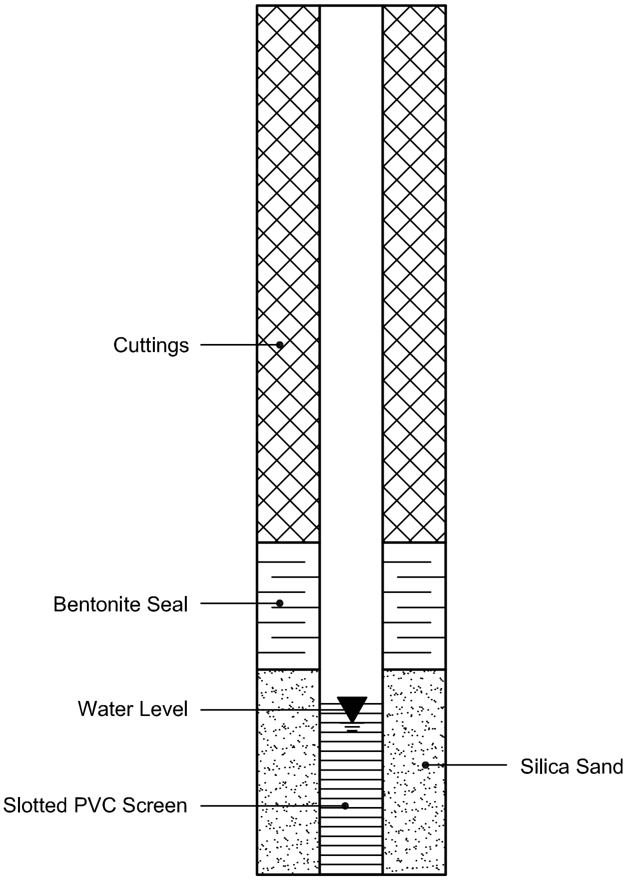


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 18-Mar-2025

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

Order Date: 12-Mar-2025

Client PO: 62582

Project Description: PG7044

	Client ID:	BH6-25 SS4	-	-	-	-	-
	Sample Date:	07-Mar-25 09:00	-	-	-		
	Sample ID:	2511352-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	71.4	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.63	-	-	-	-	-
Resistivity	0.1 Ohm.m	4.5	-	-	-	-	-
Anions							
Chloride	10 ug/g	1190	-	-	-	-	-
Sulphate	10 ug/g	322	-	-	-	-	-

APPENDIX 2

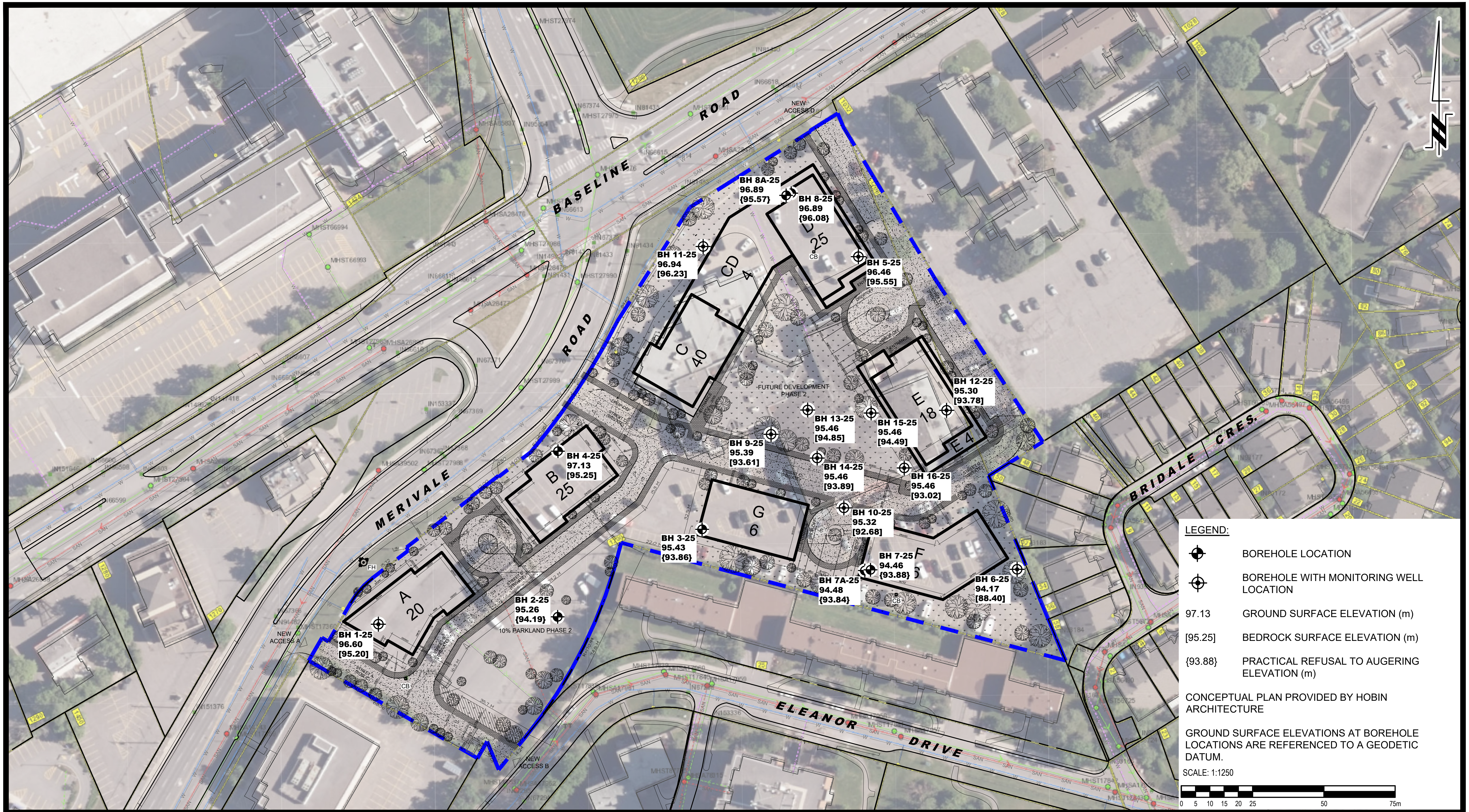
FIGURE 1 – KEY PLAN

DRAWING PG7044-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON
GROUP



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
97.13	GROUND SURFACE ELEVATION (m)
[95.25]	BEDROCK SURFACE ELEVATION (m)
{93.88}	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY HOBIN ARCHITECTURE

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

SCALE: 1:1250

0 5 10 15 20 25 50 75m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

ZENA INVESTMENT CORPORATION
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
1200 BASELINE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1250
Date: 03/2025
Drawn by: YA
Report No.: PG0744-1
Checked by: KP
Dwg. No.: PG7044-1
Approved by: SD
Revision No.:

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11x17