

Geotechnical Investigation

Proposed Building Addition

1981 Century Road
Ottawa, Ontario

Prepared for Brunstad Christian Church Ottawa

Report PG6727-1 Revision 3 dated August 24, 2026

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1.0 Introduction

Paterson Group (Paterson) was commissioned by the Brunstad Christian Church Ottawa to conduct a geotechnical investigation for the proposed addition to the existing church to be located at 1981 Century 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 the design of the proposed development, including construction considerations which may affect the design.

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

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 available drawings, it is understood that the proposed building addition will be located at the east side of the existing church hall, and will consist of a 1- to 2-storey structure with an approximate footprint of 2,500 m². A portion of the proposed building will also include a basement level which will be used primarily as storage and mechanical space.

The proposed building addition will be immediately surrounded by interlock pathways and landscaped areas. It is further understood that the proposed building addition will be serviced by a private well and septic system.

A new asphalt-paved parking area is proposed to the north of the existing parking lot. A proposed dry stormwater management (SWM) facility is proposed to the north of the proposed building addition's septic system, and east of the new parking area.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program was completed on July 17, 2023. A total of 4 boreholes were advanced to a maximum depth of 6.7 m below the existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the proposed building addition and taking into consideration underground utilities and site features.

In addition, a supplemental field program was conducted on August 10, 2026. At that time, a total of 4 boreholes were advanced to a maximum depth of 6.7 m below the existing ground to fill in gaps between the boreholes completed in 2023 within the proposed building footprint.

The borehole locations are shown on Drawing PG6727-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track-mounted auger 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 testing procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden soils.

Sampling and In Situ Testing

The borehole samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, 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, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The thickness of the overburden soils was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at borehole BH 4-23. 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.

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

During the previous investigation in 2023, 3 boreholes were fitted with flexible standpipe piezometers and 1 borehole was equipped with a monitoring well to facilitate monitoring of groundwater levels subsequent to the completion of the sampling program. A groundwater monitoring program was also conducted at borehole BH 3-23 from February to August 2024 using a data logger.

In addition, during the recent investigation in 2026, 4 boreholes were fitted with flexible standpipe piezometers.

The groundwater observations are discussed in Section 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1, and in the Groundwater Monitoring Program memo in Appendix 3.

Sample Storage

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

3.2 Field Survey

The borehole locations and the ground surface elevations at the borehole locations were surveyed using a high-precision handheld GPS and are referenced to a geodetic datum. The locations of the boreholes are presented on Drawing PG6727-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of 3 samples were submitted for Atterberg limits testing, 1 sample was submitted for grain size distribution analysis, and 1 sample was submitted for shrinkage testing. In addition, all recovered samples were submitted for moisture content testing.

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 attack against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the sample. 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 the existing Brunstad Christian Church building in the south-central portion of the site, which is immediately surrounded by asphalt-paved access lanes to the south, an asphalt-paved parking area to the west, and landscaped areas to the north and east.

The site is bordered by Century Road to the south, agricultural land to the east and north, and undeveloped land to the west. The ground surface across the subject site is relatively level at approximate geodetic elevations ranging from 91.8 to 92.7 m.

4.2 Subsurface Profile

Generally, the subsurface profile at the borehole locations consists of an approximate 0.3 m thickness of topsoil underlain by successive deposits of clayey silt to silty clay, silty sand, and glacial till. It should also be noted that an approximate 0.4 m thickness of fill was encountered underlying the topsoil at boreholes BH 1-26 to BH 4-26, and BH 1-23.

The very stiff to soft, brown to grey clayey silt to silty clay deposit was encountered underlying the topsoil and/or fill, extending to depths of about 1.2 to 4 m below the existing ground surface in boreholes BH 1-26, BH 4-26, and BH 1-23 to BH 4-23. In boreholes BH 2-26 and BH 3-26, an undisturbed compact to loose silt layer with trace sand and clay was observed below the above-noted fill layer to a depth ranging from 1.9 to 2.4 m below the existing ground surface.

A very loose to loose, grey silty sand was encountered underlying the clayey silt to silty clay, with an approximate thickness of 0.7 m in BH 1-23, BH 2-23, and BH 4-23.

A glacial till deposit was encountered underlying the above noted silty sand or silt or clayey silt to silty clay layers, consisting of a loose to compact, brown to grey silty sand to sandy silt with varying amounts of gravel, clay, cobbles, and boulders. The density of the glacial till was generally increasing with depth, although there were some fluctuations which were likely due to higher gravel content at certain localized depths.

It would generally be expected that a non-cohesive soil deposit, such as the glacial till, would have its density generally increasing with depth due to the increasing overburden stress.

Practical refusal to augering was encountered at an approximate depth of 5.9 m below the existing ground surface in BH 1-26. In addition, Practical refusal to the DCPT was encountered at an approximate depth of 10.8 m in borehole BH 4-23.

Bedrock

Based on available geological mapping, the bedrock in the subject area is part of the Oxford Formation, which consists of dolomite with an overburden drift thickness ranging from 5 to 10 m.

Atterberg Limits Testing

Atterberg limits testing was completed on the recovered silty clay samples at select borehole locations from the investigation completed in 2023. The results of the Atterberg limits test are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1. The tested silty clay samples are classified as inorganic clays of low plasticity (CL) or inorganic clays of high plasticity (CH) in accordance with the Unified Soil Classification System.

Table 1 – Atterberg Limits Results						
Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
BH 1-23-SS3	1.83	29	19	10	23	CL
BH 3-26-SS4	1.83	33	18	15	22	CL
BH 4-26-SS4	1.83	61	22	39	54	CH
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content; CL: Inorganic Clays of Low Plasticity; CH: Inorganic Clays of High Plasticity						

Grain Size Distribution and Hydrometer Testing

Grain size distribution and hydrometer analysis was completed on one selected recovered sample from the investigation completed in 2023. The results of the grain size distribution and hydrometer analysis are presented in Table 2 and on the Grain Size Distribution sheets in Appendix 1.

Table 2 – Grain Size Distribution and Hydrometer Results

Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 1-23-SS4	2.59	0.0	0.1	53.9	46.0

Shrinkage Testing

Linear shrinkage testing was completed on one sample recovered at a depth of 1.07 from BH 3-23 from the previous investigation completed in 2023. The shrinkage limit and shrinkage ratio of the tested clayey silt sample were found to be 23.13% and 1.666, respectively.

The results of the linear shrinkage testing are presented in Appendix 1.

4.3 Groundwater

Groundwater levels were recorded at each test hole location and are presented in Table 3 below on August 18, 2026, and July 24, 2026. The groundwater level readings are also presented in the Soil Profile and Test Data sheets in Appendix 1.

Table 3 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-26	92.36	0.53	91.83	August 18, 2026
BH 2-26	92.25	0.25	92	
BH 3-26	92.20	1.00	91.2	
BH 4-26	92.62	5.55	87.07	
BH1-23	92.70	0.82	91.88	
BH2-23	92.10	0.21	91.89	
BH 1-23	92.70	1.78	90.92	July 24, 2023
BH 2-23	92.10	1.99	90.11	
BH 3-23*	91.87	1.10	90.77	
BH 4-23	92.03	1.25	90.78	
Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS and referenced to a geodetic datum.				
* - Boreholes with monitoring wells.				

The groundwater monitoring program also encountered groundwater at borehole BH 3-23 ranging from depths of 0.03 to 0.97 m below the existing ground surface. Based on the site observations, the long-term groundwater level is estimated to range between approximate geodetic elevations of **90.0 to 91.8 m**. However, it should 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 suitable for the proposed development. It is recommended that foundation support for the proposed building addition consist of one of the following:

- ☐ Helical piles which have all bearing plates embedded within the undisturbed, compact to dense glacial till, or
- ☐ Conventional spread footings bearing on the undisturbed, compact to dense glacial till. Where this option is used for slab-on-grade areas of the proposed building, it is expected that sub-excavation will be required to reach the undisturbed, compact to dense glacial till bearing stratum, followed by the placement of lean concrete from this bearing stratum up to the underside of footing elevation (USF).

Due to the presence of the clayey silt to silty clay deposit, a permissible grade raise restriction has been provided for the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Fill Placement

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

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in 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.

Lean Concrete Filled Trenches

Unless helical piles are used for foundation support of the slab-on-grade areas of the proposed building addition, zero-entry vertical trenches should be excavated at the footing locations down to the undisturbed, compact to dense glacial till and then backfilled with lean concrete up to the founding elevation (minimum 17 MPa 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying undisturbed, compact to dense glacial till.

An experienced inspector will observe the effort required by the excavation equipment during the excavation to determine that any disturbed and/or loose glacial till has been removed down to a suitable bearing surface consisting of the undisturbed, compact to dense glacial till. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Conventional Spread Footings

Conventional spread footings bearing directly on the undisturbed, compact to dense glacial till, or supported on lean concrete trenches which are placed over the undisturbed, compact to dense glacial till, can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. The bearing resistance values for footings were calculated using conventional bearing capacity methods, including Peck et al. (1974), which correspond to maximum post-construction total and differential settlements of 25 mm and 20 mm, respectively, when applying the appropriate factors of safety.

An undisturbed soil bearing surface consists of a surface 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.

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 glacial till bearing surface when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1.5H:1V (or shallower) passes through in situ soil of the same or higher capacity as that of the bearing medium.

Helical Piles

As noted above in Section 5.1, helical piles may also be used as foundation support of the proposed building addition.

The helical piles should penetrate the fill, clayey silt to silty clay, and very loose to loose silty sand, and derive their support with all bearing plates fully embedded in the undisturbed, compact to dense glacial till deposit. It is recommended that each helical pile consist of a 75 mm diameter, steel pipe shaft with a 6.6 mm wall thickness and a triple-helix lead section with 200 mm, 250 mm, and 300 mm diameter helical bearing plates.

Helical piles installed in accordance with the recommendations provided above are considered suitable to provide compression capacities of **100 kN** under serviceability limit states (SLS) conditions and **150 kN** under ultimate limit states (ULS) conditions.

Helical piles designed using the above-noted capacities at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. The capacities of the helical piles were calculated using conventional bearing capacity methods provided in the textbook entitled “Helical Piles: A Practical Guide to Design and Installation” by Howard A. Perko, Ph.D., P.E. (2009), which correspond to maximum post-construction total and differential settlements of 25 mm of 20 mm, respectively, when applying the appropriate resistance factors.

To achieve the capacities indicated above, each helical pile should be installed to a minimum final installation torque of 8,500 ft-lbs. This installation torque was calculated using the correlations in the textbook entitled “Helical Piles: A Practical Guide to Design and Installation” by Howard A. Perko, Ph.D., P.E. (2009). Alternate helical pile configurations to those provided above may be utilized subject to the review and approval of Paterson.

Paterson will make continuous inspections, as required under Section 4.2.2.3.(2) of Division B of the Ontario Building Code (OBC) 2024 for deep foundation units, where the torque will be monitored and recorded from a torque gauge connected to the installation equipment.

A square, steel top plate will be bolted to the top of each helical pile, which will extend up into the rebar of each pile cap and/or grade beam. The specific connection detail will be designed by the structural engineer.

Permissible Grade Raise Restrictions

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **1.5 m** is recommended for grading 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

Portions of the clayey silt to silty clay deposit and the very loose to loose silty sand deposit at this site are considered potentially susceptible to cyclic straining and liquefaction, respectively. However, the foundation design recommendations provided herein specify that the foundation loads of the proposed building addition are to be transferred through these deposits using lean concrete trenches or helical piles, to the underlying compact to dense glacial till deposit which is not considered potentially susceptible to cyclic straining for liquefaction.

Therefore, provided the foundation design recommendations are followed, the site class for seismic site response can be taken as **Class X_D**. The seismic site class was determined by calculating the average standard penetration resistance (N₆₀) for cohesionless soil layers and the average undrained shear strength (su) for cohesive soil layers in the top 30 m, in accordance with Table 4.1.8.4.-B of the OBC 2024.

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

It should be noted that the clayey silt to silty clay and very loose to loose silty sand/sandy silt are considered to have a post-liquefaction residual shear strength (S_r) which is sufficient to support site features such as septic beds, playgrounds, and walkways. Hence, cyclic straining and liquefaction effects only need to be considered in the design of heavier structures which would exceed the S_r , such as the proposed building addition.

Reference should be made to the latest revision of the Ontario Building Code 2024 for a full discussion of the earthquake design requirements.

5.5 Slab-on-Grade/Basement Slab Construction

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building addition, the existing fill or native soil surface will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Where the subgrade consists of the existing fill, it should be proof-rolled with several passes of a vibratory drum roller. Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, is recommended for backfilling below the floor slab.

For slab-on-grade areas, the upper 200 mm of underslab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed building addition should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

For basement areas, it is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone. Further, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided underlying the basement slabs. 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 basement walls of the proposed building addition, where present. However, the conditions can be represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m³. Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 12.2 kN/m³, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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/m³)
- H = height of the wall (m)

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

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

Seismic Earth Pressures

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

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

- $a_c = (1.45 - a_{max}/g)a_{max}$
- γ = unit weight of fill of the applicable retained soil (kN/m³)
- H = height of the wall (m)
- g = gravity, 9.81 m/s²

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.357 g according to the 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 $P_o = 0.5 K_o \gamma H^2$, 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 the OBC 2024.

5.7 Pavement Design

The recommended pavement structures for car only parking areas, heavy truck parking areas, and access lanes are presented in Tables 4 and 5 below.

Table 4 – 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 in situ soil, fill, or OPSS Granular B Type I or II material placed over in situ soil.	

Table 5 – Recommended Pavement Structure – Access Lanes, Loading Zones and Heavy Truck Parking Areas	
Thickness (mm)	Material Description
40	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course – HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE – OPSS Granular A Crushed Stone
300	SUBBASE – OPSS Granular B Type II
SUBGRADE – Either in situ soil, fill, or OPSS Granular B Type I or II material placed over in situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD using suitable vibratory equipment.

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.

Due to the low permeability of the subgrade materials 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. The subgrade surface should be crowned to promote water flow to the drainage lines.

5.8 Stormwater Management Pond

It is understood that the proposed dry SWM pond will be located at the north end of the subject site. Based on the boreholes completed within the vicinity of the SWM pond, it is anticipated that the proposed structure will be founded within the clayey silt and glacial till.

Given the clayey silt and glacial till anticipated on the bottom and lower sides of the SWM pond, a liner is recommended. The pond liner may consist of one of the following:

- ☐ A 500 mm thick layer of workable, brown silty clay which is compacted with several passes of a sheepsfoot roller, and placed directly over the clayey silty and/or glacial till, or a
- ☐ 150 mm thick layer of 19 mm clear crushed stone placed over the clayey silt and/or glacial till subgrade, followed by a scrim-reinforced geosynthetic clay liner (such as the Bentofix Thermal Lock SRNWL Series Geosynthetic Clay Liner, or approved equivalent) which is capped with a 300 mm thick layer of OPSS Granular A material.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the portion of the proposed building addition which contains the basement. The system should consist of a 100 mm diameter perforated and corrugated plastic pipe, surrounded on all-sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the basement. The pipe should have a positive outlet, such as a gravity connection to the storm sewer, where gravity drainage is feasible, or alternatively to a sump pit which is equipped with a duplex pump and alarm system.

A geocomposite drainage board, such as Delta Terraxx, should be installed over the exterior basement walls and connected to the perimeter foundation drainage system.

Underslab Drainage

Underslab drainage is recommended underlying the basement slab. For preliminary design purposes, it is recommended that 100 mm perforated pipes be placed at approximate 6 m centres underlying the basement floor. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

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 within 1.8 m below finished grade. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. Below the frost depth (approximately 1.8 m), the foundation walls can be backfilled with the site excavated fill, provided that this material is maintained in an unfrozen state and at a suitable moisture content for compaction.

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 a maximum of 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

6.2 Protection of Footings Against Frost Action

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

Exterior unheated foundations, such as isolated 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 an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

Unsupported Excavations

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by temporary shoring systems from the start of the excavation until the structure is backfilled.

Based on the subsurface conditions encountered and the proposed building setback from the property lines, it is anticipated that sufficient space will be available to slope the excavation.

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 soils are considered to be 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. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Underpinning

Should lean concrete trenches be utilized to support the proposed building foundation, then the excavation in the vicinity of the existing building would extend below its footings.

In this case, in order to mitigate impacts to the support of these existing footings, underpinning would be required. Conventional underpinning piers constructed in stages, such as using the “piano key method”, would be appropriate, and would need to extend to the same depth as the proposed lean concrete trenches. Underpinning piers to this depth may need lateral support such as tieback anchors. The underpinning design should be prepared by an engineer specializing in these works, and should be provided to Paterson for review.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. 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 the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

To avoid the long-term lowering of the groundwater level at this site, clay seals should be provided at site boundaries and at strategic locations in the service trenches where the excavation is below the groundwater level. The barriers should be at least 1.5 m long (in the trench direction) 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 compactable brown silty clay placed in maximum 225 mm thick loose layers 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

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate 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.

Dewatering Permit

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 to Neighbouring Properties

As the portion of the proposed building addition in the vicinity of the existing building on-site will be a slab-on-grade structure which will not require a perimeter drainage system, long-term groundwater lowering is not anticipated in the vicinity of any existing structures. Therefore, no adverse effects are expected to nearby structures as a result of groundwater lowering.

Also, the cohesive soils encountered at the subject site are sufficiently overconsolidated such that dewatering to the depth of the foundation drainage system for a typical basement level (approximately 4 m below finished grades) would not cause settlement of surrounding surface features.

Further, as the proposed building addition will be setback from the site limits, no impacts to the neighbouring properties are anticipated as a result of excavation at the subject site.

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 into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Considerations

Paterson completed a soils review of the site to determine the applicable tree planting setbacks, in accordance with *the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines)*. Atterberg limits testing was completed for recovered samples of the clayey silt to silty clay at selected locations throughout the subject site. Sieve analysis testing was also completed on selected soil samples. The above-noted test results were completed on samples taken at depths between the anticipated underside of foundation elevation and a 3.5 m depth below finished grade. The results of the laboratory testing are presented in Tables 1 and 2 in Section 4.2 and in Appendix 1.

Based on the results of our review, the plasticity index was found to be less than 40% for the tested clayey silt to silty clay samples, and is therefore considered to be low to medium sensitivity clay/silt with respect to shrinkage from trees.

The following tree planting setbacks are therefore recommended for the low to medium sensitivity silty clay deposit throughout the subject site.

Large trees (mature height over 14 m) can be planted within these areas provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits may be reduced to **4.5 m** for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m), provided that the conditions noted below are met:

- ❑ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the grading plan.

- ❑ A small tree must be provided with a minimum of 25 m³ of available soils volume while a medium tree must be provided with a minimum of 30 m³ of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.
- ❑ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the landscape architect.
- ❑ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).
- ❑ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision grading plan.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

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, and landscaping plan(s) from a geotechnical perspective.
- ☐ Review of the geotechnical aspects of the drainage and/or waterproofing design system, if not designed by Paterson, prior to construction.

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

- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and granular fill materials used.
- ☐ 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 ensure that the specified level of compaction has been 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 our present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

The 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 the Brunstad Christian Church Ottawa, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.



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



Drew Petahtegoose, P.Eng.

Report Distribution:

- ☐ Brunstad Christian Church Ottawa (Email Copy)
- ☐ Paterson Group (1 Copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMIT TESTING RESULTS

GRAIN SIZE AND HYDROMETER TESTING RESULTS

SHRINKAGE TESTING RESULTS

ANALYTICAL TESTING RESULTS

COORD. SYS.: MTM ZONE 9 **EASTING:** 364540.41 **NORTHING:** 5005487.11 **ELEVATION:** 92.36

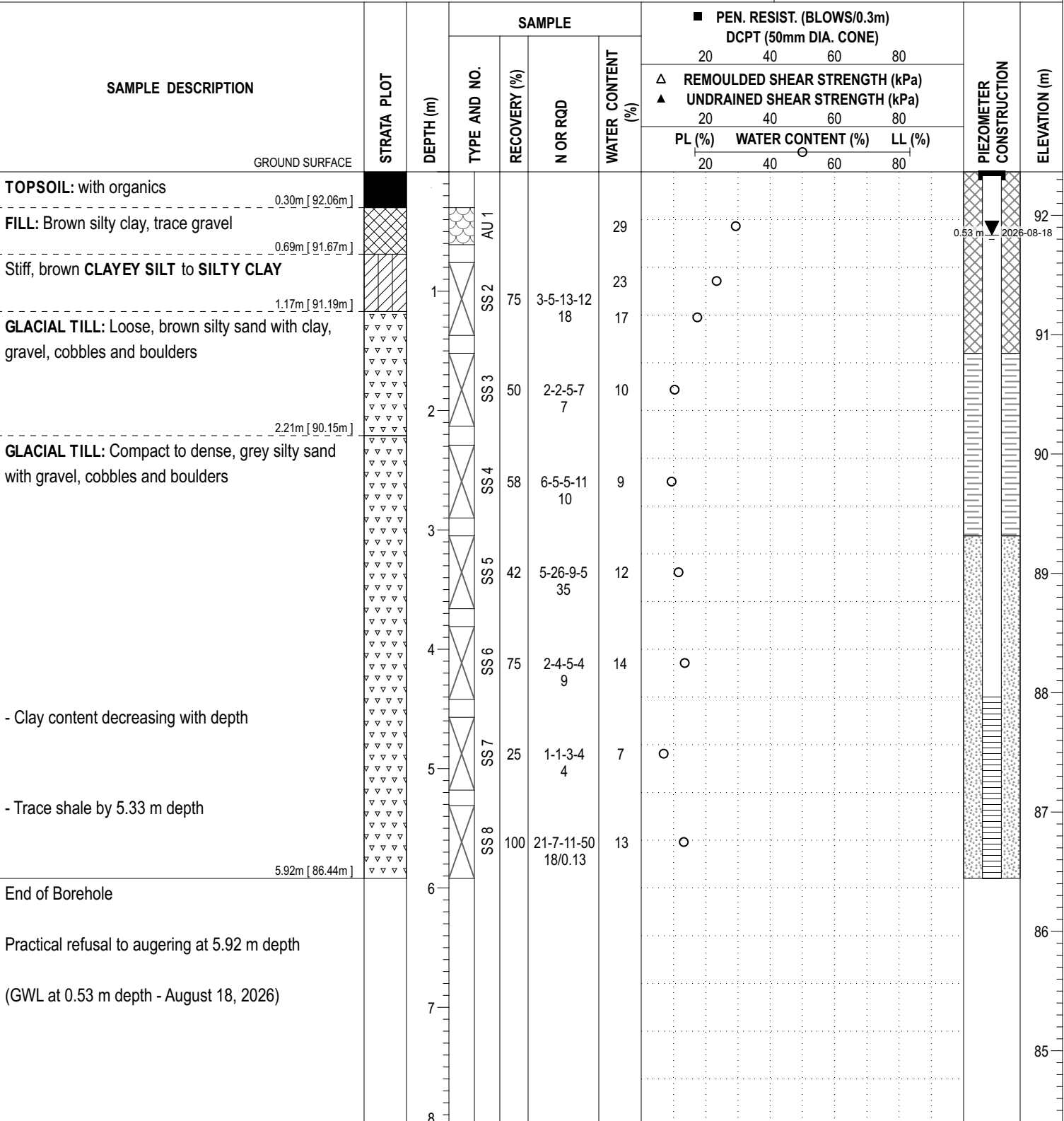
PROJECT: Proposed Building Addition

FILE NO. : PG6727

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010

DATE: August 10, 2026

HOLE NO. : BH 1-26


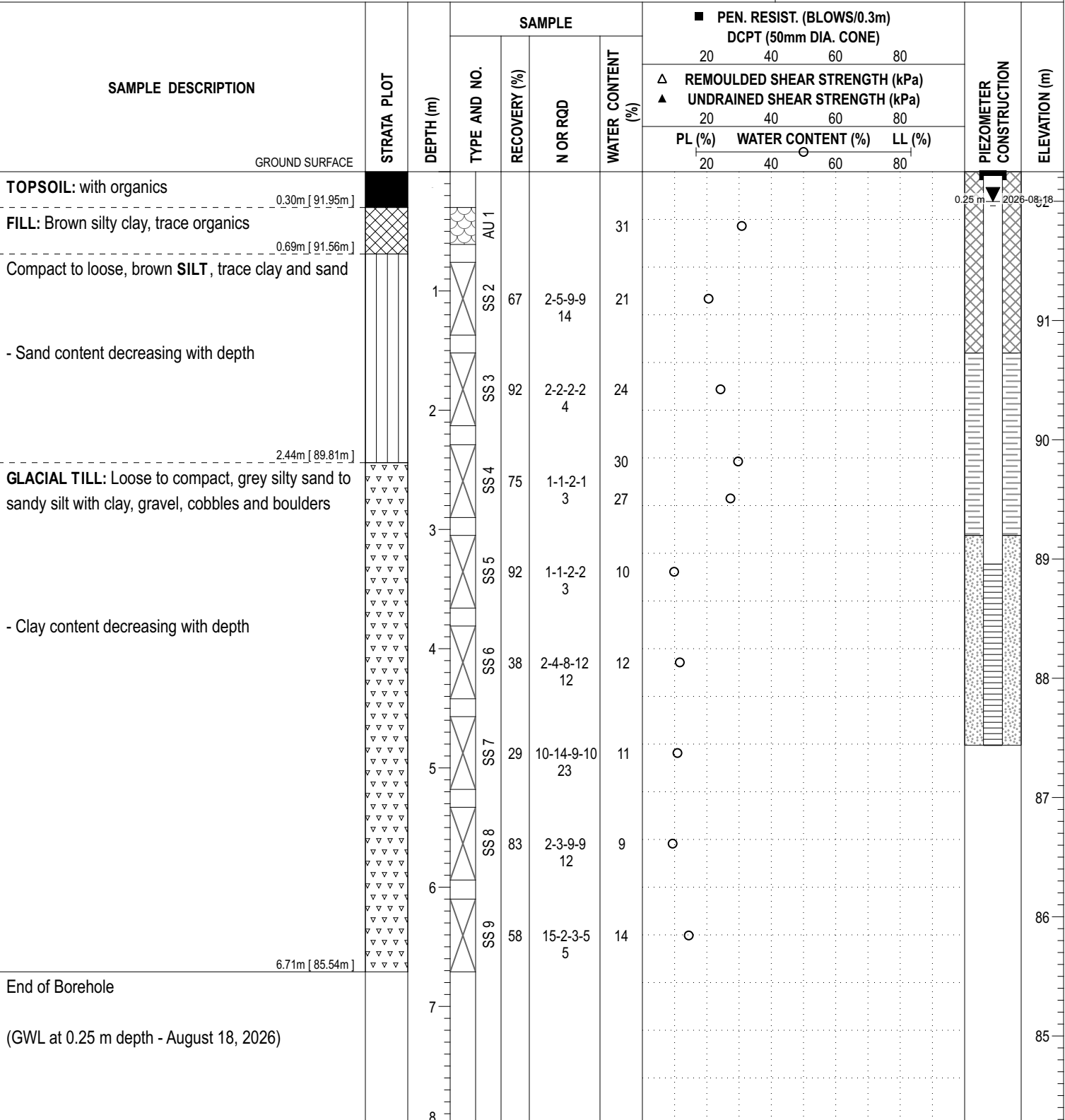
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COORD. SYS.: MTM ZONE 9 **EASTING:** 364563.36 **NORTHING:** 5005500.49 **ELEVATION:** 92.25

PROJECT: Proposed Building Addition **FILE NO. :** PG6727

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** August 10, 2026 **HOLE NO. :** BH 2-26



DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

COORD. SYS.: MTM ZONE 9 **EASTING:** 364523.40 **NORTHING:** 5005509.04 **ELEVATION:** 92.20

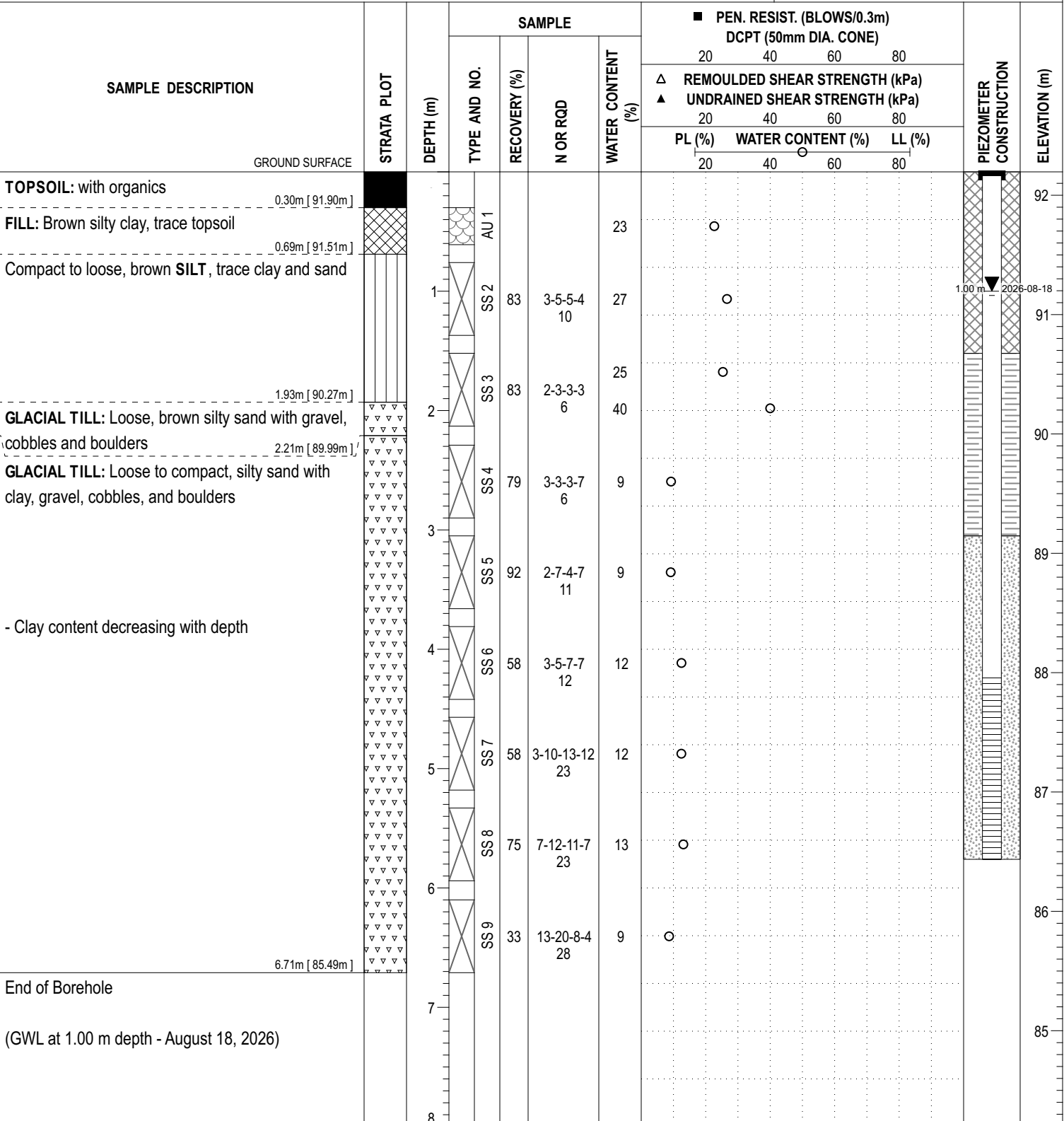
PROJECT: Proposed Building Addition

FILE NO. : PG6727

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010

DATE: August 10, 2026

HOLE NO. : BH 3-26


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COORD. SYS.: MTM ZONE 9

EASTING: 364493.99

NORTHING: 5005515.09

ELEVATION: 92.62

PROJECT: Proposed Building Addition

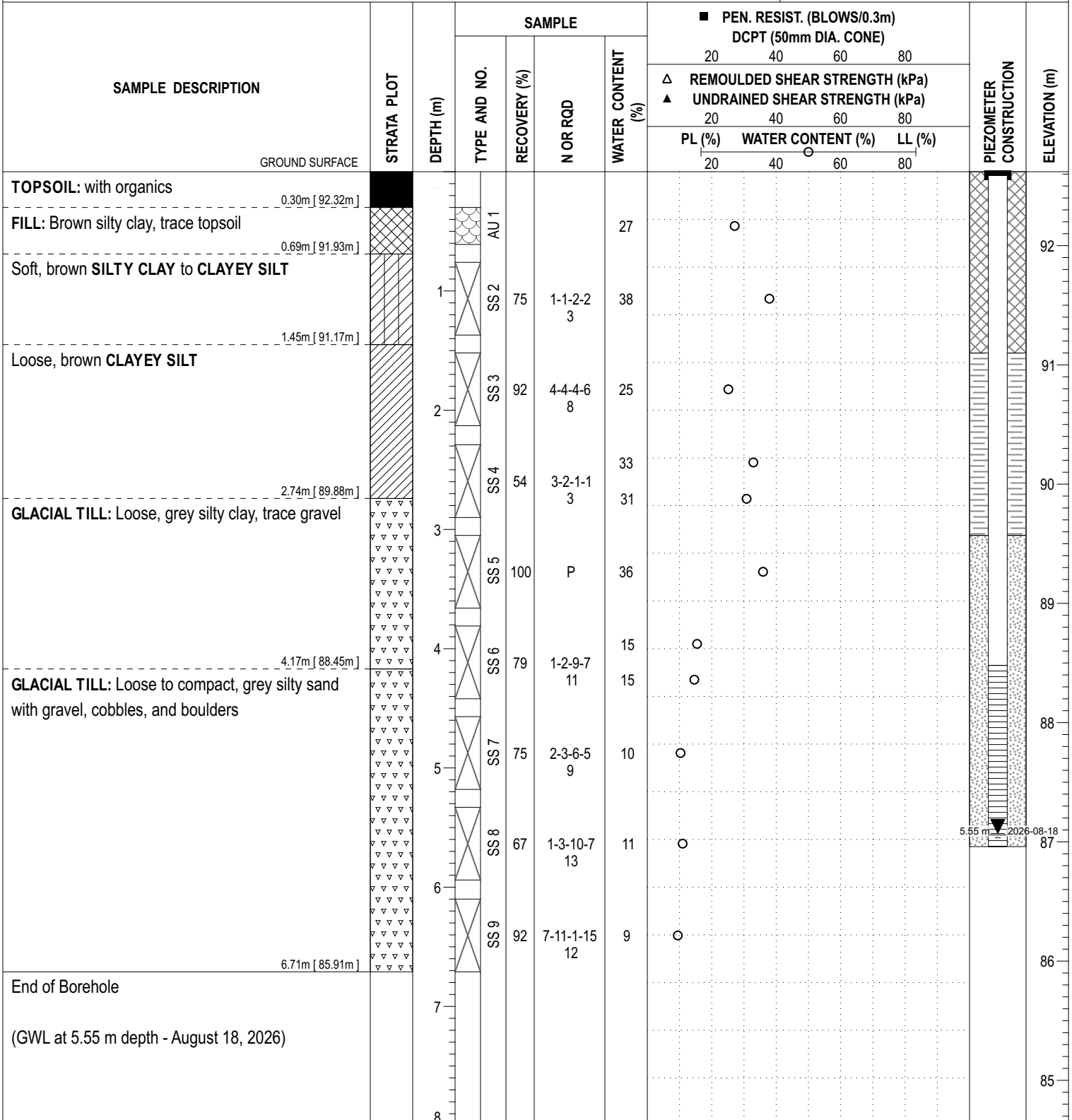
FILE NO. : PG6727

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS: Datum: NAD1983 (Canada)

Geoid: HT2-2010

DATE: August 10, 2026

HOLE NO. : BH 4-26


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**PATERSON
GROUP**

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1981 Century Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 364500.506 **NORTHING:** 5005505.065 **ELEVATION:** 92.70

PROJECT: Geotechnical Investigation - Proposed Church Addition

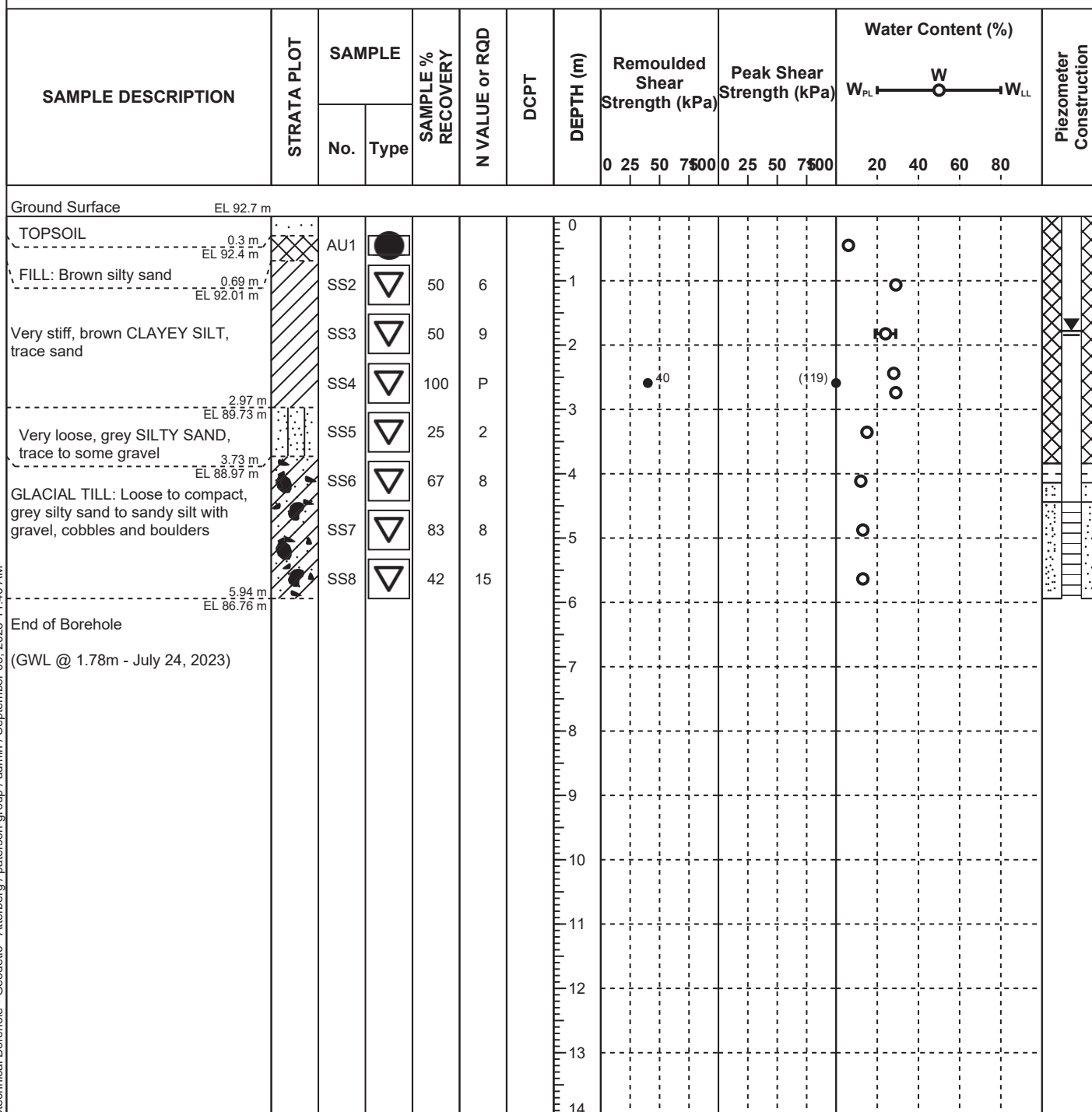
FILE NO. PG6727

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: July 17, 2023

HOLE NO. BH 1-23



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FILE NO. PG6727

HOLE NO. BH 2-23

REMARKS: DATE: July 17, 2023

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SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1981 Century Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 364517.298 **NORTHING:** 5005546.179 **ELEVATION:** 91.87

PROJECT: Geotechnical Investigation - Proposed Church Addition

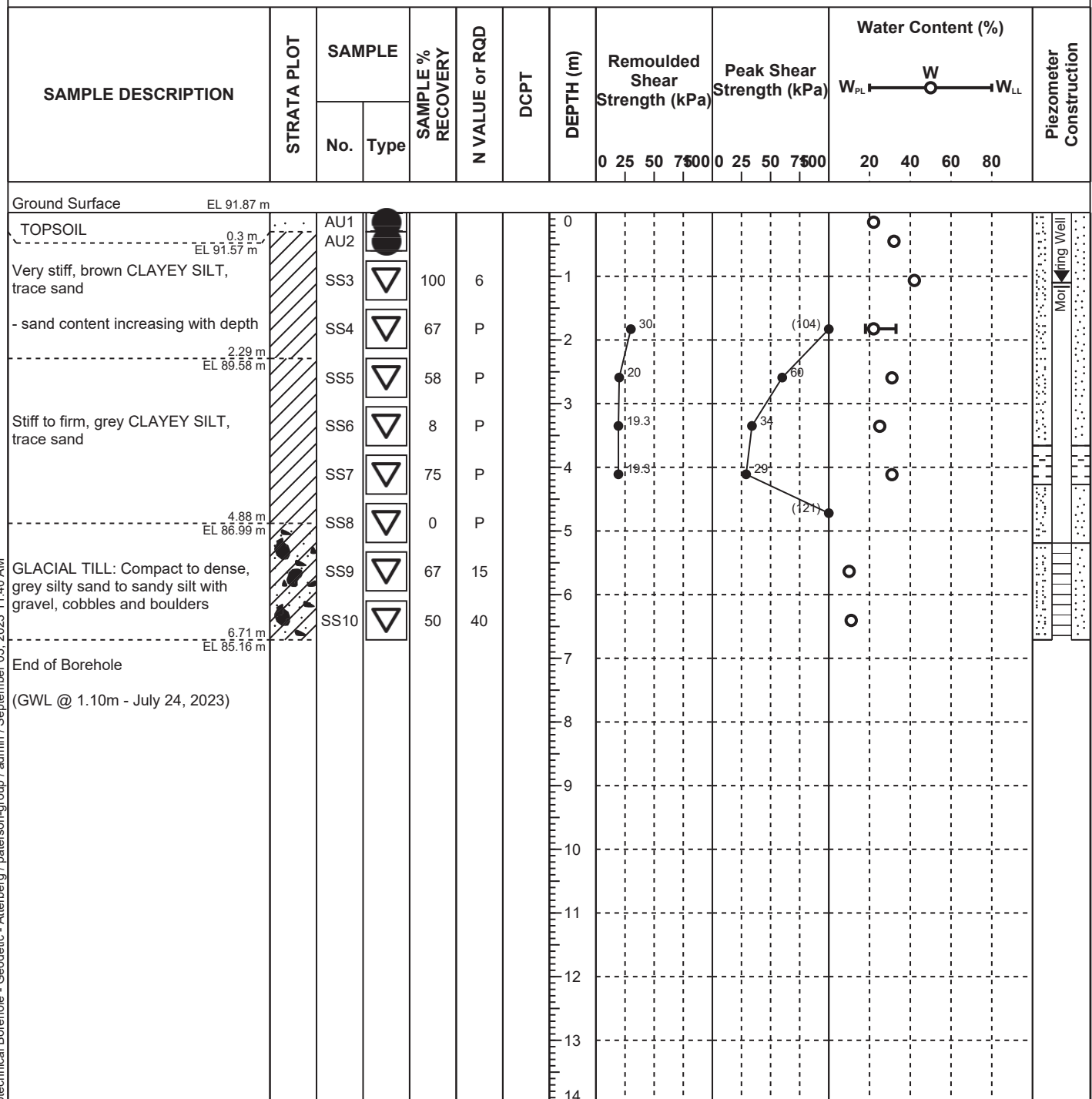
FILE NO. PG6727

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: July 17, 2023

HOLE NO. BH 3-23



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SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

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PROJECT: Geotechnical Investigation - Proposed Church Addition

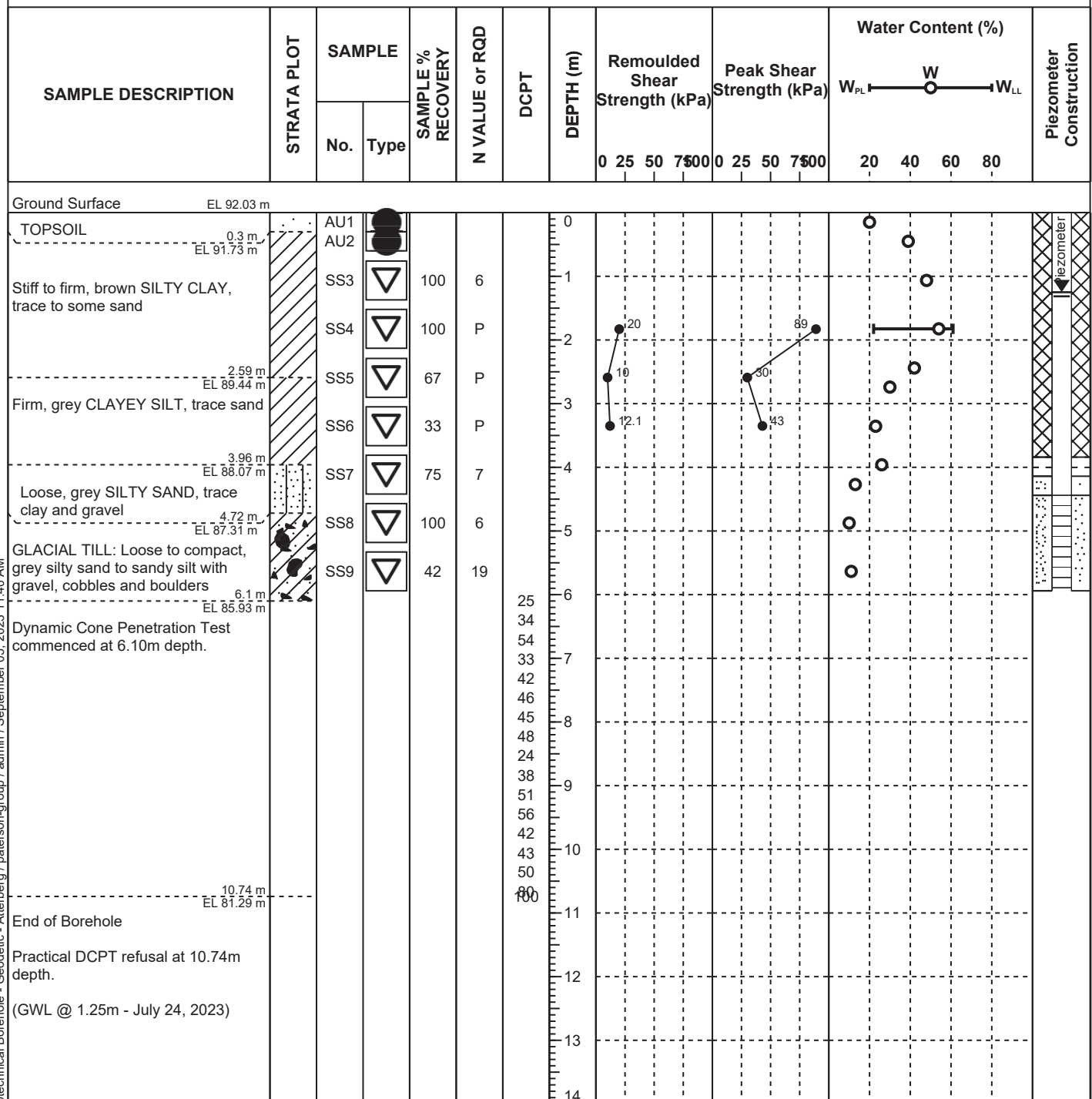
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REMARKS:

DATE: July 17, 2023

HOLE NO. BH 4-23



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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

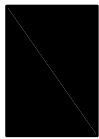
p' _o	-	Present effective overburden pressure at sample depth
p' _c	-	Preconsolidation pressure of (maximum past pressure on) sample
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / 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)

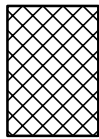
STRATA PLOT



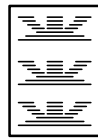
Topsoil



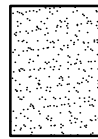
Asphalt



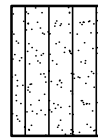
Fill



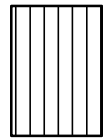
Peat



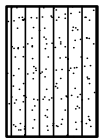
Sand



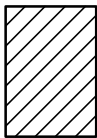
Silty Sand



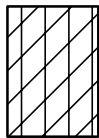
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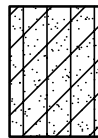
Sandy Silt



Clay



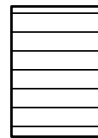
Silty Clay



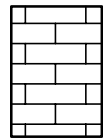
Clayey Silty Sand



Glacial Till



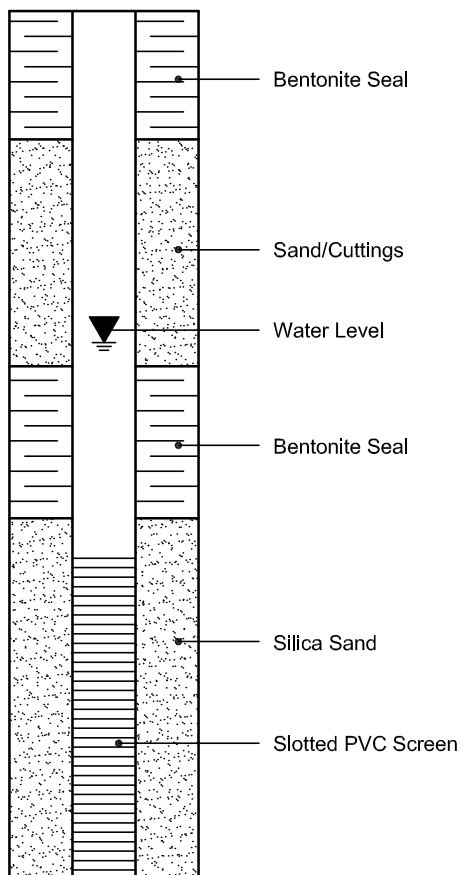
Shale



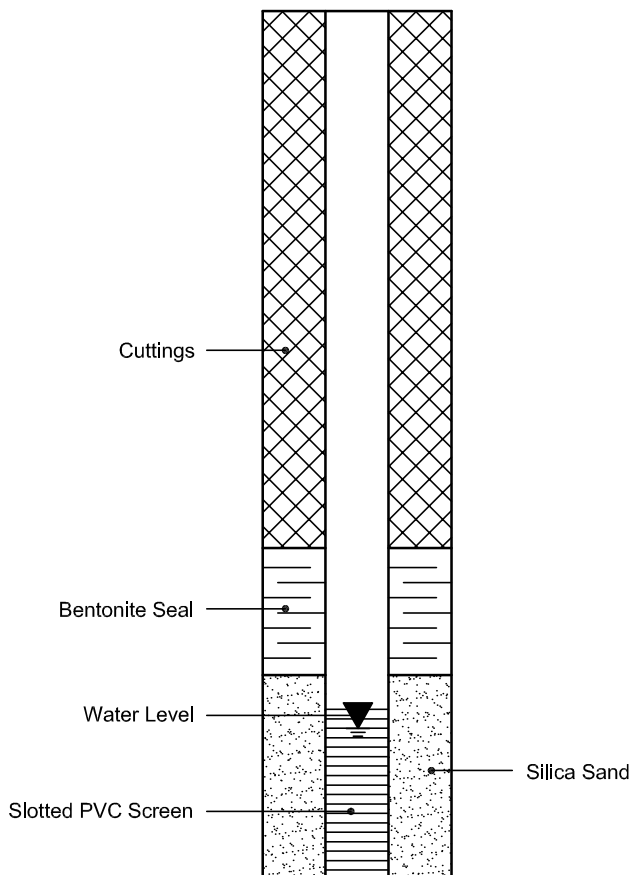
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION

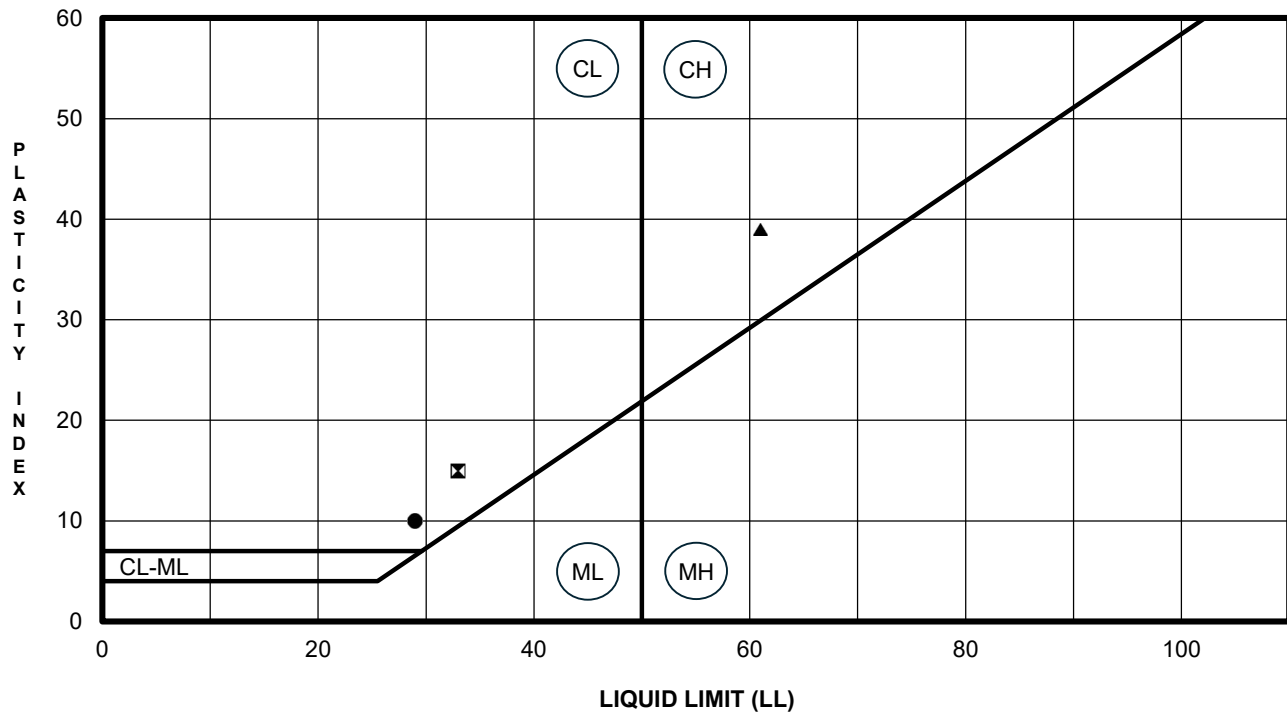




**PATERSON
GROUP**

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

ATTERBERG LIMITS' RESULTS



Specimen Identification	LL	PL	PI	Fines	Classification
● BH 1-23 SS3	29	19	10		CL - Inorganic clays of low plasticity
⊠ BH 3-23 SS4	33	18	15		CL - Inorganic clays of low plasticity
▲ BH 4-23 SS4	61	22	39		CH - Inorganic clays of high plasticity
★					
⊙					
⊕					
○					
△					
⊗					
⊕					
□					
⊗					
⊕					
☆					
⊗					
■					
◆					
◇					

CLIENT Brunstad Christian Church Ottawa

FILE NO.

PG6727

PROJECT Geotechnical Investigation - Proposed Building Addition

DATE

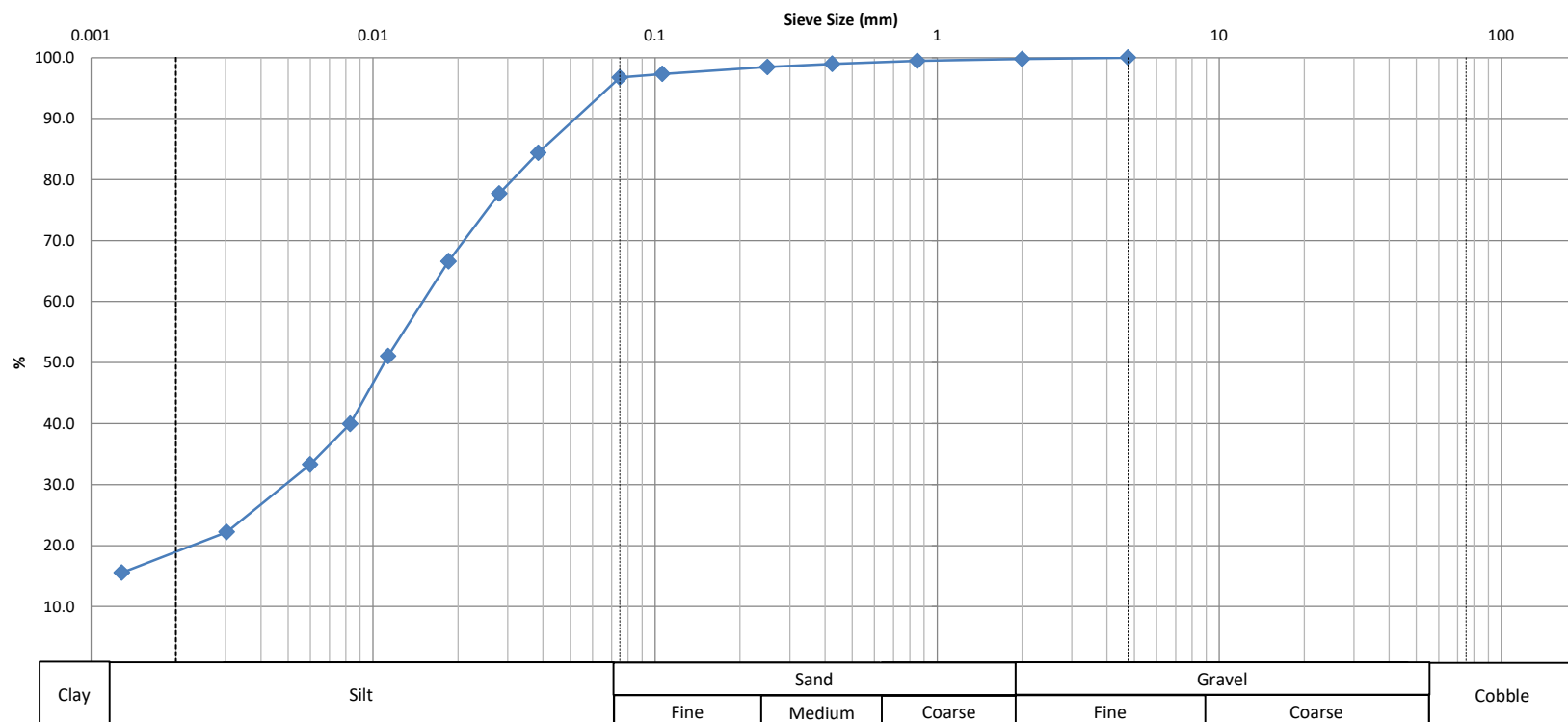
26-Jul-23

1981 Century Road, Ottawa, ON



**SIEVE ANALYSIS
ASTM C136**

CLIENT:	Brunstad Christian Church	DEPTH:	7'6" - 9'6"	FILE NO:	PG6727
CONTRACT NO.:		BH OR TP No.:	BH1-23 SS4(Top)	LAB NO:	45271
PROJECT:	1981 Century Road			DATE RECEIVED:	20-Jul-23
				DATE TESTED:	24-Jul-23
DATE SAMPLED:	17-Jul-23			DATE REPORTED:	11-Aug-23
SAMPLED BY:	C.E			TESTED BY:	DJ



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
	D100	D60	D30	D10	Gravel (%)	28.2					
					0.0	3.3					
Comments:											
Curtis Beadow						Joe Forsyth, P. Eng.					
REVIEWED BY:											

CLIENT:	Brunstad Christian Church	DEPTH:	7'6" - 9'6"	FILE NO.:	PG6727
PROJECT:	1981 Century Road	BH OR TP No.:	BH1-23 SS4(Top)	DATE SAMPLED:	17-Jul-23
LAB No. :	45271	TESTED BY:	DJ	DATE RECEIVED:	20-Jul-23
SAMPLED BY:	C.E	DATE REPT'D:	11-Aug-23	DATE TESTED:	24-Jul-23

SAMPLE INFORMATION

SAMPLE MASS		SPECIFIC GRAVITY	
170.4		2.700	
INITIAL WEIGHT	50.00	HYGROSCOPIC MOISTURE	
WEIGHT CORRECTED	44.42	TARE WEIGHT	0.00
WT. AFTER WASH BACK SIEVE	1.54	AIR DRY	191.80
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	170.40
		CORRECTED	0.888

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75	0.0	0.0	100.0
2.0	0.4	0.2	99.8
Pan	170.0		
0.850	0.15	0.5	99.5
0.425	0.39	1.0	99.0
0.250	0.65	1.5	98.5
0.106	1.22	2.7	97.3
0.075	1.52	3.3	96.7
Pan	1.54		
SIEVE CHECK	0.0	MAX = 0.3%	

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (°C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	9:34	44.0	6.0	23.0	0.0386	84.6	84.4
2	9:35	41.0	6.0	23.0	0.0281	77.9	77.7
5	9:38	36.0	6.0	23.0	0.0185	66.8	66.6
15	9:48	29.0	6.0	23.0	0.0113	51.2	51.1
30	10:03	24.0	6.0	23.0	0.0083	40.1	40.0
60	10:33	21.0	6.0	23.0	0.0060	33.4	33.3
250	1:43	16.0	6.0	23.0	0.0030	22.3	22.2
1440	9:33	13.0	6.0	23.0	0.0013	15.6	15.5

Moisture = 28.2%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.
		



**Linear Shrinkage
ASTM D4943-02**

CLIENT:	Brunstad Christian Church	DEPTH	2'-6" to 4'-6"	FILE NO.:	PG6727
PROJECT:	1981 Century Road	BH OR TP No:	BH3-23 SS3	DATE SAMPLED	17-Jul-23
LAB No:	45270	TESTED BY:	CP	DATE RECEIVED	20-Jul-23
SAMPLED BY:	C.E	DATE REPORTED:	11-Aug-23	DATE TESTED	24-Jul-23

LABORATORY INFORMATION & TEST RESULTS

Moisture		No. of Blows(7)	Calibration (Two Trials)		Tin NO.(x22)
Tare		5.07	Tin	4.85	4.86
Soil Pat Wet + Tare		72.48	Tin + Grease	5.07	5.07
Soil Pat Wet		67.41	Glass	48.97	48.97
Soil Pat Dry + Tare		52.51	Tin + Glass + Water	91.55	91.49
Soil Pat Dry		47.44	Volume	37.51	37.45
Moisture		42.10	Average Volume	37.48	

Soil Pat + String	47.66
Soil Pat + Wax + String in Air	53.39
Soil Pat + Wax + String in Water	18.47
Volume Of Pat (Vdx)	34.92

RESULTS:

Shrinkage Limit	23.13
Shrinkage Ratio	1.666
Volumetric Shrinkage	31.593
Linear Shrinkage	8.744

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.
		

Certificate of Analysis

Report Date: 24-Jul-2023

Client: Paterson Group Consulting Engineers

Order Date: 18-Jul-2023

Client PO: 57933

Project Description: PG6727

Client ID:	BH4-23 SS3	-	-	-	-
Sample Date:	17-Jul-23 09:00	-	-	-	-
Sample ID:	2329160-01	-	-	-	-
Matrix:	Soil	-	-	-	-
MDL/Units					

Physical Characteristics

% Solids	0.1 % by Wt.	79.8	-	-	-	-
----------	--------------	------	---	---	---	---

General Inorganics

pH	0.05 pH Units	7.41	-	-	-	-
Resistivity	0.1 Ohm.m	39.9	-	-	-	-

Anions

Chloride	10 ug/g	14	-	-	-	-
Sulphate	10 ug/g	115	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6727-1 – TEST HOLE LOCATION PLAN

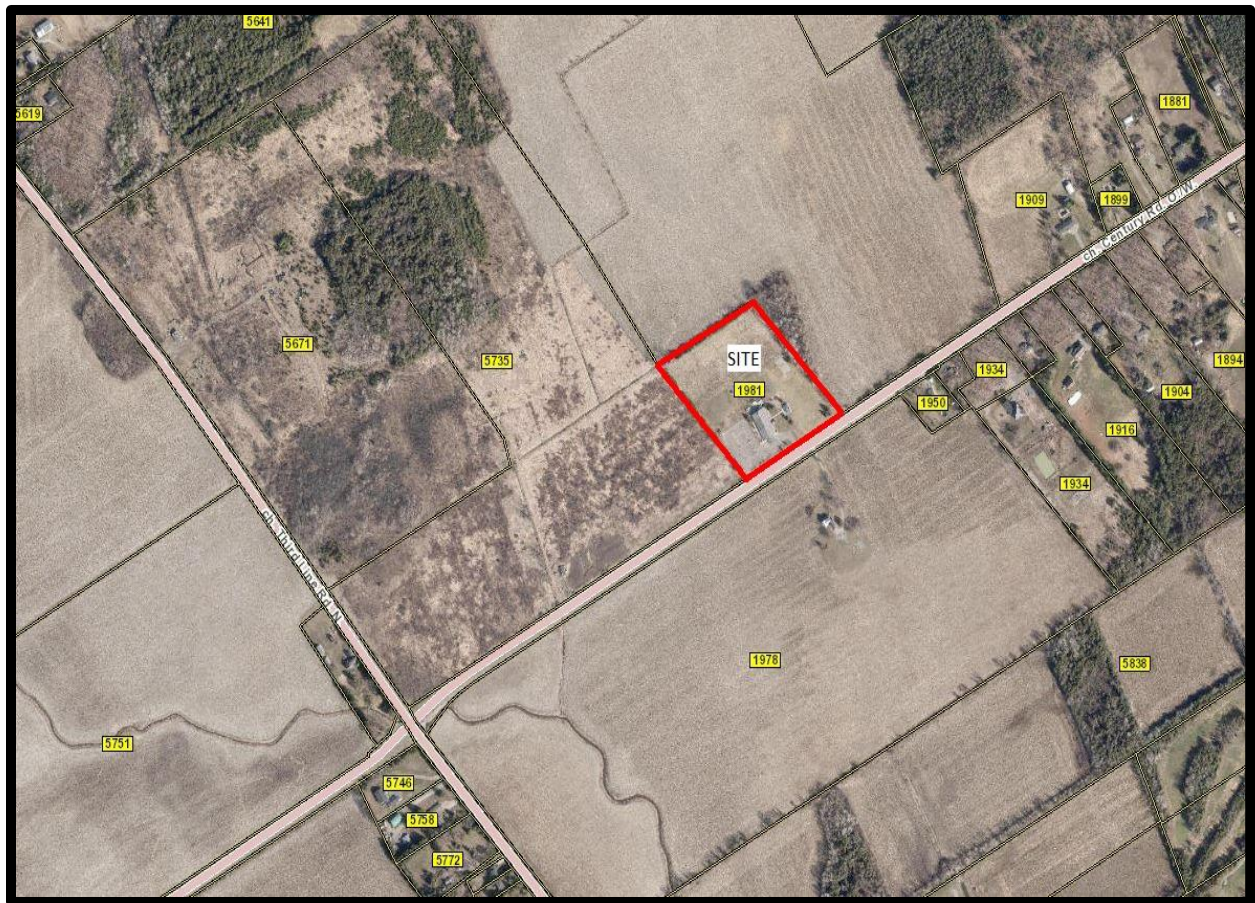
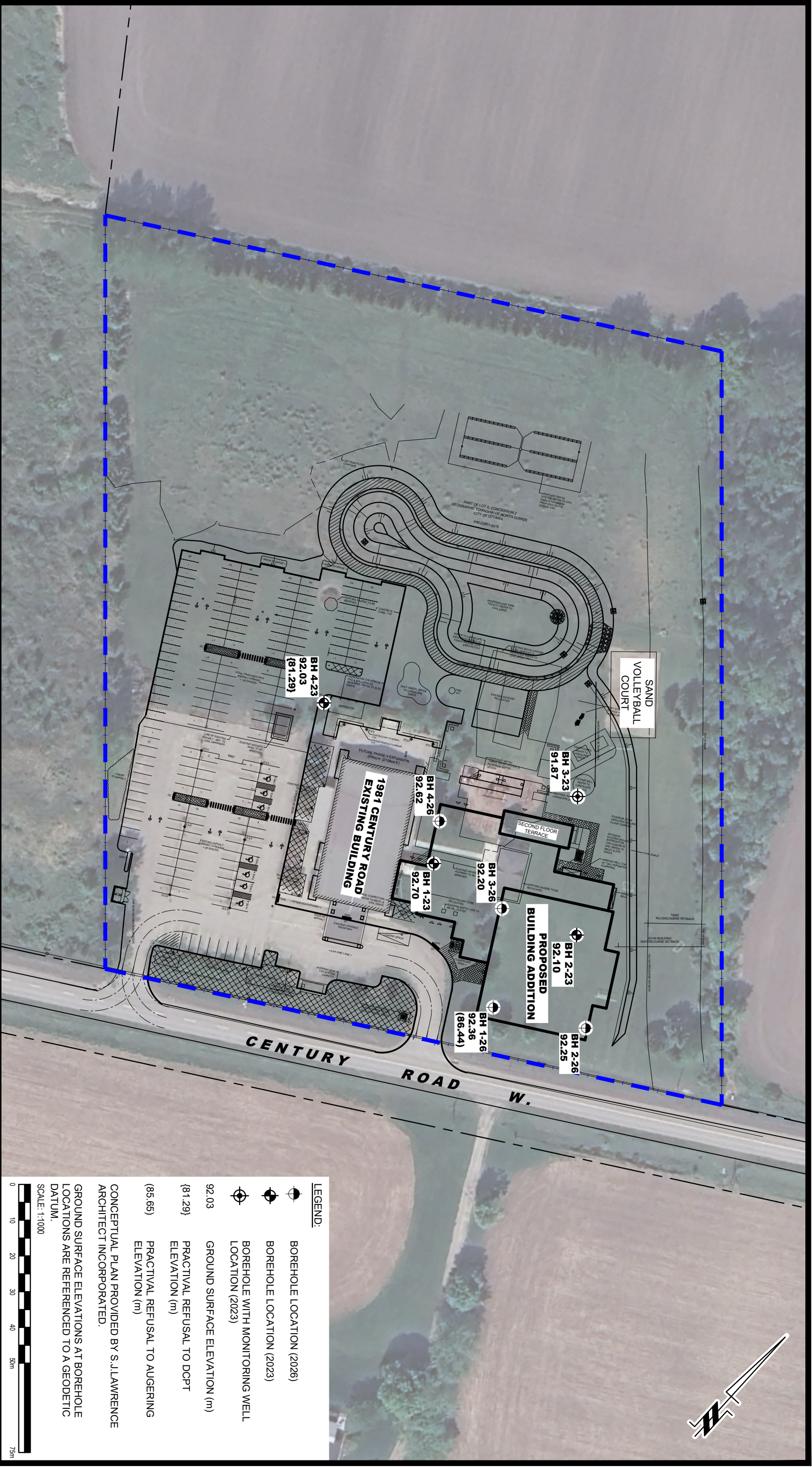


FIGURE 1

KEY PLAN



3	ADDED 2026 BOREHOLE LOCATION BH 1-26 TO BH 4-26	14/08/2026	YZ
2	UPDATED CONCEPTUAL PLAN	13/11/2025	SD
1	UPDATED CONCEPTUAL PLAN	19/09/2025	SD
NO.	REVISIONS	DDMMYY	INITIAL

PATERSON GROUP

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TEL: (613) 226-7381

BRUNSTAD CHRISTIAN CHURCH OTTAWA

GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING ADDITION
1981 CENTURY ROAD

ONTARIO

Title:

OTTAWA,

Scale:	1:1000	Date:	09/2023
Drawn by:	GK	Report No.:	PG6727-1
Checked by:	YZ	Dwg. No.:	PG6727-1
Approved by:	FA	Revision No.:	3

- LEGEND:**
- BOREHOLE LOCATION (2026)
 - BOREHOLE LOCATION (2023)
 - BOREHOLE WITH MONITORING WELL LOCATION (2023)
 - GROUND SURFACE ELEVATION (m)
 - {81.29} PRACTICAL REFUSAL TO DCPT ELEVATION (m)
 - (85.65) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
- CONCEPTUAL PLAN PROVIDED BY S.J.LAWRENCE ARCHITECT INCORPORATED.
- GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.
- SCALE: 1:1000



APPENDIX 3

GROUNDWATER MONITORING PROGRAM

PH4720-MEMO.02 DATED AUGUST 23, 2024



re: Groundwater Monitoring Program
Proposed Building Addition
1981 Century Road – Ottawa

to: Peter Twilley – ptwilley@arrowservice.ca

date: August 23, 2024

file: PH4720-MEMO.02

Further to your request and authorization, Paterson Group (Paterson) conducted a groundwater monitoring program in support of a Low Impact Development (LID) design for the proposed building addition at the aforementioned site. This report should be read in conjunction with Paterson Report PG6727-1 dated September 5, 2023.

1.0 Background Information

A geotechnical field investigation was carried out on July 17, 2023. At that time, a total of four (4) boreholes were advanced to a maximum depth of 6.7 m below existing grade (bgs). The boreholes were distributed in a manner to provide general coverage of the study area, taking into consideration existing site features.

Field Survey

The borehole locations, and ground surface elevations at each borehole location, were surveyed by Paterson using a high precision, handheld GPS and referenced to a geodetic datum. The location and ground surface elevation at each borehole location is presented on Drawing PG6727-1 - Test Hole Location Plan attached to the current memorandum.

Subsurface Profile

The subsurface profile at the borehole locations generally consisted of topsoil underlain by a very stiff to firm brown to grey clayey silt to silty clay followed by a very loose to loose grey silty sand. The above noted layers were underlain by a loose to compact glacial till deposit comprised of a silty sand matrix with varying amounts of gravel, cobbles and boulders. It should be noted that a layer of fill material was observed underlying the topsoil at BH 1-23. Practical refusal to DCPT was encountered in BH 4-23 at a depth of 10.8 m bgs.

Details of the subsurface profile can be found in the Soil Profile and Test Data Sheets attached to the current report.



Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ 1.5 m of slotted 51 mm diameter PVC screen at the base of the borehole.
- ☐ 51 mm diameter PVC riser pipe from the top of the screen to ground surface.
- ☐ No.3 silica sand backfill within the annular space around the screen.
- ☐ Bentonite hole plug placed directly above PVC slotted screen extending to the existing ground surface.
- ☐ The 51 mm diameter PVC riser was covered with a protective steel flush mount well casing at ground surface.

Specific details of the installation of the monitoring well is further included in the Soil Profile and Test Data Sheet attached to the current report.

2.0 Groundwater Monitoring Program

The monitoring well installed at BH 3-23 was equipped with a Van Essen Instrument Mini-Diver Water Level Logger on February 22, 2024, to accurately monitor fluctuations in the groundwater levels. In addition, a Van Essen Instruments Baro-Diver was installed in BH3-23 to monitor changes in atmospheric pressure. The Mini-Diver was programmed to continuously measure and record groundwater levels throughout the subject site at a rate of 1 reading every 24 hours for a period of approximately 6 months.

The results of the groundwater fluctuations and correlated precipitation events at the monitoring well location between February 22, 2024 and August 26, 2024, have been summarized in Figure 1 attached to the current report.

3.0 Groundwater Monitoring Results

The data presented in Figure 1 illustrates the collected groundwater elevations between February 22, 2024 and August 26, 2024. The groundwater readings measured within the monitoring well varied from an elevation of 90.90 m asl to a maximum elevation of 91.84 m asl. The measured low and high groundwater elevations are summarized in Table 1 below.



Based on our analysis of the data logger groundwater readings, seasonal groundwater fluctuations can be observed at the well location with a difference in elevation between low and high readings of 0.94 m.

Table 1: Groundwater Monitoring Summary				
Monitoring Well ID	Ground Surface Elevation (m asl)	Low Groundwater Elevation (m asl)	High Groundwater Elevation (m asl)	Difference in Groundwater Elevation (m asl)
BH 3-23	91.87	90.90	91.84	0.94

We trust that this information satisfies your requirements.

Best Regards,

Paterson Group Inc.

Nicholas Zulinski, P.Ge., géo.



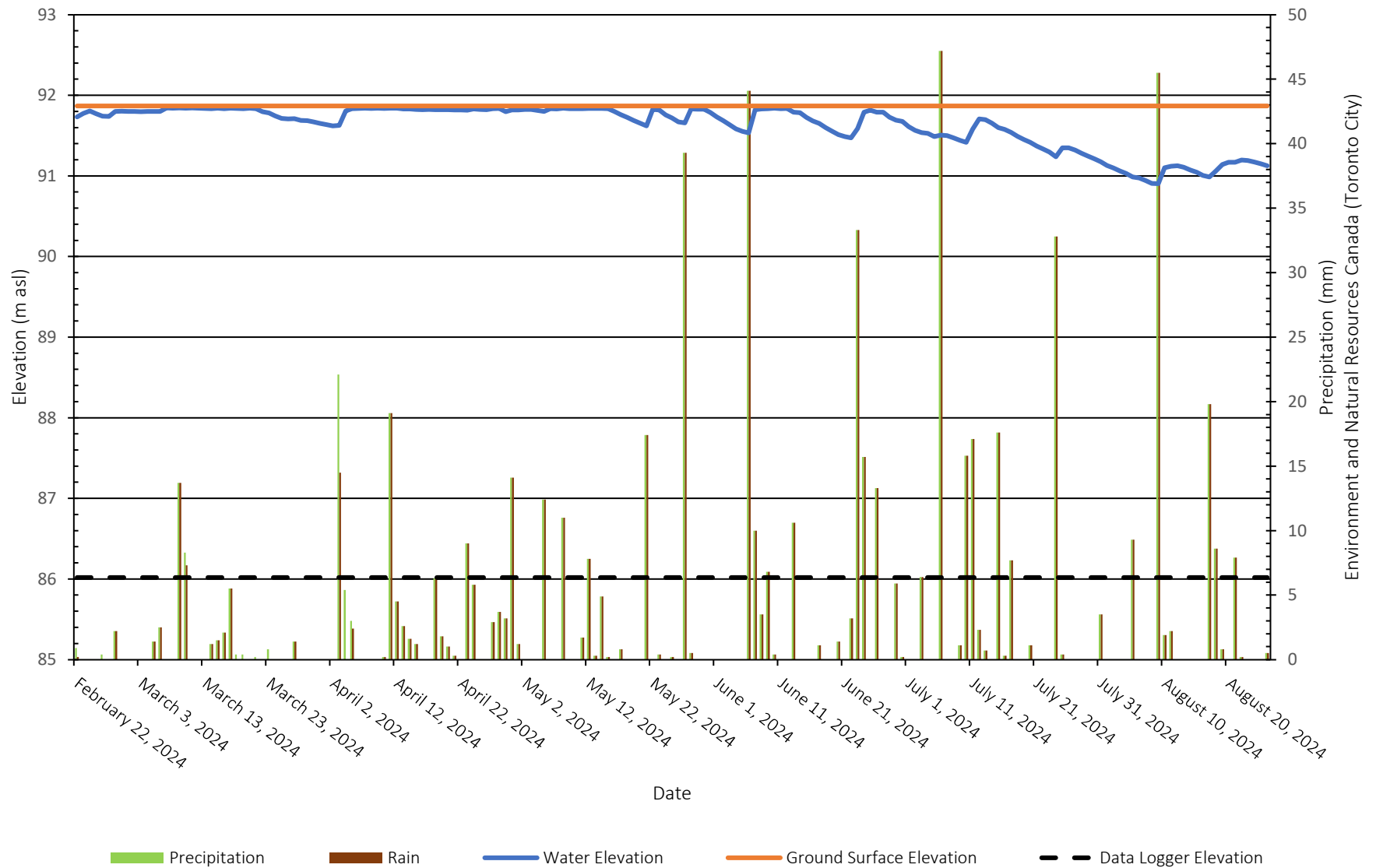
Erik Ardley, P.Ge.

Attachments

- ☐ Figure 1 – Groundwater Monitoring Levels
- ☐ Soil Profile and Test Data Sheets
- ☐ Drawing PG6727-1 – Test Hole Location Plan



BH3-23 - Monitoring Well Water Elevations



SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1981 Century Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 364500.506 **NORTHING:** 5005505.065 **ELEVATION:** 92.70

PROJECT: Geotechnical Investigation - Proposed Church Addition

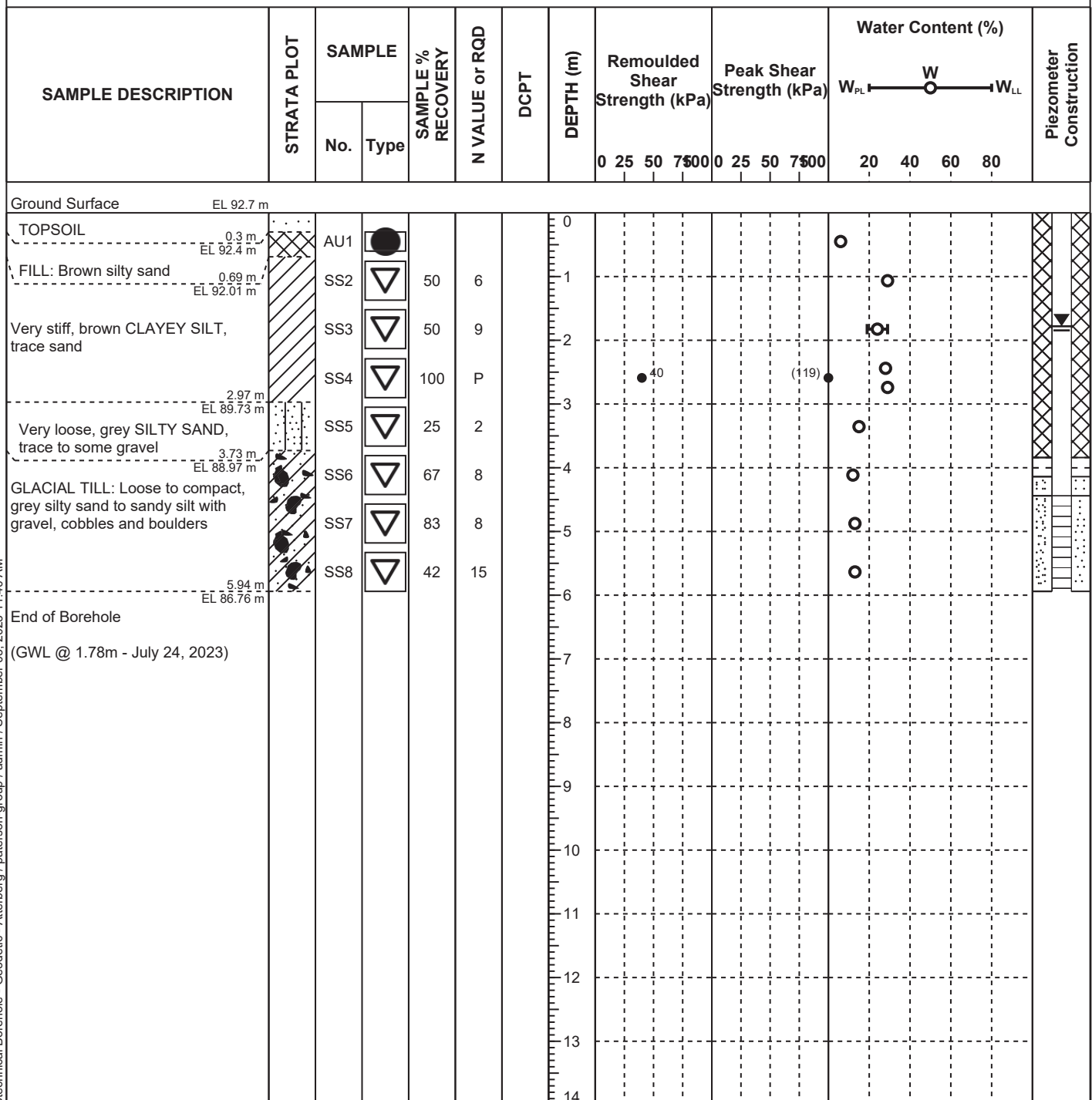
FILE NO. **PG6727**

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: July 17, 2023

HOLE NO. BH 1-23



DISCLAIMER: THE DATA PRESENTED IN THIS LOG IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHO IT WAS PRODUCED. THIS LOG SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1981 Century Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 364543.428 **NORTHING:** 5005517.55 **ELEVATION:** 92.10

PROJECT: Geotechnical Investigation - Proposed Church Addition

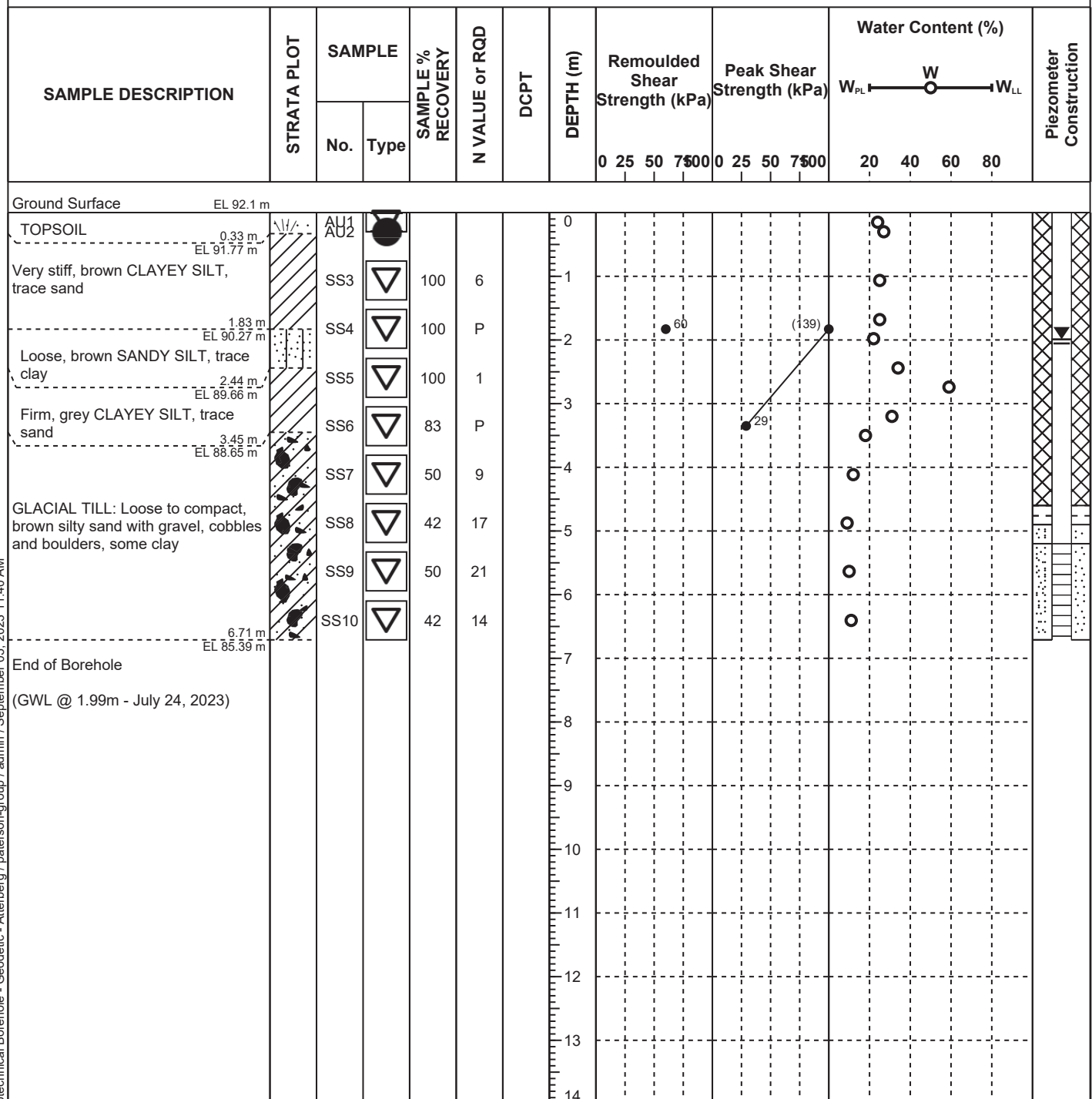
FILE NO. PG6727

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: July 17, 2023

HOLE NO. BH 2-23



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SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1981 Century Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 364517.298 **NORTHING:** 5005546.179 **ELEVATION:** 91.87

PROJECT: Geotechnical Investigation - Proposed Church Addition

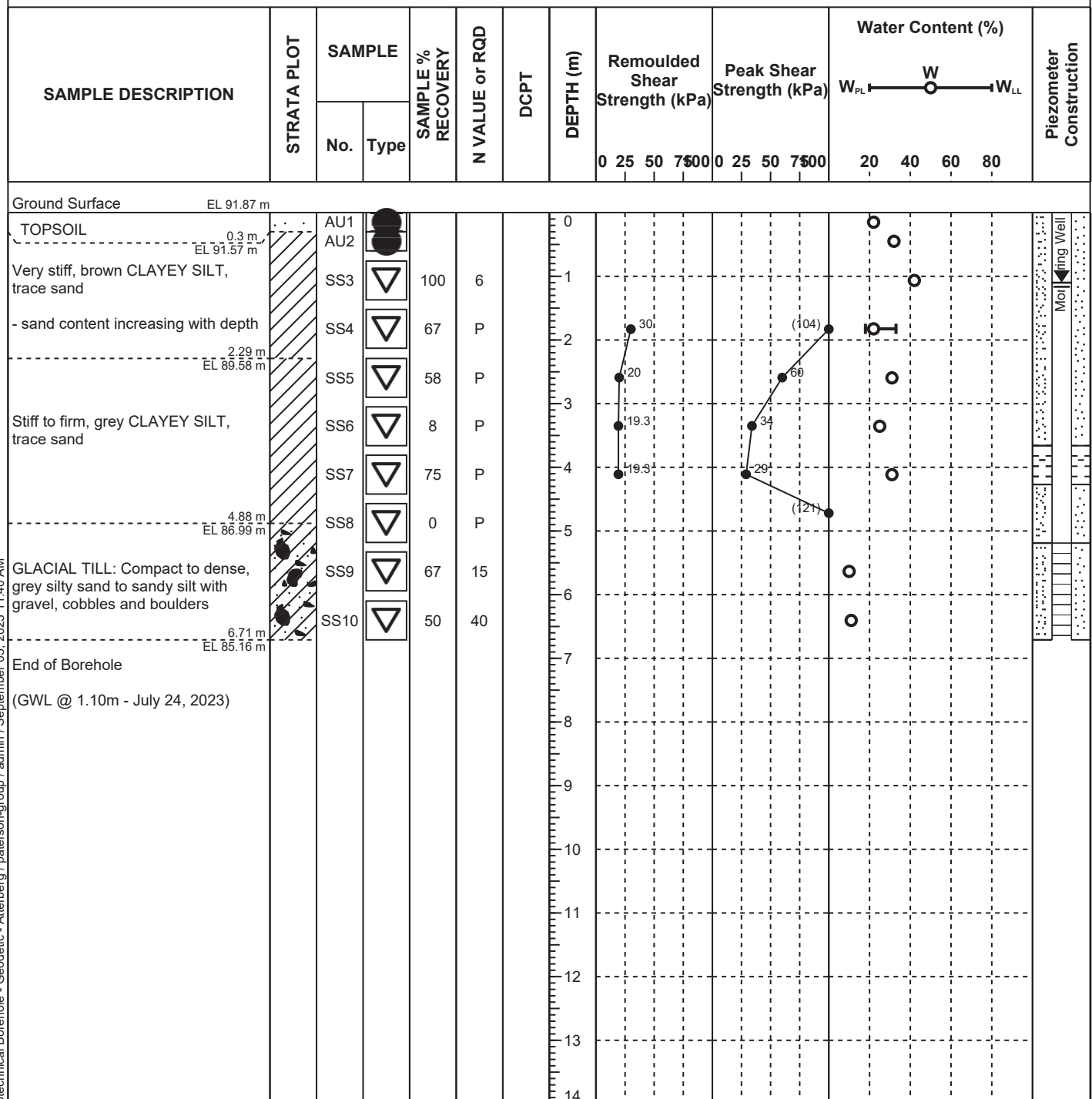
FILE NO. **PG6727**

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: July 17, 2023

HOLE NO. BH 3-23



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SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1981 Century Road, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 364447.399 **NORTHING:** 5005517.13 **ELEVATION:** 92.03

PROJECT: Geotechnical Investigation - Proposed Church Addition

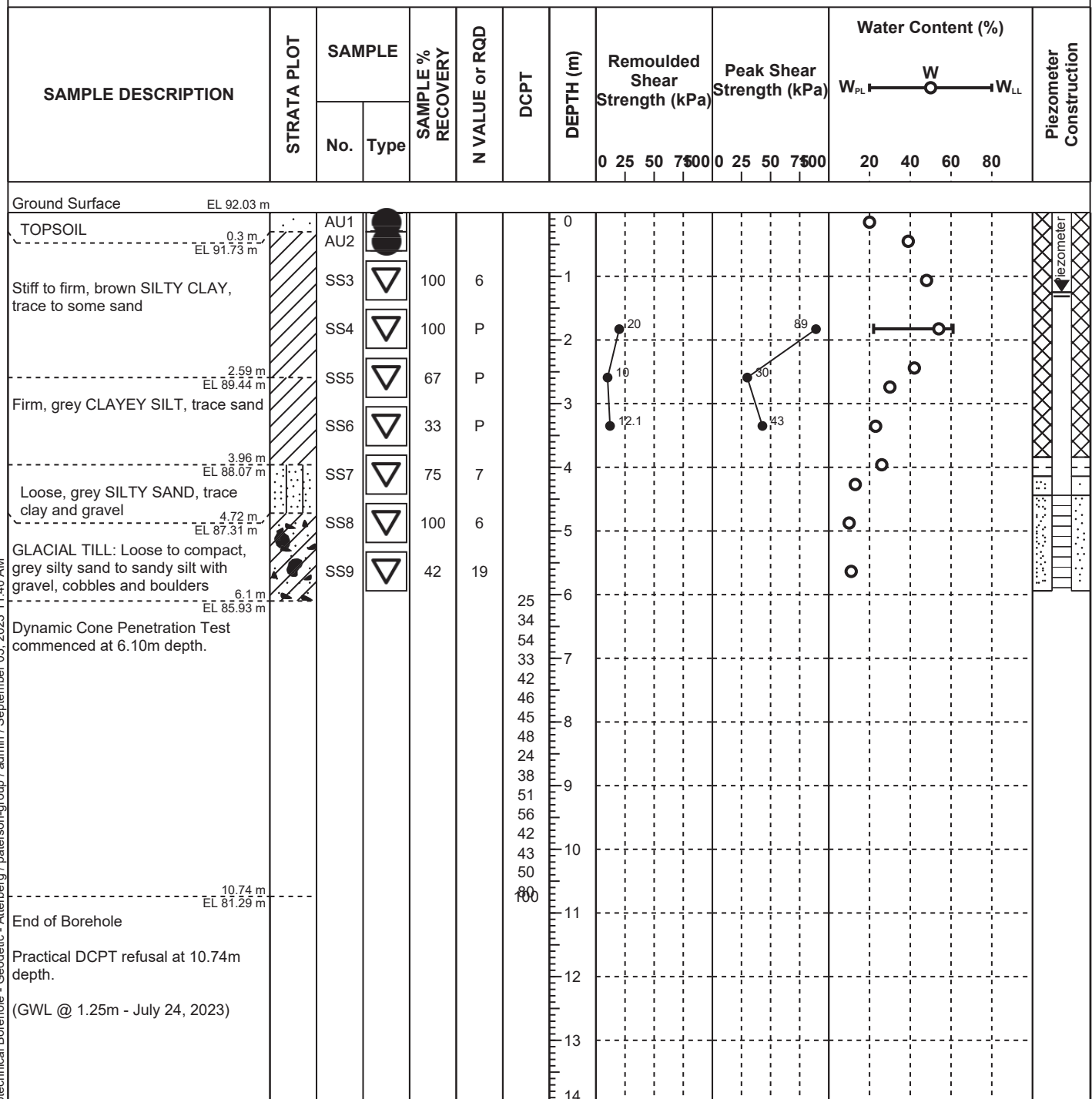
FILE NO. PG6727

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: July 17, 2023

HOLE NO. BH 4-23



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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

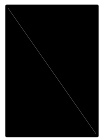
p' _o	-	Present effective overburden pressure at sample depth
p' _c	-	Preconsolidation pressure of (maximum past pressure on) sample
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / 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)

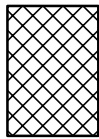
STRATA PLOT



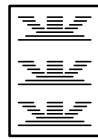
Topsoil



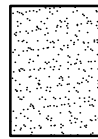
Asphalt



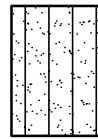
Fill



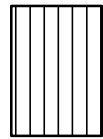
Peat



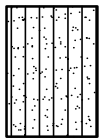
Sand



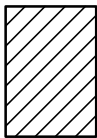
Silty Sand



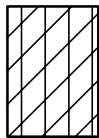
Silt



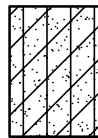
Sandy Silt



Clay



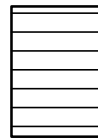
Silty Clay



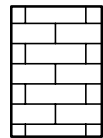
Clayey Silty Sand



Glacial Till



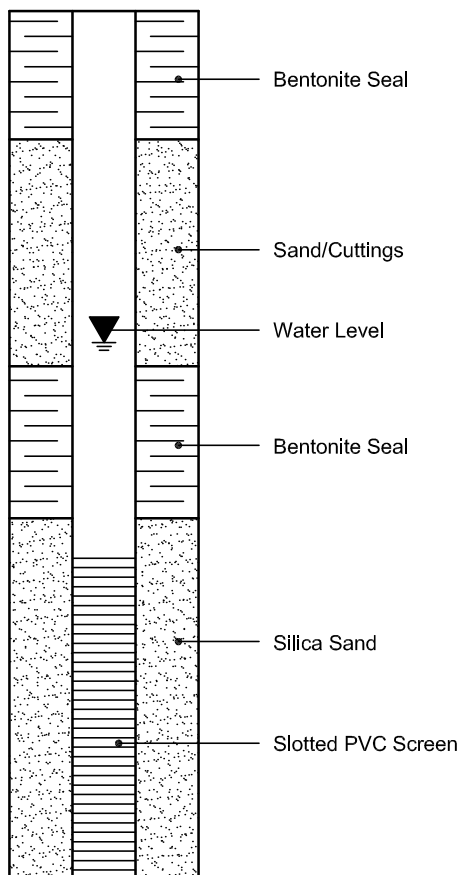
Shale



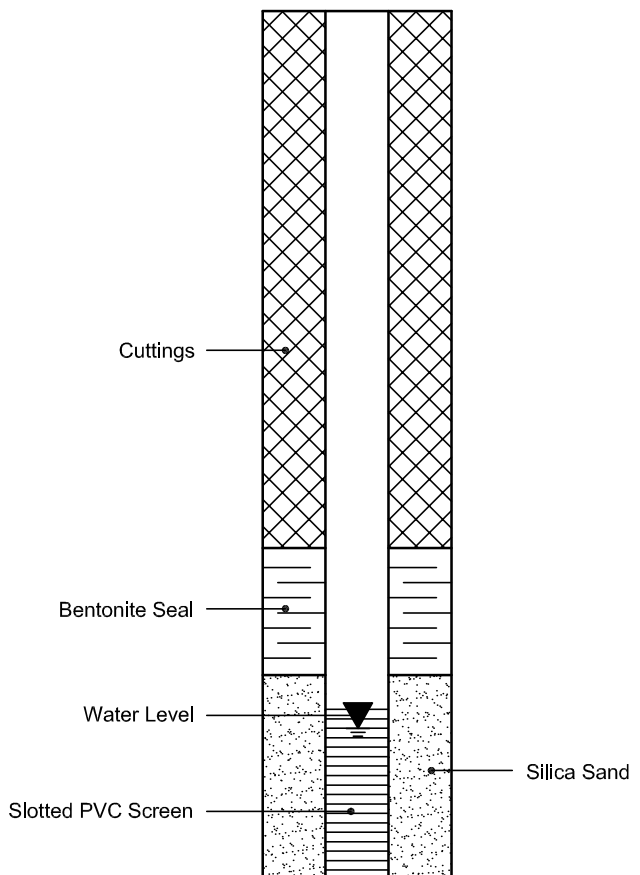
Bedrock

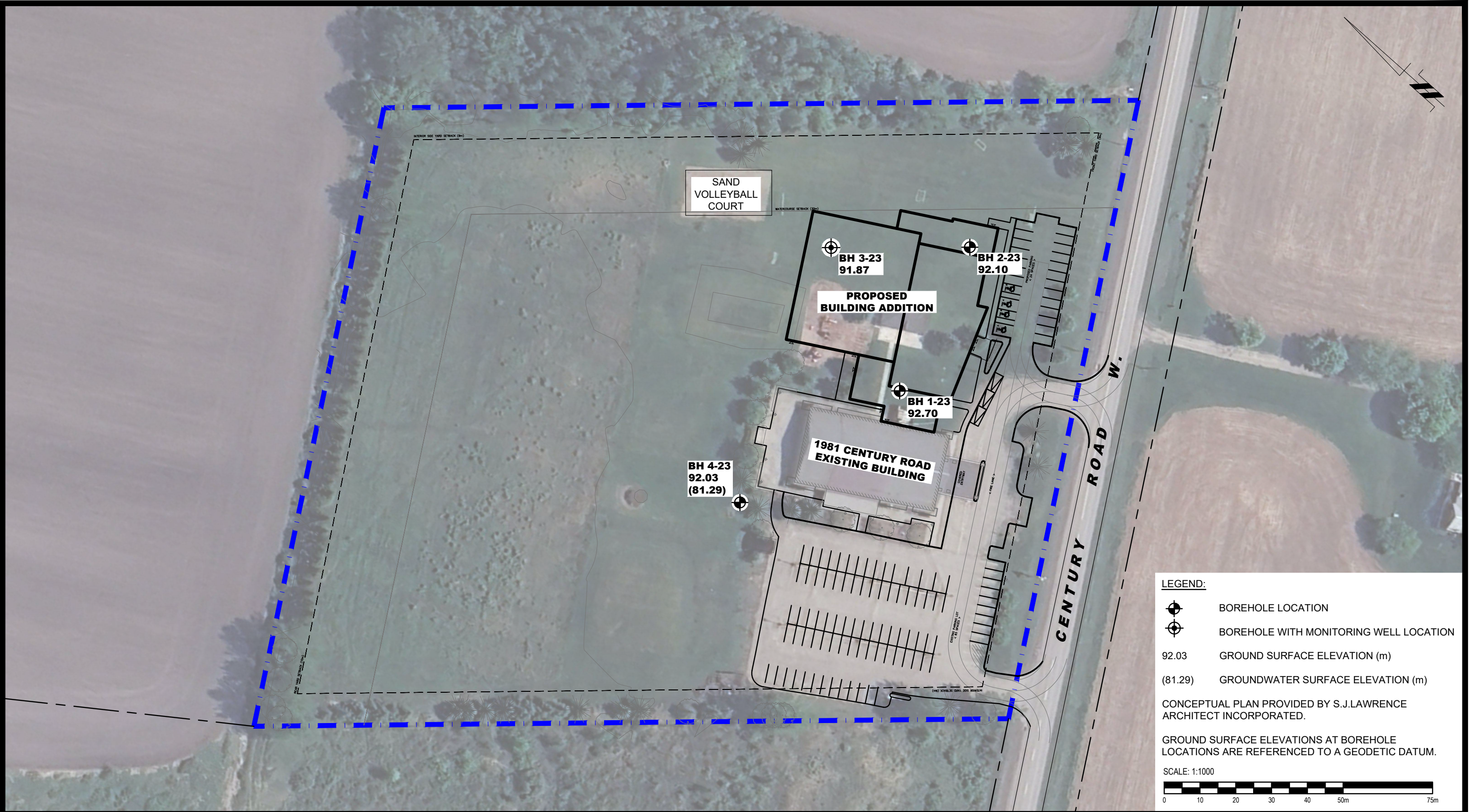
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





LEGEND:

 BOREHOLE LOCATION

 BOREHOLE WITH MONITORING WELL LOCATION

92.03 GROUND SURFACE ELEVATION (m)

(81.29) GROUNDWATER SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY S.J.LAWRENCE ARCHITECT INCORPORATED.

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

SCALE: 1:1000



 9 AURIGA DRIVE OTTAWA, ON K2E 7T9 TEL: (613) 226-7381				BRUNSTAD CHRISTIAN CHURCH OTTAWA GEOTECHNICAL INVESTIGATION PROPOSED BUILDING ADDITION 1981 CENTURY ROAD ONTARIO			Scale:	1:1000	Date:	09/2023
							Drawn by:	GK	Report No.:	PG6727-1
							Checked by:	KP	Dwg. No.:	PG6727-1
							Approved by:	SD	Revision No.:	
	NO.	REVISIONS	DATE	INITIAL	OTTAWA, Title:			TEST HOLE LOCATION PLAN		