PATERSON GROUP

Geotechnical Investigation

Proposed Building Addition

1981 Century Road
Ottawa, Ontario

Prepared for Brunstad Christian Church Ottawa

Report PG6727-1 Rev. 1 dated May 16, 2025

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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, and

- ❑ 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.

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 2,400 m^2. An asphalt-paved parking area will be located to the south of the proposed building addition, with landscaped areas to the east and north, and the existing hall immediately to the west.

It is further understood that proposed building addition will be serviced by a private well and septic system.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation 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. 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 current investigation, 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.

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.

3.2 Field Survey

The borehole locations, and the ground surface elevations at the borehole locations, were surveyed using a handheld GPS unit 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 Review

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.

All samples from the current investigation will be stored in the laboratory for 1 month after this report is completed. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. 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 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 elevation 92 m.

4.2 Subsurface Profile

Overburden

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 borehole BH 1-23.

The very stiff to firm, brown to grey clayey silt to silty clay deposit was encountered underlying the topsoil and/or fill, extending to depths of about 1.8 to 4 m below the existing ground surface. Atterberg limits testing results for the clayey silt to silty clay are presented in Table 1 below, and grain size distribution testing results are presented in Table 2 on the next page.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
BH 1-23 SS3	1.80	29	19	10	23.6	CL
BH 3-23 SS3	1.80	33	18	15	22.1	CL
BH 4-23 SS3	1.80	61	22	39	54.2	CH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content;
CL: Inorganic Clay of Low Plasticity
CH: Inorganic Clay of High Plasticity

Table 2 - Summary of Grain Size Distribution Analysis					
Test Hole	**Sample**	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
BH 1-23	SS4	0	3.3	77.2	19.5

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.

A glacial till deposit was encountered underlying the silty sand, consisting of a loose to compact, grey silty sand to sandy silt with varying amounts of gravel, cobbles, and boulders.

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 consists of dolomite of the Oxford Formation with a drift thickness of 5 to 10 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells on July 24, 2023.The measured groundwater levels are presented in Table 3 below.

Table 3 - Summary of Groundwater Level Readings				
Borehole Number	**Ground Surface Elevation (m)**	**Groundwater Level (m)**	**Groundwater Elevation (m)**	**Recording Date**
BH 1-23	92.70	1.78	90.92	July 24, 2023
BH 2-23	92.10	1.99	90.11	July 24, 2023
BH 3-23*	91.87	1.10	90.77	July 24, 2023
BH 4-23	92.03	1.25	90.78	July 24, 2023
Note: Ground surface elevations at borehole locations are referenced to a geodetic datum. * Denotes monitoring well location.				

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:

- ❑ lean concrete trenches which are bearing on the undisturbed, compact to dense glacial till, or

- ❑ helical piles which have all bearing plates embedded within the undisturbed, compact to dense glacial till.

Due to the presence of the clayey silt to silty clay deposit, permissible grade raise restrictions have 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 proposed building, zero-entry vertical trenches should be excavated at the footing locations to the undisturbed, compact to dense glacial till, and backfilled with lean concrete to the founding elevation (minimum 17 MPa 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying 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 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.

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 be used as an alternate to the lean concrete trenches for 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.

To achieve the capacities indicated above, each helical pile should be installed to a minimum final installation torque of 8,500 ft-lbs. Alternate helical pile configurations to those provided above may be utilized subject to the review and approval of Paterson.

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 D**. If a higher seismic site class (Class A or B) is required for the proposed building addition, 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) 2012.

Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill, the existing fill or native clayey silt to silty clay subgrade, approved by the geotechnical consultant at the time of excavation, will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade construction. Where the subgrade consists of the existing fill, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as OPSS Granular B Type II.

It is recommended that the upper 200 mm of sub-floor fill consist of OPSS Granular A crushed stone. All backfill materials required to raise grade within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of the material's SPMDD.

5.6 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** - Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil.	

Table 5 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE – Existing imported 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.

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 proposed building. 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 structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining, non-frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, can be used for this purpose.

Excavated on-site fill could also be re-used for backfilling the exterior sides of the foundation walls. However, this material would need to be maintained in an unfrozen state and at a suitable moisture content for compaction if it is to be re-used on-site for this purpose.

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.

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water.
Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

Impacts to Neighbouring Properties

As the proposed building addition will be a slab-on-grade structure which will not require a perimeter drainage system, long-term groundwater lowering is not anticipated, and therefore no adverse effects are expected to neighbouring properties as a result of groundwater lowering.

Further, as the proposed slab-on-grade structures 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 Landscaping 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 must be satisfied 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 as indicated procedural changes below.

- ❑ A small tree must be provided with a minimum 25 m^3 of available soil volume while a medium tree must be provided with a minimum of 30 m^3 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).

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.

6.8 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.

7.0 Recommendations

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

- ❑ Review of the Grading and Servicing Plans, once available.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Continuous observation of helical pile installations, if utilized for foundation support of the proposed building addition.
- ❑ 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.

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 by 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.

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Report Distribution:

- ❏ Brunstad Christian Church Ottawa (e-mail copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1981 Century Road, Ottawa, Ontario

PATERSON GROUP

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

PROJECT: Geotechnical Investigation - Proposed Church Addition

FILE NO. PG6727

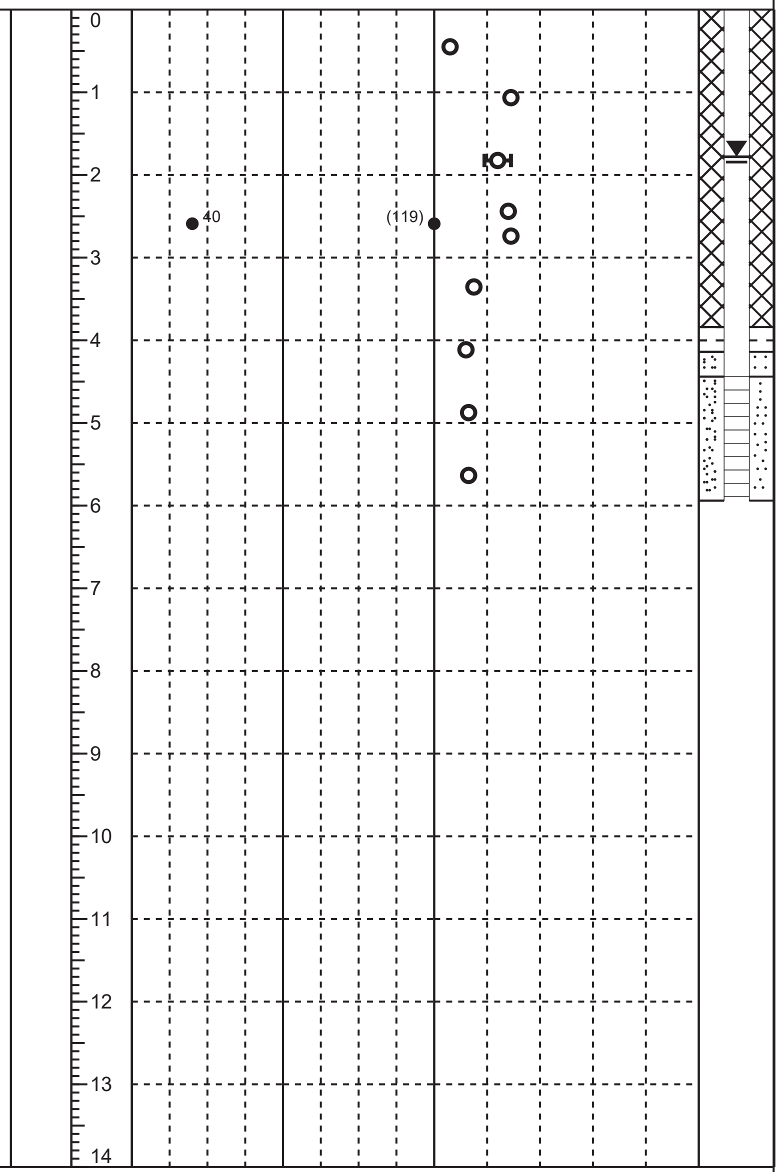
BORINGS BY: CME Low Clearance Drill

HOLE NO. BH 1-23

REMARKS:

DATE: July 17, 2023

SAMPLE DESCRIPTION	Elevation / Depth	SAMPLE No.	Type	SAMPLE % RECOVERY	N VALUE or RQD	DEPTH (m)
Ground Surface	EL 92.7 m					0
TOPSOIL	0.3 m, EL 92.4 m	AU1	Auger			
FILL: Brown silty sand	0.69 m, EL 92.01 m	SS2	Split spoon	50	6	1
Very stiff, brown CLAYEY SILT, trace sand		SS3	Split spoon	50	9	2
	2.97 m, EL 89.73 m	SS4	Split spoon	100	P	
Very loose, grey SILTY SAND, trace to some gravel	3.73 m, EL 88.97 m	SS5	Split spoon	25	2	3
GLACIAL TILL: Loose to compact, grey silty sand to sandy silt with gravel, cobbles and boulders		SS6	Split spoon	67	8	4
		SS7	Split spoon	83	8	5
	5.94 m, EL 86.76 m	SS8	Split spoon	42	15	
End of Borehole						6
(GWL @ 1.78m - July 24, 2023)						



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PATERSON GROUP

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

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: July 17, 2023

FILE NO. PG6727

HOLE NO. BH 2-23

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	DCPT	DEPTH (m)	Remoulded Shear Strength (kPa) 0 25 50 75 100	Peak Shear Strength (kPa) 0 25 50 75 100	Water Content (%) W_{PL} — W — W_{LL} 20 40 60 80	Piezometer Construction
Ground Surface EL 92.1 m							0				
TOPSOIL 0.33 m EL 91.77 m		AU1 AU2									
Very stiff, brown CLAYEY SILT, trace sand		SS3		100	6		1				
1.83 m EL 90.27 m		SS4		100	P		2	60	(139)		
Loose, brown SANDY SILT, trace clay 2.44 m EL 89.66 m		SS5		100	1						
Firm, grey CLAYEY SILT, trace sand 3.45 m EL 88.65 m		SS6		83	P		3		29		
GLACIAL TILL: Loose to compact, brown silty sand with gravel, cobbles and boulders, some clay		SS7		50	9		4				
		SS8		42	17		5				
		SS9		50	21		6				
6.71 m EL 85.39 m		SS10		42	14						
End of Borehole							7				
(GWL @ 1.99m - July 24, 2023)							8				
							9				
							10				
							11				
							12				
							13				
							14				

RSLog / Geotechnical Borehole - Geodetic - Atterberg / paterson-group / admin / September 05, 2023 11:40 AM

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1981 Century Road, Ottawa, Ontario

PATERSON GROUP

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

PROJECT: Geotechnical Investigation - Proposed Church Addition

BORINGS BY: CME Low Clearance Drill

REMARKS:

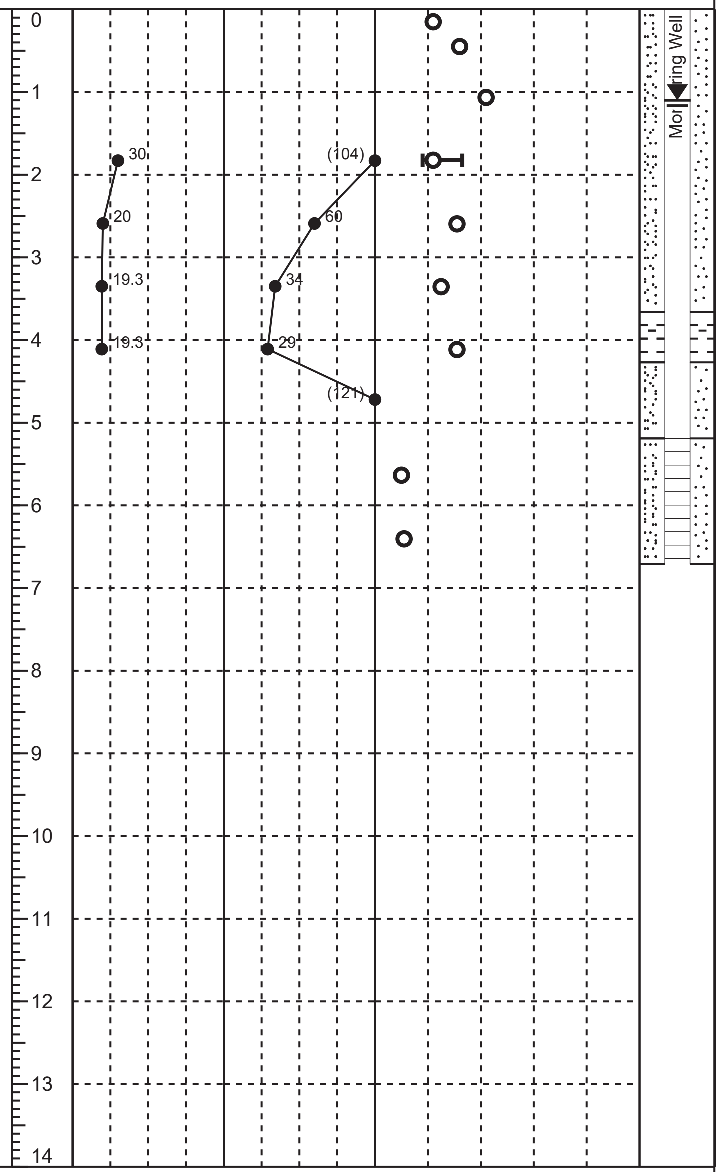
DATE: July 17, 2023

FILE NO. PG6727

HOLE NO. BH 3-23

SAMPLE DESCRIPTION	SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD
Ground Surface EL 91.87 m				
TOPSOIL 0.3 m EL 91.57 m	AU1			
	AU2			
Very stiff, brown CLAYEY SILT, trace sand	SS3		100	6
- sand content increasing with depth	SS4		67	P
2.29 m EL 89.58 m	SS5		58	P
Stiff to firm, grey CLAYEY SILT, trace sand	SS6		8	P
	SS7		75	P
4.88 m EL 86.99 m	SS8		0	P
GLACIAL TILL: Compact to dense, grey silty sand to sandy silt with gravel, cobbles and boulders	SS9		67	15
6.71 m EL 85.16 m	SS10		50	40
End of Borehole				
(GWL @ 1.10m - July 24, 2023)				

STRATA PLOT | DCPT | DEPTH (m) | Remoulded Shear Strength (kPa) | Peak Shear Strength (kPa) | Water Content (%) | Piezometer Construction



SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

1981 Century Road, Ottawa, Ontario

PATERSON GROUP

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

PROJECT: Geotechnical Investigation - Proposed Church Addition

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: July 17, 2023

FILE NO. PG6727

HOLE NO. BH 4-23

SAMPLE DESCRIPTION	SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	DCPT	Remoulded Shear Strength (kPa)	Peak Shear Strength (kPa)
Ground Surface EL 92.03 m							
TOPSOIL (0.3 m, EL 91.73 m)	AU1						
	AU2						
Stiff to firm, brown SILTY CLAY, trace to some sand	SS3		100	6			
	SS4		100	P		20	89
2.59 m, EL 89.44 m							
Firm, grey CLAYEY SILT, trace sand	SS5		67	P		10	30
	SS6		33	P		12.1	43
3.96 m, EL 88.07 m							
Loose, grey SILTY SAND, trace clay and gravel	SS7		75	7			
4.72 m, EL 87.31 m							
GLACIAL TILL: Loose to compact, grey silty sand to sandy silt with gravel, cobbles and boulders	SS8		100	6			
	SS9		42	19			
6.1 m, EL 85.93 m							
Dynamic Cone Penetration Test commenced at 6.10m depth.					25 34 54 33 42 46 45 48 24 38 51 56 42 43 50 80 100		
10.74 m, EL 81.29 m							
End of Borehole							
Practical DCPT refusal at 10.74m depth.							
(GWL @ 1.25m - July 24, 2023)							

STRATA PLOT | DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14

Water Content (%): W_{PL} — W — W_{LL} : 20 40 60 80

Piezometer Construction: Piezometer

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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

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)
Dxx	-	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
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = $D60 / D10$

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

CONSOLIDATION TEST

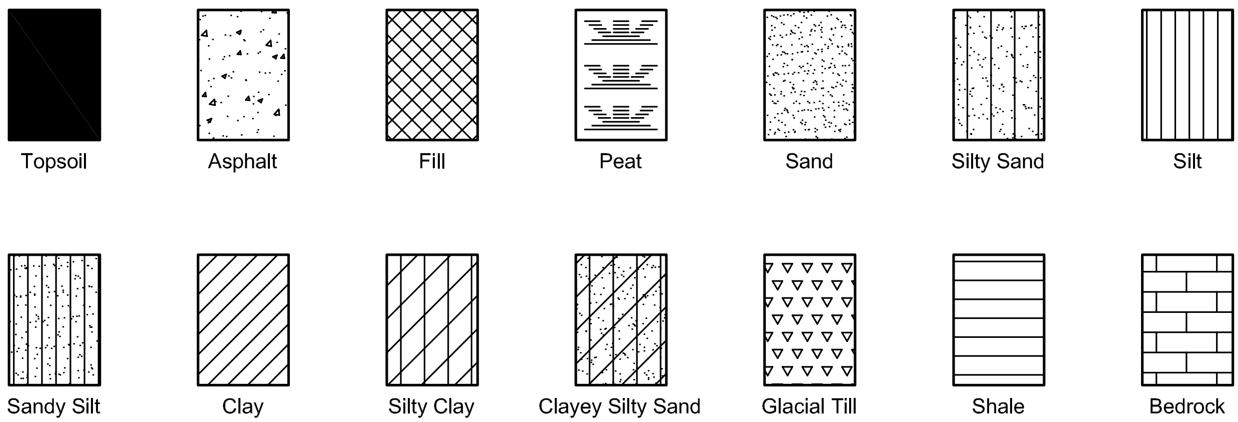
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

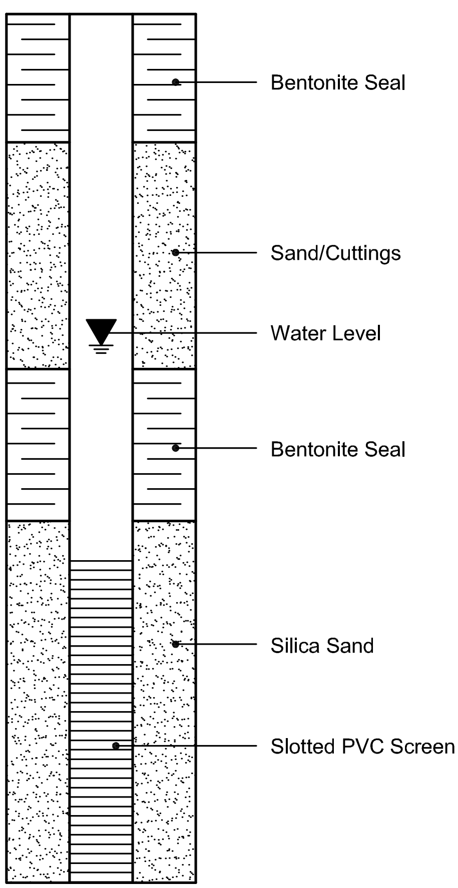
SYMBOLS AND TERMS (continued)

STRATA PLOT

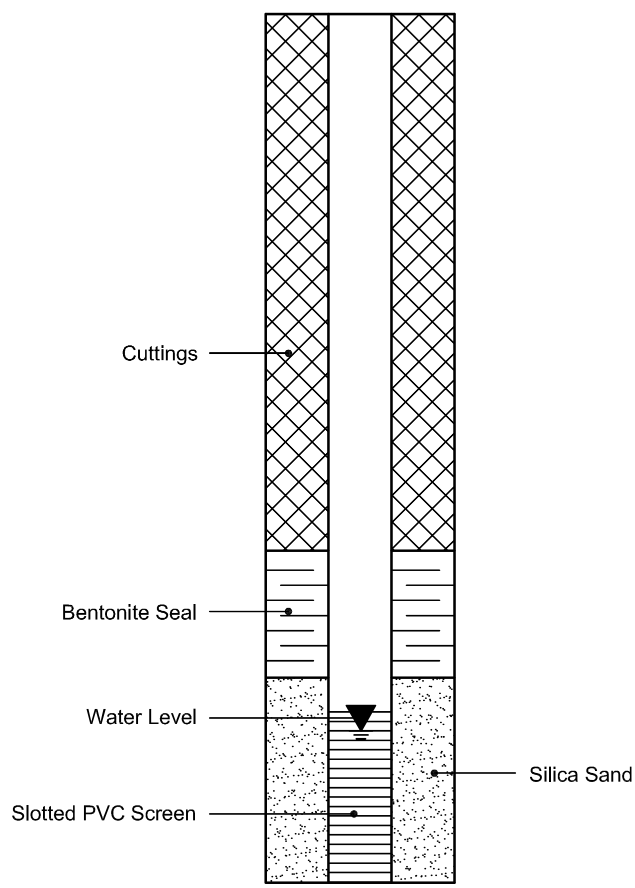


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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

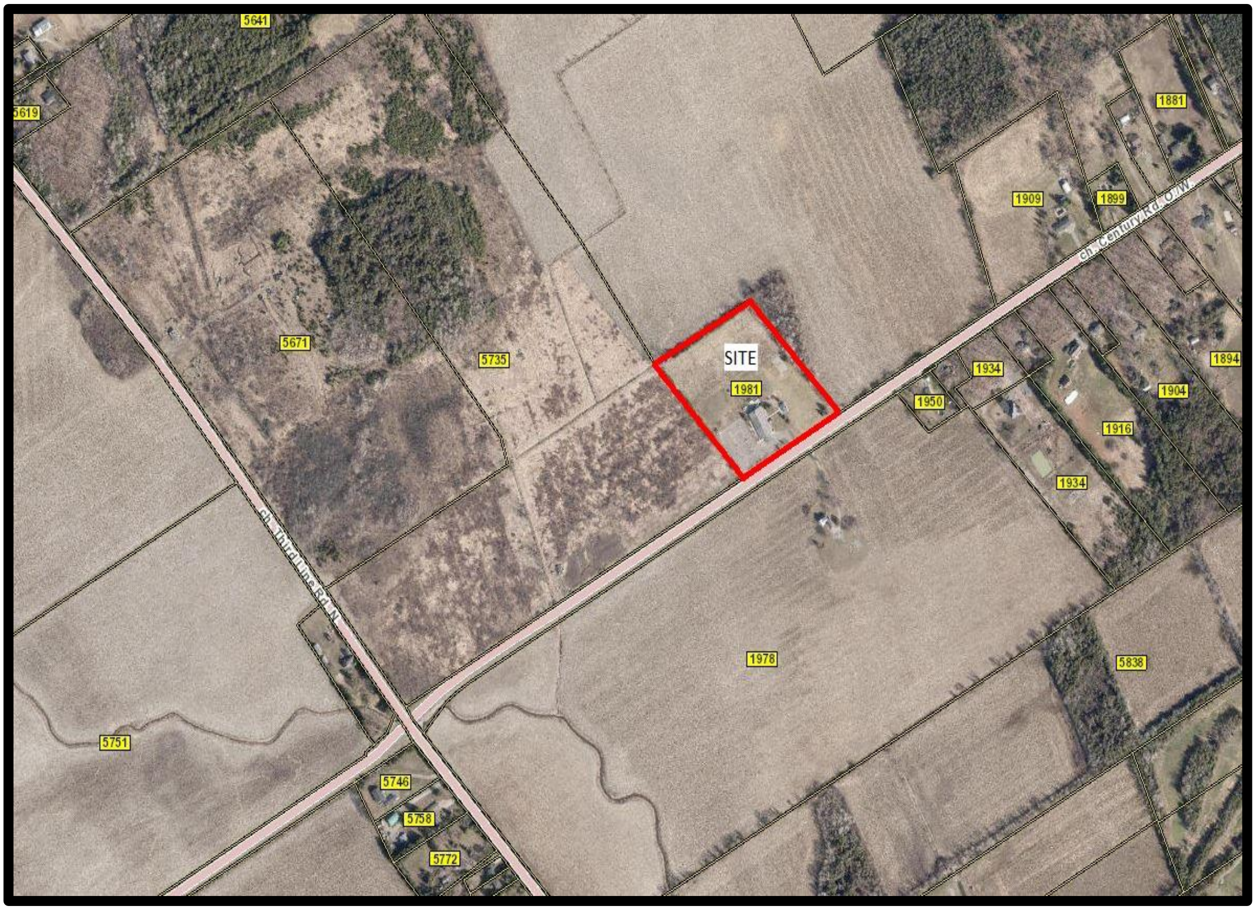
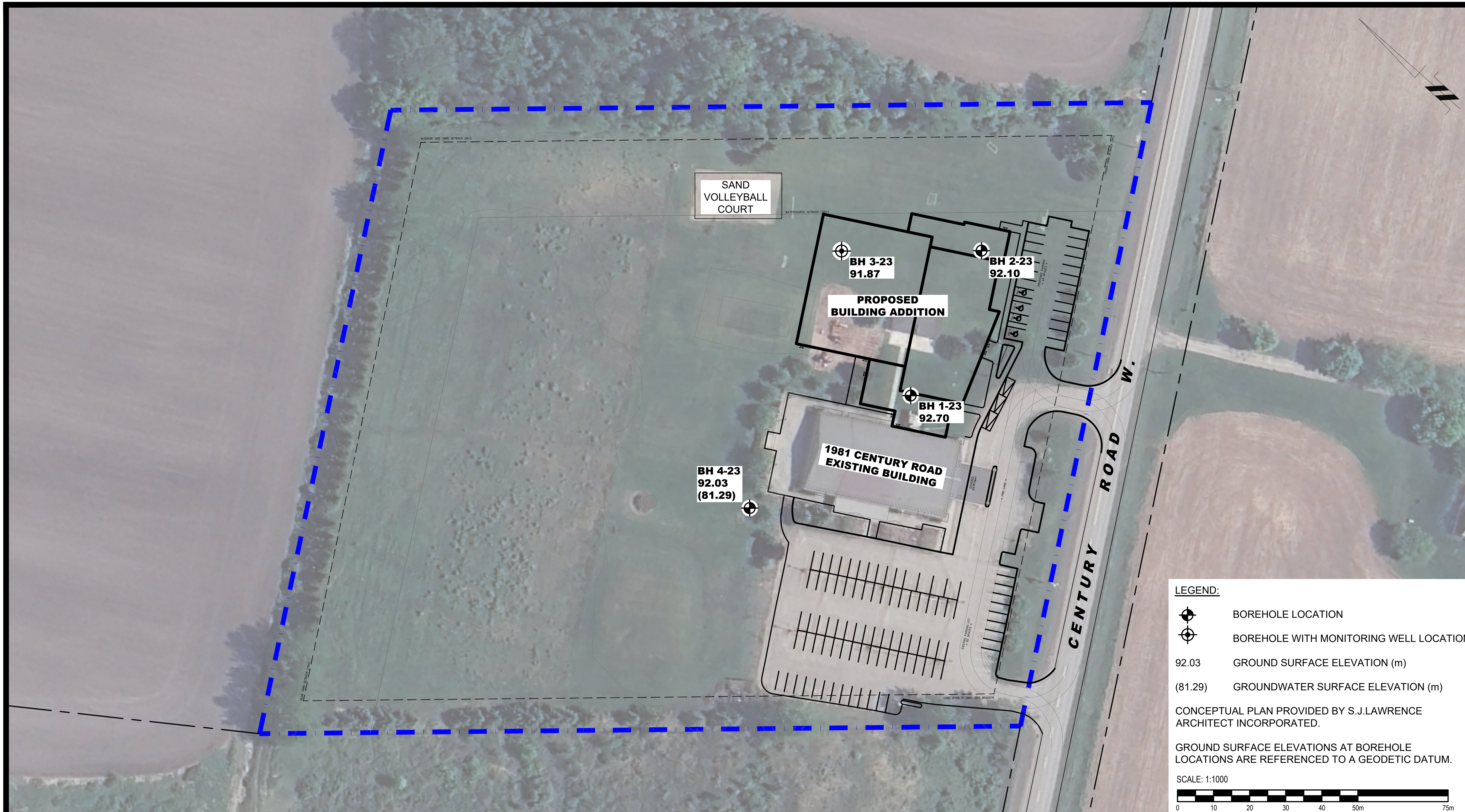


FIGURE 1

KEY PLAN

PATERSON GROUP



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

0 10 20 30 40 50m 75m

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

NO.	REVISIONS	DATE	INITIAL

BRUNSTAD CHRISTIAN CHURCH OTTAWA

GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING ADDITION
1981 CENTURY ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

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.:

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11x17

APPENDIX 3

GROUNDWATER MONITORING PROGRAM

PH4720-MEMO.02 DATED AUGUST 23, 2024

memorandum

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 where 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.Geo., géo.

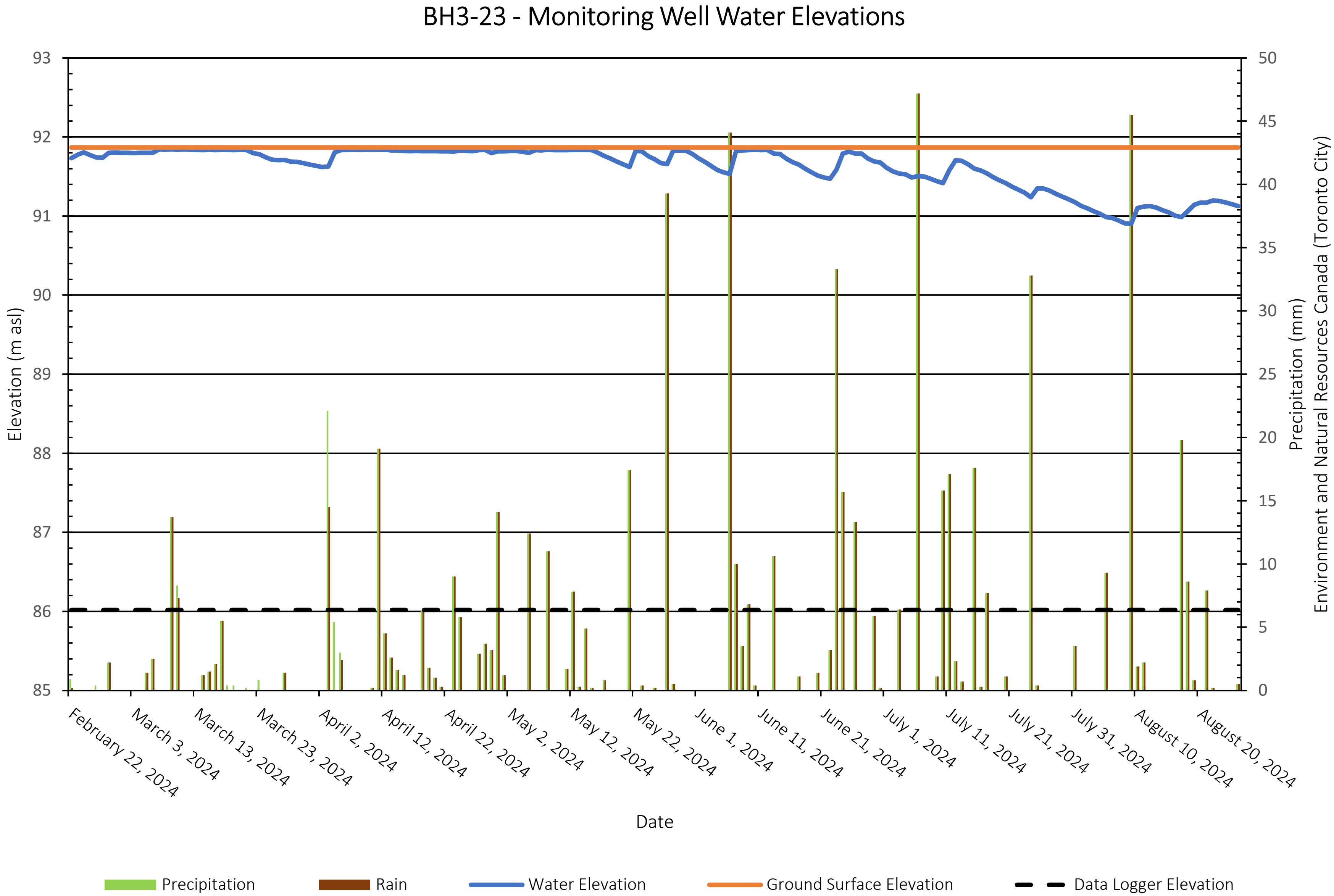
Erik Ardley, P.Geo.

Attachments

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

File: PH4720

Brunstad Christian Church
1981 Century Road, Ottawa, ON
Monitoring Well Water Elevations



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