Geotechnical Engineering

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Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7S8

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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Geotechnical Investigation

Proposed Addition to Existing Church
205 Greenbank Road
Ottawa, Ontario

Prepared For

Woodvale Pentecostal Church

September 10, 2021

Report: PG5951-1

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Appendices

1.0 Introduction

Paterson Group Inc. (Paterson) was commissioned by Woodvale Pentecostal Church to prepare a Geotechnical Investigation Report for the proposed addition to the existing church located at 205 Greenbank Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report). The objectives of the Geotechnical Investigation Report were to:

- ❑ Review the subsoil and groundwater conditions at this site using the factual information from previous boreholes that were completed at the site.
- ❑ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

Paterson conducted a previous geotechnical investigation at the site prior to the construction of the current church structure, as detailed in Report No. PG0193-1, dated May 21, 2004. At that time additional boreholes were put down in the location of a future addition that is now the subject of this report.

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 proposed development as they are understood at the time of this report.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed building addition will be located along the southwest side of the existing church and will consist of a two-storey structure with an approximate footprint of 1,300 m^2. A patio is also proposed along the south and west sides of the addition.

3.0 Available Subsurface Information

3.1 Previous Field Investigation

The fieldwork for the previous geotechnical investigation, referenced above, was carried out on May 4, 2004 and consisted of five (5) deep boreholes, of which two (2) boreholes were located within the footprint of the proposed addition. These latter boreholes, labelled BH 4 and BH 5, were sampled to approximate depths of 7.3 metres and then further extended with a cone to the depths of practical refusal. The locations of these boreholes are shown on Drawing PG5951-1 - Test Hole Location Plan in Appendix 2.

The boreholes were advanced using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations and sampling and testing the overburden prior to completing the boreholes with a cone probe.

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. 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 presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing using a vane apparatus was carried out in cohesive soils.

The thickness of the silty clay deposit and the depth to the inferred bedrock surface were evaluated during the course of the investigation by the use of dynamic cone penetration tests (DCPTs). The DCPT consists of driving a steel 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 of penetration.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles at the five boreholes within the existing building and the addition area are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report. In particular, boreholes BH 4 and BH 5 are located within the footprint of the proposed addition.

Groundwater

Flexible PVC standpipes were installed in the boreholes to permit monitoring of the groundwater levels subsequent to completion of the sampling program.

3.2 Field Survey

The ground surface elevation at each test hole location was referenced to a temporary benchmark (TBM), which consists of the top spindle of the fire hydrant located on the east side of Greenbank Road, south of Bateman Drive. A geodetic elevation of 93.51 m was provided for the TBM on a survey plan prepared by Farley Smith & Denis Surveying Limited. The location of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG5951-1 - Test Hole Location Plan in Appendix 2.

The reader should be aware that the ground elevations provided for the boreholes were accurate at the time of the 2004 fieldwork program but have changed slightly since that time based on the 2004 church construction work. The profiles below the surficial fill materials are expected to represent current subsurface conditions.

3.3 Laboratory Testing

Soil samples were recovered from the subject site at the time of the previous geotechnical investigation and visually examined in our laboratory to review the results of the field logging.

One (1) soil sample from the 2004 investigation was submittted 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 sample. The results are presented in Appendix 1 and discussed further under subsection 6.6

4.0 Observations

4.1 Surface Conditions

The greater part of the subject site is currently occupied by the existing Woodvale Pentecostal Church and associated parking and landscaped areas.

The subject site is bordered to the north and east by Bateman Drive, to the west by Greenbank Road, and by a police station to the south. The ground surface across the subject site is relatively level at approximate elevations of 92.5 to 93.5 m.

4.2 Subsurface Profile

Overburden

The primary focus of this interpretation will be on the conditions at boreholes BH 4 and BH 5 within the proposed addition area, although all five (5) deep borehole logs are provided in Appendix 1. Generally, the subsurface profile encountered at boreholes BH 4 and BH 5 consists of an approximate 40 mm thickness of asphaltic concrete overlying fill, which in turn overlies native silty clay.

The fill material generally consists of silty sand with crushed gravel, and was observed to extend to an approximate depth of 0.6 m below the ground surface.

For the silty clay deposit, which was encountered underlying the fill, the upper portion of the silty clay has developed a brown desiccated crust. In situ shear vane field tests carried out within the silty clay crust yielded peak undisturbed shear strength values in excess of 120 kPa. These values reflect a very stiff consistency in the silty clay crust.

Grey silty clay was encountered below the brown silty clay crust at approximate depths of between 5 and 6 m below ground surface. One in situ shear vane field test conducted within the grey silty clay layer yielded an undisturbed shear strength value of 80 kPa. This value is indicative of a stiff consistency.

Practical refusal to the DCPT was encountered at approximate depths of 10.3 and 11.8 m below the 2004 ground surface in boreholes BH 4 and BH 5, respectively, relating to elevations of 82.9 and 81.1 m, respectively.

Bedrock

Based on available geological mapping, bedrock in the area of the subject site consists of interbedded sandstone and dolomite of the March Formation with an overburden drift thickness of 15 to 25 m.

4.3 Groundwater

Groundwater level readings were measured in the standpipes on May 10, 2004. The measured groundwater level (GWL) readings are presented in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Surface Elevation (m)	Groundwater Levels (m)	Groundwater Elevation (m)	Recording Date
BH 1	93.27	2.30	90.97	May 10, 2004
BH 2	92.68	1.79	90.89	
BH 3	93.34	2.27	91.07	
BH 4	93.19	2.37	90.82	
BH 5	92.95	4.31	88.64	

Note: The elevation of the groundwater surface at each borehole location is referenced to the top spindle of the fire hydrant located on the east side of Greenbank Road, south of Bateman Drive, to which an elevation of 93.51 m was assigned as shown on the base plan prepared by Farley Smith & Denis Surveying Limited for the 2004 project.

It should be noted that the recorded groundwater levels could have been influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour, moisture content and consistency of the recovered soil samples.

Based on these observations, the long-term low groundwater level is expected to be located between approximately 4 and 5 m depth, with the normal groundwater level being at 1.8 to 2.5 m depth. However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed two storey basementless slab-on-grade building addition. It is recommended that the proposed addition be founded on conventional spread footings placed on an undisturbed, very stiff silty clay bearing surface.

Due to the presence of the silty clay deposit, the proposed addition will be subjected to grade raise restrictions.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill should be stripped from under the proposed building addition and other settlement sensitive structures. However, it is anticipated, based on the available information, that the existing fill within the future building footprint, free of deleterious material and significant amounts of organics, can be left in place below the proposed building addition footprint outside of lateral support zones for the footings.

It is recommended that the existing fill layer be proof-rolled several times **under dry conditions and above freezing temperatures** and approved by Paterson personnel at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

Fill Placement

Fill used for grading beneath the proposed 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.

5.3 Foundation Design

Square spread footings with a maximum width of 3 m placed on an undisturbed, very stiff silty clay bearing surface at or above geodetic elevation 91.3 m can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ULS of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance at ULS.

Strip footings with a maximum width of 2 m placed on an undisturbed, very stiff silty clay bearing surface at or above geodetic elevation 91.3 m can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, disturbed or frozen soil, have been removed, in the dry, prior to placement of concrete footings.

Footings designed using the bearing resistance at SLS values provided above will be subjected to potential post-construction total and differential settlement of 25 and 20 mm, respectively.

As a general procedure, it is recommended that the footings for the proposed addition that are located adjacent to the existing structure be founded at the same level as the existing footings. This accomplishes three objectives. First, the long-term behaviour of the two structures at their connection will be similar due to the similar bearing medium. Second, there will be minimal stress added to the existing structure from the new structure. Third, the bearing of the new structure will not be influenced by any backfill from the existing structure. Note that there will still be a potential that the new addition will settle differentially with respect to the existing structure to the limits noted above.

It should be noted that the potential total settlement of the proposed addition will be differential with respect to the existing church building. Provision should be made in the design of the addition to accommodate a potential differential settlement of 25 mm between the existing building and its addition.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a silty clay 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.

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. This restriction is not expected to be an issue, as the addition will have the same ground floor level as the existing building.

5.4 Design for Earthquakes

Analyses have been conducted (Table 4 in Appendix 1) to evaluate the site class for seismic site response for the subsurface profile at all five deep boreholes using the equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012. **Class C** is the applicable site class for seismic site response at this site, as presented in Table 4.1.8.4.A of the OBC 2012.

Soils underlying the subject site are not susceptible to seismic liquefaction. Reference should be made to the latest revision of the OBC 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 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

If required as part of the proposed scope of the building addition, pavement structures for car only parking areas, heavy truck parking areas and access lanes are presented in Tables 2 and 3, below.

Table 2 - 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 3 - 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.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping 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 its load carrying capacity.

The subgrade soil will consist predominantly of silty clay. Because of the impervious nature of these materials, consideration should be given to installing subdrains at each catch basin. These drains should be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. 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. Discharge of the subdrains should be directed by gravity to storm sewers or deeper drainage ditches.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 150 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 pipes should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

Backfill material against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as Delta Drain 6000) connected to a drainage system is provided. Imported granular materials, such as clean sand or OPSS Granular B Type I material should be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as isolated piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure 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

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, the shallow depth of excavation, and the proposed building setback from the property lines, it is anticipated that sufficient space will be available to open cut and slope the excavation sides.

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 (if required) 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.

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 fill or silty sand. If the bedding is placed on silty clay, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 99% 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, from the “crust”, 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. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

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

The silty clay deposit encountered at the site is very stiff to stiff and is considered to be low to medium sensitivity clay, from a tree planting perspective. Therefore, where footings are founded over a silty clay bearing surface, large trees (mature height over 14 m) can be planted provided a tree to foundation setback equal to the full mature height of the tree is utilized (i.e. 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). It should be noted that shrubs and other small plantings are permitted within the **4.5 m** setback area.

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

The results of the analytical testing show that the sulphate content is less than 0.1% (1 mg/g or 1000 ug/g). This result is indicative that Type GU (general use) cement, as per CSA A23.1 Section 4.2.1.1.2, is appropriate for concrete mixes to be in contact with the ground at this site.

The chloride content is less than 2000 ug/g and the pH of the sample is greater than 5. These results indicate that they are not significant factors in creating a corrosive environment for ferrous metals at this site. The resistivity value is greater than 1,500 ohm-cm and is indicative of a moderate to slightly aggressive corrosive environment.

The appropriate concrete exposure class is “N”, for soil content based on chloride content, where freezing and thawing (F-1 or F-2 exposure class) is not an issue.

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 plan, once available.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

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

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

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

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

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Sept. 10, 2021
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Scott S. Dennis, P.Eng.

Andrew J. Tovell, P.Eng.

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

TABLE 4: BH SUMMARY OF SEISMIC SITE CLASS

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154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Woodvale Pentecostal Church, 205 Greenbank Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Elevation = 93.51m.

FILE NO. PG0193

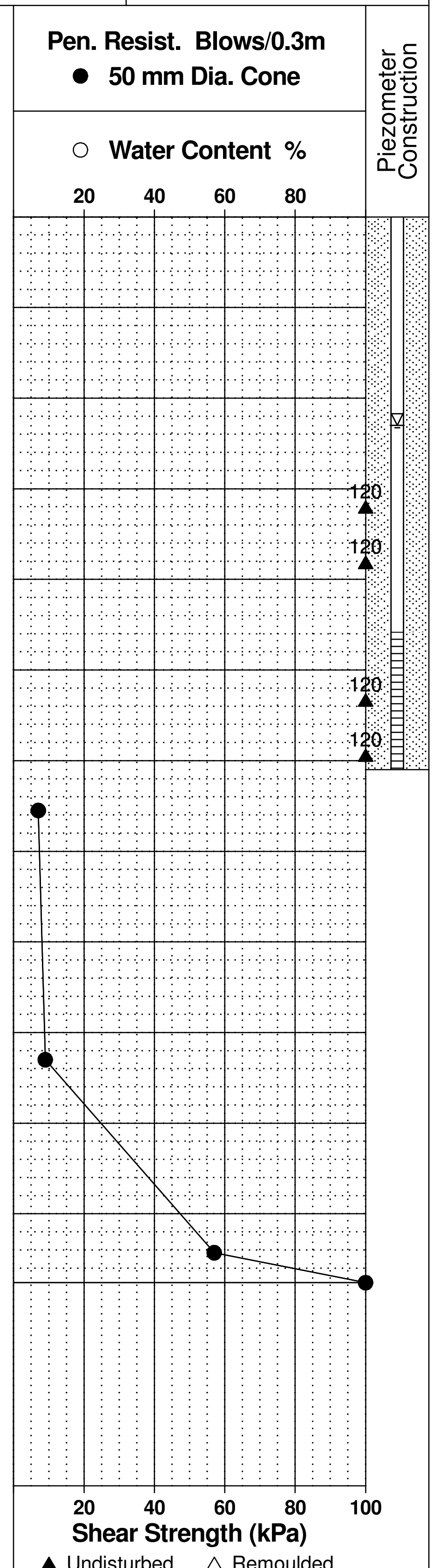
REMARKS

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger **DATE** May 4, 04

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.09				
FILL: Brown silty fine to medium sand with gravel	0.76	AU	1		
Very stiff, brown **SILTY CLAY**, frequent sand seams		SS	2	83	17
		SS	3	100	18
		SS	4	100	8
		SS	5	100	W
		SS	6	100	5
- grey-brown by 5.3m depth	6.10				
Dynamic Cone Penetration Test commenced @ 6.10m depth.	11.76				
End of Borehole					
Cone refusal @ 11.76m depth					
(GWL @ 2.30m-May 10/04)					

DEPTH (m)	ELEV. (m)
0	93.27
1	92.27
2	91.27
3	90.27
4	89.27
5	88.27
6	87.27
7	86.27
8	85.27
9	84.27
10	83.27
11	82.27



patersongroup Consulting Engineers

154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Woodvale Pentecostal Church, 205 Greenbank Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Elevation = 93.51m.

REMARKS

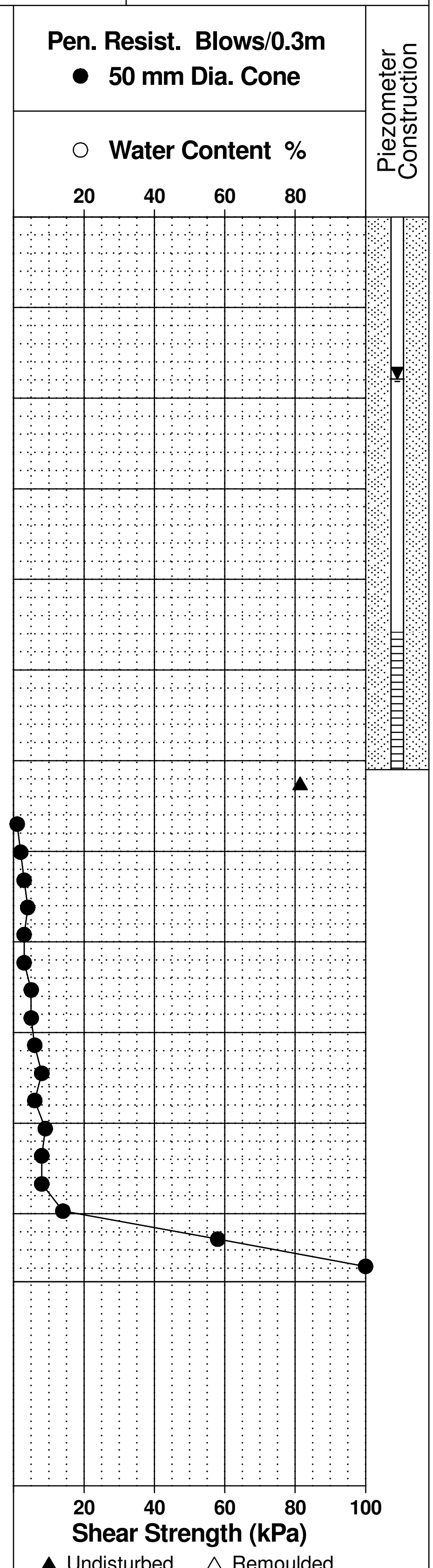
BORINGS BY CME 55 Power Auger

DATE May 4, 04

FILE NO. PG0193

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.68
TOPSOIL 0.15							
FILL: Brown silty fine to medium sand with gravel 0.76		AU	1				
						1	91.68
Very stiff to stiff, brown **SILTY CLAY**, frequent sand seams		SS	2	83	13	2	90.68
		SS	3	100	8	3	89.68
- grey-brown by 3.8m depth		SS	4	100	11	4	88.68
		SS	5	100	7	5	87.68
- grey by 5.3m depth		SS	6	100	6	6	86.68
6.40		SS	7	100	6		
Dynamic Cone Penetration Test commenced @ 6.4m depth						7	85.68
						8	84.68
						9	83.68
						10	82.68
						11	81.68
11.75							
End of Borehole Cone refusal @ 11.75m depth (GWL @ 1.79m-May 10/04)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Woodvale Pentecostal Church, 205 Greenbank Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Elevation = 93.51m.

REMARKS

BORINGS BY CME 55 Power Auger

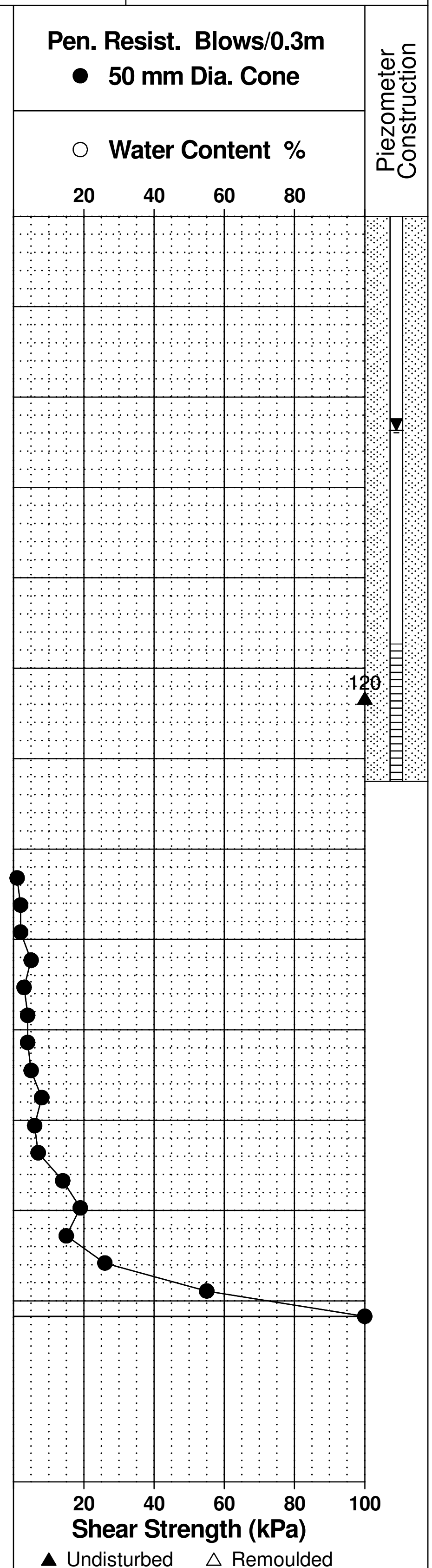
DATE May 4, 04

FILE NO. PG0193

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.13				
FILL: Brown silty fine to medium sand with gravel and brick fragments	0.60	AU	1		
Dense, brown **SANDY SILT**	1.37	SS	2	62	30
Very stiff, brown **SILTY CLAY**, frequent sand seams - grey-brown by 3.8m depth	7.32	SS	3	4	30
		SS	4	79	15
		SS	5	100	11
		SS	6	100	5
		SS	7	100	4
		SS	8	100	5
Dynamic Cone Penetration Test commenced @ 7.32m depth	12.17				
End of Borehole Cone refusal @ 12.17m depth (GWL @ 2.27m-May 10/04)					

DEPTH (m)	ELEV. (m)
0	93.34
1	92.34
2	91.34
3	90.34
4	89.34
5	88.34
6	87.34
7	86.34
8	85.34
9	84.34
10	83.34
11	82.34
12	81.34



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154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Woodvale Pentecostal Church, 205 Greenbank Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Elevation = 93.51m.

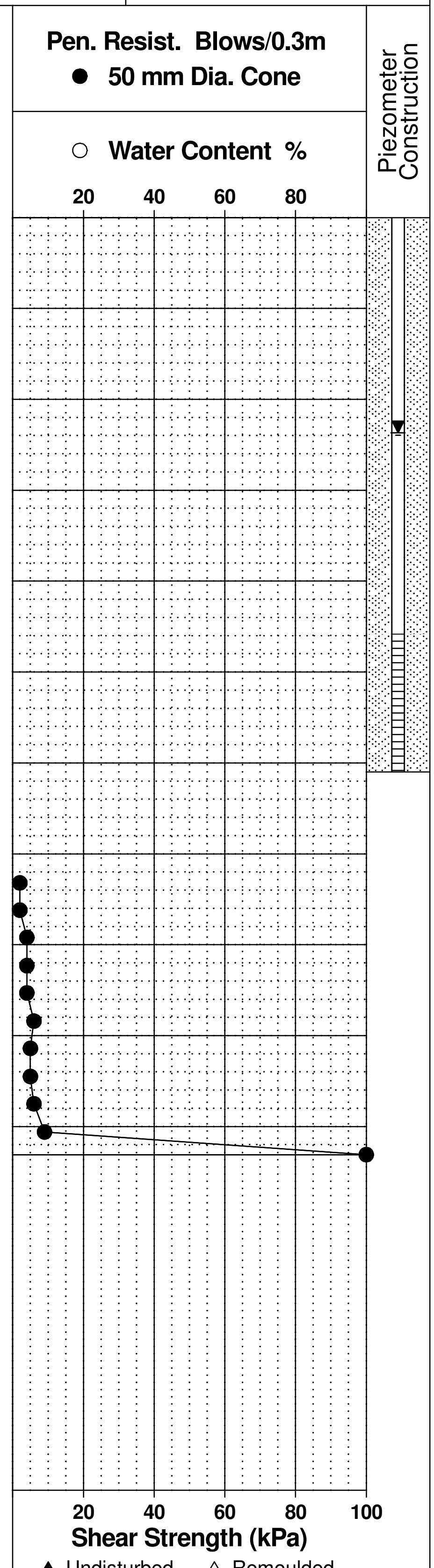
FILE NO. PG0193

REMARKS

HOLE NO. BH 4

BORINGS BY CME 55 Power Auger **DATE** May 4, 04

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.19
Asphaltic concrete	0.04	AU	1				
FILL: Brown silty fine to medium sand with gravel	0.60	SS	2	79	11	1	92.19
Very stiff, brown **SILTY CLAY**, frequent sand seams		SS	3	100	13	2	91.19
		SS	4	100	9	3	90.19
- grey brown by 3.8m depth		SS	5	100	12	4	89.19
		SS	6	100	10	5	88.19
- grey by 5.3m depth		SS	7	100	5	6	87.19
	7.32	SS	8	100	3	7	86.19
Dynamic Cone Penetration Test commenced @ 7.32m depth	10.31					8	85.19
						9	84.19
						10	83.19
End of Borehole Cone refusal @ 10.31m depth (GWL @ 2.37m-May 10/04)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Woodvale Pentecostal Church, 205 Greenbank Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant (see plan). Elevation = 93.51m.

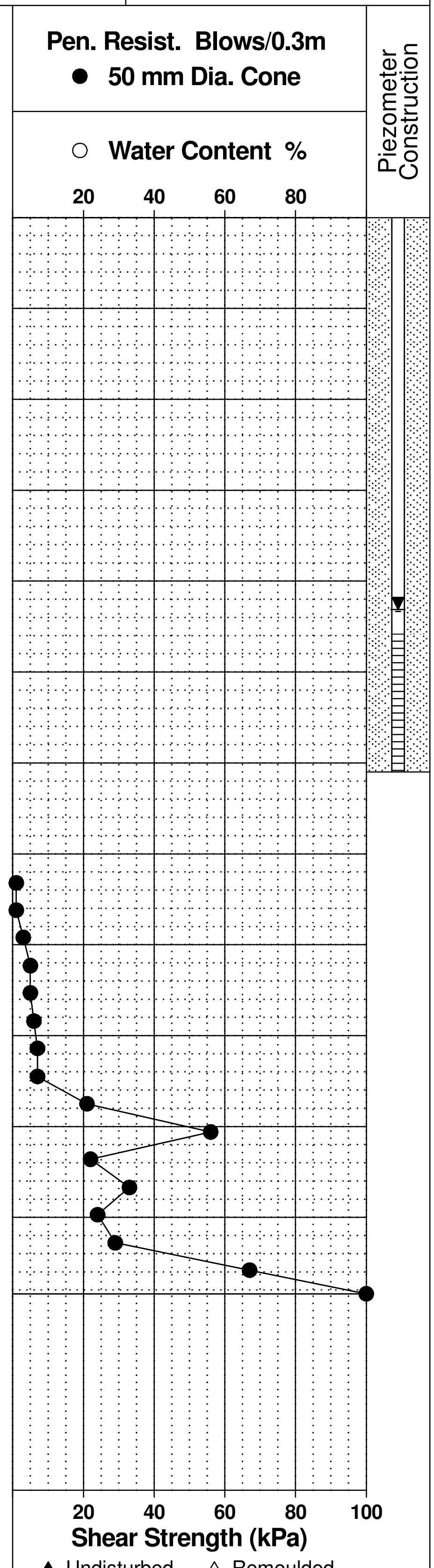
FILE NO. PG0193

REMARKS

HOLE NO. BH 5

BORINGS BY CME 55 Power Auger **DATE** May 4, 04

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.95
Asphaltic concrete 0.04							
FILL: Brown silty fine to medium sand with gravel 0.60		AU	1				
		SS	2	83	8	1	91.95
		SS	3	100	6	2	90.95
		SS	4	100	6		
Very stiff, brown **SILTY CLAY**		SS	5	100	7	3	89.95
		SS	6	100	6	4	88.95
		SS	7	100	6	5	87.95
- grey by 5.3m depth		SS	8	100	1	6	86.95
7.32						7	85.95
Dynamic Cone Penetration Test commenced @ 7.32m depth						8	84.95
						9	83.95
						10	82.95
						11	81.95
11.84							
End of Borehole Cone refusal @ 11.84m depth (GWL @ 4.31m-May 10/04)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

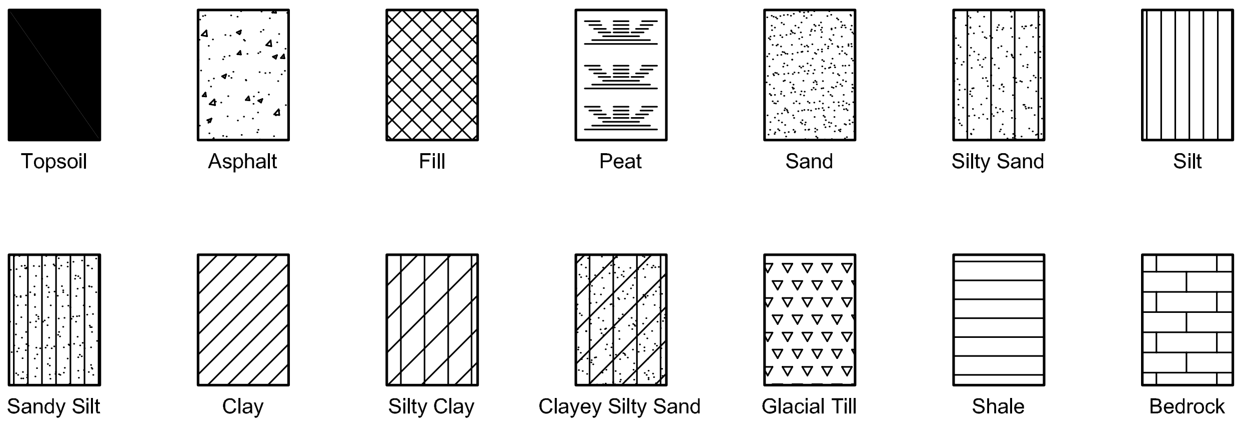
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

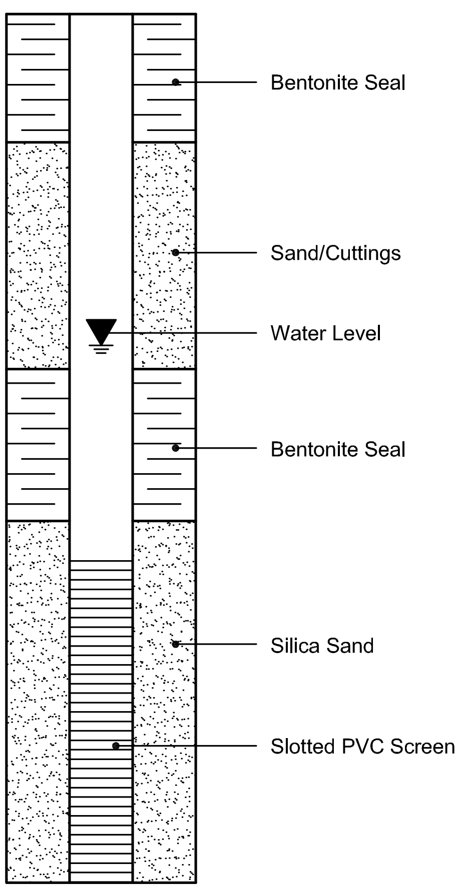
SYMBOLS AND TERMS (continued)

STRATA PLOT

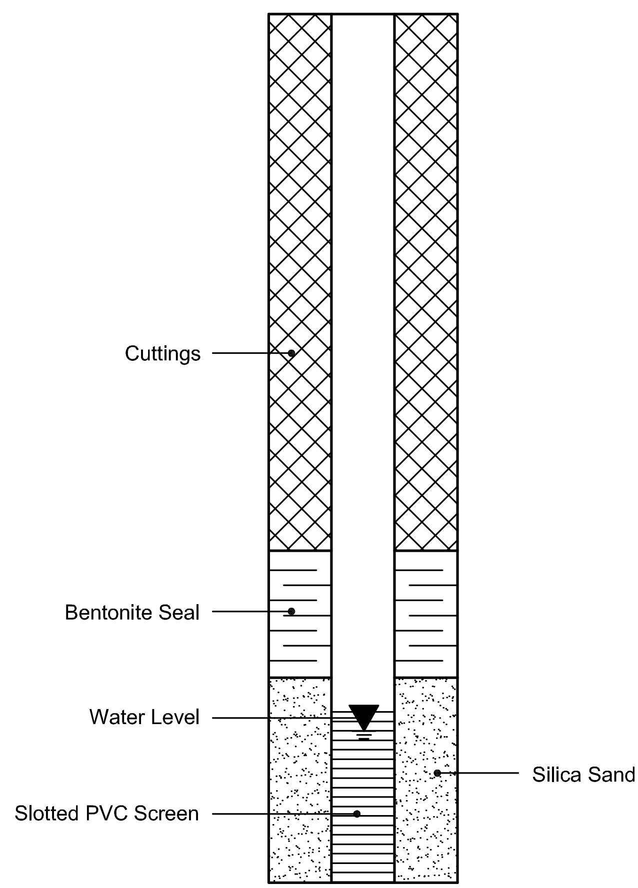


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Paracel Laboratories Ltd. **Order #: J1400**

Certificate of Analysis

Report Date: 12-May-2004
Order Date: 06-May-2004

Client: **Paterson Group Inc.**
Client PO: **3123** Project: **PG0193**

Matrix: Soil
Sample Date: 04/05/2004

Parameter	MDL/Units	BH1 SS3
		J1400.1
Chloride	5 ug/g	30
Sulphate	5 ug/g	70
pH	0.05 pH units	7.25
Resistivity	0.1 ohm.m	51

Table 4: BH Summary of Seismic Site Class - OBC 2012 - Using Typical Vs Values

Project:	**Woodvale Pentecostal Church - Additions and Renovations**				**Note: Analyses Use Approximate Interior USF**			
Report No:	PG5951-1	**Date:**	September 10, 2021		**Level of 92.0 m.**			
					Analyses based on representative Vs values			
PGA	0.32	**Region:**	City of Ottawa (Nepean)		**for BH stratigraphy and bedrock depths are**			
					based on inferred bedrock surface levels.			

Layer Description	**Layer Vs**	**Depths (m) of Various Layers at Specified Borehole**						
		BH 1	**BH 2**	**BH 3**	**BH 4**	**BH 5**	**BH**	**BH**
silty clay crust	200	5.0	4.5	4.5	4.0	4.2	0.0	0.0
grey silty clay (input)	128	5.1	5.8	5.2	4.8	4.6	0.0	0.0
Vs by Equation (= 125 + 1.1667*Z)		130.9	131.0	130.7	130.1	130.1	125.0	125.0
sandy silt to silty fine sand	200	0.0	0.0	0.0	0.0	0.0	0.0	0.0
post-glacial clay	200	0.0	0.0	0.0	0.0	0.0	0.0	0.0
glacial till	300	0.4	0.8	1.2	0.3	2.0	0.0	0.0
weathered bedrock	1200	1.0	1.0	1.0	1.0	1.0	0.0	0.0
sound sandstone bedrock	2000	18.5	17.9	18.1	19.9	18.2	0.0	0.0
Total of Thicknesses:	**N/A**	**30**	**30**	**30**	**30**	**30**	**0**	**0**
More than 3 m Soft Soil? (Y/N)		N	N	N	N	N		
<u>Average Vs and Site Class by Each Method:</u>								
Vs Input for Grey Clay:	**Avg. Vs**	393.4	373.8	389.6	433.0	408.0		
	Class	**C**	**C**	**C**	**C**	**C**		
Vs Eqn. for Grey Clay:	**Avg. Vs**	398.0	378.7	393.8	436.9	411.3		
	Class	**C**	**C**	**C**	**C**	**C**		

Note:
1. An average Vs of 128 m/s is typical of highly plastic silty clays of Eastern Ontario.
2. Equation Vs = 125 + 1.1667*Z for sensitive silty clay from Hunter, Burns, et. al. - Figure 16
   (this is a conservative interpretation without the high Vs Lemieux BH)

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5951-1 - TEST HOLE LOCATION PLAN

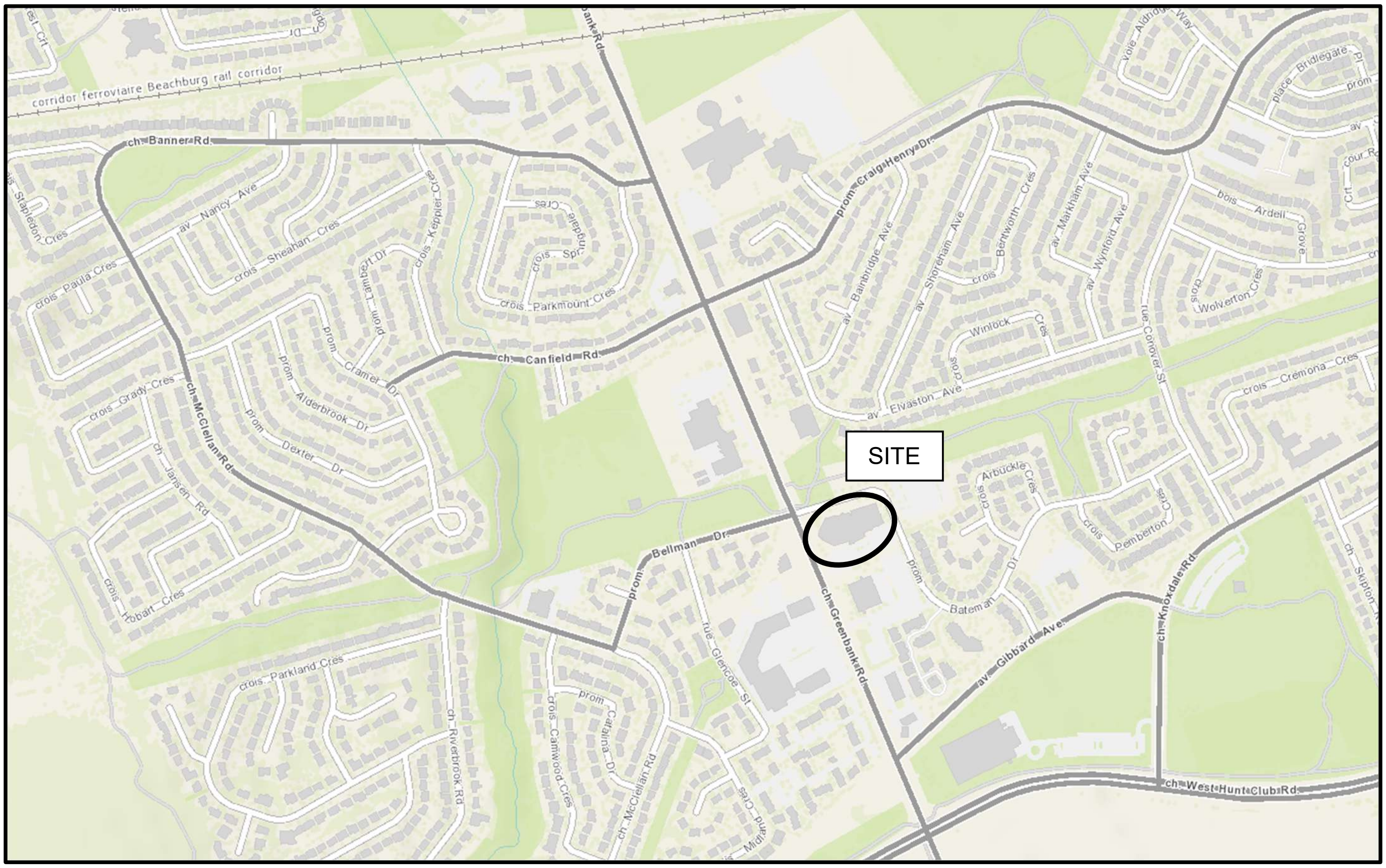
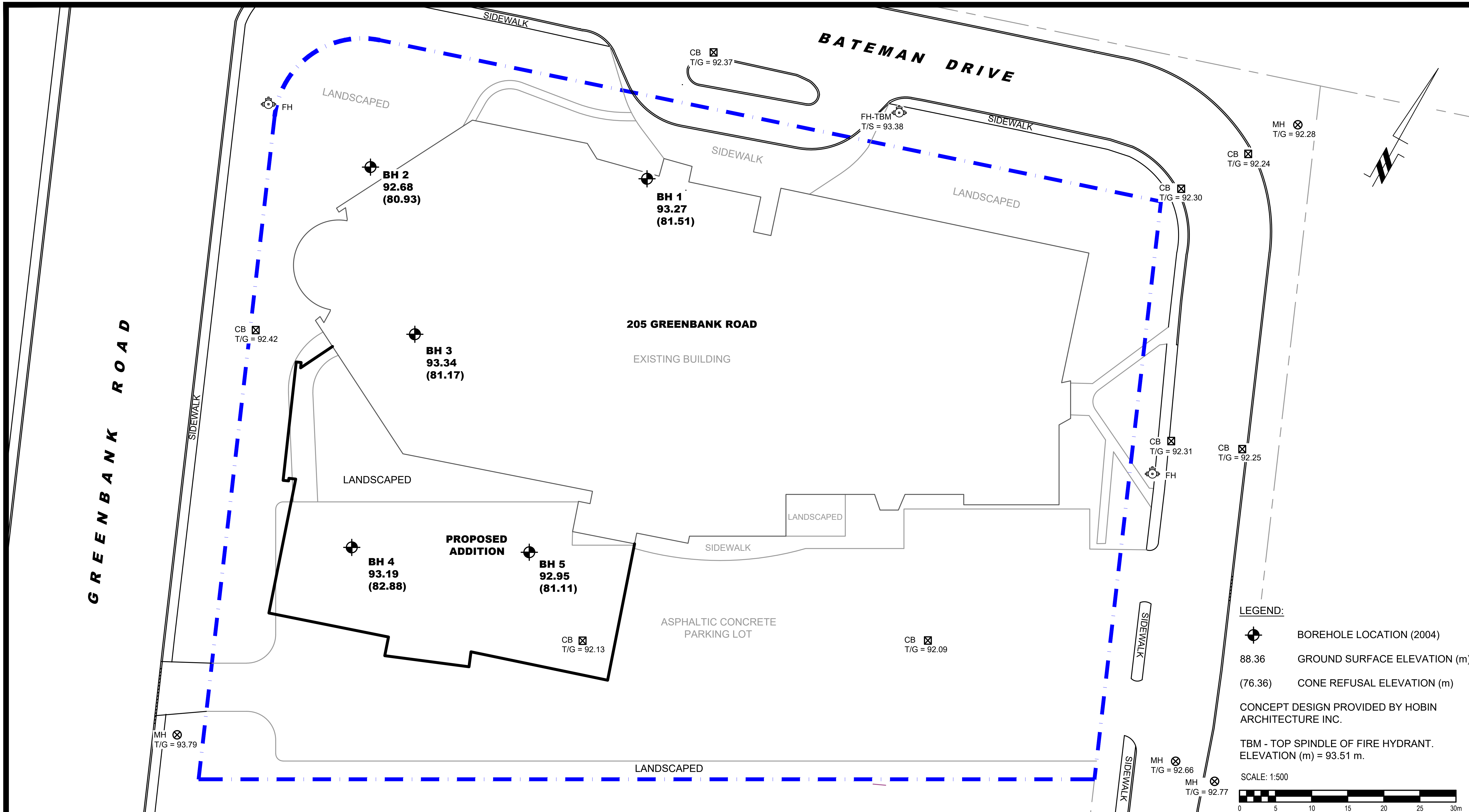


FIGURE 1

KEY PLAN



patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

WOODVALE PENTECOSTAL CHURCH
GEOTECHNICAL INVESTIGATION
PROPOSED ADDITION TO EXISTING CHURCH
205 GREENBANK ROAD
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 09/2021
Drawn by: JM	Report No.: PG5951-1
Checked by: KP	Dwg. No.: PG5951-1
Approved by: SD	Revision No.:

p:\autocad drawings\geotechnical\pg59xx\pg5951\pg5951-1 test hole location plan.dwg

11x17