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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

Geotechnical Investigation

Proposed Residential Development
98 & 100 Bearbrook Road
Ottawa, Ontario

Prepared For

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November 5, 2021

Report: PG5883-1 Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Landric Homes to conduct a geotechnical investigation for the proposed residential development site to be located at 98 & 100 Bearbrook Road in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of boreholes.

- Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings and correspondence with the client, it is understood that the proposed development will consist of a multi-storey residential building with two underground levels. Associated access lanes, parking areas, walkways, and landscaped areas are also anticipated as part of the development. It is expected that the proposed buildings will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on July 9, 2021 and consisted of advancing a total of 4 boreholes to a maximum depth of 5.9 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5883-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a low-clearance, rubber track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of advancing each test hole to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

The soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) 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 was carried out at regular depth intervals in cohesive soils using a vane apparatus.

The thickness of the overburden was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at borehole BH 1-21. 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

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

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5883-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of 1 shrinkage test, 1 grain size distribution analyses, and 2 Atterberg limits tests were completed on selected soil samples. The results are presented in Subsection 4.2 and on Grain Size Distribution and Hydrometer Testing, and Atterberg Limit's Results and Shrinkage Test Results sheets presented in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures, one of which was collected from BH4-21. 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 Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The ground surface across the subject site is relatively flat and at grade with the surrounding roadways. The subject site consists of two residential lots currently occupied by single-family residential dwellings with associated garages, landscaped areas, fences, and driveways. The site is occupied by a significant number of mature trees.

The site is bordered by residential dwellings to the north, Bearbrook Road to the east, an elementary school to the and west, and by a commercial plaza to the south. The existing ground surface across the site is relatively level with an approximate geodetic elevation of 74.5 m.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of topsoil underlain by fill extending to depths ranging from 0.7 to 1.4 m. The fill was generally observed to consist of brown silty sand with trace to some clay and some topsoil.

A very stiff to stiff, brown silty clay layer was encountered underlying the fill. The silty clay was observed to transition into a stiff to firm grey silty clay between 2.8 to 3.5 m depth and extended to the end of the boreholes.

A DCPT test was extended to a depth of 29.4 m into BH1-21. Practical refusal was not encountered.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic shale of the Billings formation, with an overburden drift thickness of 25 to 50 m depth.

Atterberg Limit and Shrinkage Tests

Atterberg limits testing, as well as associated moisture content testing, was completed on the recovered silty clay samples at selected locations throughout the subject site. The results of the Atterberg limits are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results					
Sample	**Depth (m)**	**LL (%)**	**PL (%)**	**PI (%)**	**Classification**
BH2-21 SS4	2.3 – 2.9	60	26	34	CH
BH3-21 SS3	1.5 – 2.1	62	26	36	CH
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; CH: Inorganic Clay of High Plasticity MH: Inorganic Silt of High Plasticity					

The results of the moisture contest test are presented in Table 2 and on the Soil Profile and Test Data Sheet in Appendix 1.

The results of the shrinkage limit test indicate a shrinkage limit of 21.0% and a shrinkage ratio of 1.69.

Table 2 – Moisture Content Results			
Borehole	**Sample**	**Depth (m)**	**Water Content (%)**
BH1-21	SS3	1.5 – 2.1	35.6
BH1-21	SS4	2.3 – 2.9	52.7
BH1-21	SS5	3.1 – 3.7	53.5
BH1-21	SS6	3.9 - 4.5	55.6
BH2-21	SS3	1.5 – 2.1	34.0
BH2-21	SS4	2.3 – 2.9	55.8
BH3-21	SS5	3.1 – 3.7	53.1
BH3-21	SS2	0.7 – 1.3	37.5
BH3-21	SS3	1.5 – 2.1	52.4
BH3-21	SS4	3.1 – 3.7	52.5
BH4-21	SS3	1.5 – 2.1	37.0
BH4-21	SS4	2.3 – 2.9	51.3
BH4-21	SS5	3.1 – 3.7	54.2
BH4-21	SS6	4.6 – 5.2	51.3

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was also completed on one (1) selected soil sample. The results of the grain size analysis are summarized in Table 3 and presented on the Grain-size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 3 - Summary of Grain Size Distribution Analysis

Test Hole	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH4-21	SS4	0.0	0.2	30.3	69.5

4.3 Groundwater

Groundwater levels were measured on September 1, 2021 within the installed polytube piezometers. The measured groundwater levels are presented in Table 4 below.

Table 4 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21	74.65	2.48	72.17	September 1, 2021
BH 2-21	74.64	3.04	71.60	
BH 3-21	74.42	1.58	72.84	
BH 4-21	74.58	2.01	72.57	

Note: The ground surface elevation at each borehole location was surveyed using a high-precision GPS and referenced to a geodetic datum.

It should be noted that long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 2.5 to 3.0 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

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 suitable for the proposed development. Conventional footing foundations can be considered if the bearing resistance values are compliant with the anticipated building loads. Where design loads exceed the given bearing resistance values, consideration may be given to a raft foundation. It is expected the foundation will be placed on a very stiff to stiff, brown silty clay or a stiff to firm, grey silty clay.

Due to the presence of a silty clay deposit, a permissible grade raise restriction is required for the subject site for all footings founded on a silty clay bearing surface. The above and other considerations are discussed in the following paragraphs.

Given the proximity of the underground parking levels to the property lines, it is expected that a temporary shoring system may be required to support the excavation sides, such as a soldier pile and lagging system. This is discussed further in Section 6.3.

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. It is anticipated that existing fill within the proposed building footprint, free of deleterious material and significant amounts of organics, and approved by the geotechnical consultant at the time of construction can be left in place outside of the proposed building footprint and within landscaped areas. However, it is recommended that the existing fill layer be proof-rolled by a vibratory roller making several passes under dry and above freezing conditions and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

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

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery.
The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 99% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If excavated brown very stiff to stiff brown silty clay, free of organics and deleterious materials, is to be used to build up the subgrade level for areas to be paved, the silty clay, under dry conditions, should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD using a sheepsfoot roller. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000, connected to a perimeter drainage system is provided.

Protection of Subgrade

Since the subgrade material will consist of a silty clay deposit, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay to potential disturbance.

5.3 Foundation Design

Spread Footing Foundation

Conventional style pad footings, up to 3 m wide, and strip footings, up to 6 m wide, founded on an undisturbed, very stiff to stiff, brown silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa** incorporating a geotechnical factor of 0.5.

Conventional style pad footings, up to 3 m wide, and strip footings, up to 6 m wide, founded on an undisturbed, stiff to firm, grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **125 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **kPa** incorporating a geotechnical factor of 0.5.

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

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided herein will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Raft Foundation

Consideration can be provided to a raft foundation if the building loads are acceptable. If this option is considered, additional soils information including unidimensional consolidation testing is required. Based on the available data, the following parameters should be considered for raft design.

For design purposes, the raft foundation base is assumed to be located at 6-7 m depth to accommodate underground parking. The bearing medium will consist of a sensitive grey silty clay which is susceptible to disturbance under construction traffic. The bearing surface should be protected to prevent disturbance.

The factored bearing resistance (contact pressure) at ULS can be designed for **350 kPa.** A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **240 kPa** will be considered acceptable. The loading conditions for the contact pressure are based on sustained loads, generally considered to be 100% Dead Load and 50% Live Load.

The modulus of subgrade reaction was calculated to be 6 MPa/m for a contact pressure of **240 kPa**. The raft foundation design considers the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

A raft foundation bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided herein will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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 silty clay and engineered fill bearing media when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

Permissible Grade Raise

A permissible grade raise restriction of **2 m** is recommended for the subject site. If greater permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for foundations constructed at the subject site. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the native silty clay will be considered an acceptable subgrade upon which to commence backfilling for basement slab construction. It is anticipated that the basement area for the proposed building will be mostly parking and the recommended pavement structures noted in Subsection 5.8 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

All backfill material within the footprint of the proposed buildings (but outside the zones of influence of the footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD. Within the zones of influence of the footings, the backfill material should be compacted to a minimum of 98% of its SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material. A clear crushed stone fill is recommended for backfilling below the floor slab for limited span slab-on-grade areas, such as front porch or garage footprints. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone below basement floor slabs.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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

Lateral Earth Pressure

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

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

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

Actual earth pressures could be higher than the “at-rest” case if care is not exercisedduring the compaction of the backfill materials to stay at least 0.3 m away from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a \cdot \gamma \cdot H^2/g$ where:

$a_c = (1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)

H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earthforce component (P_O) under seismic conditions can be calculated using:

$P = 0.5\ K{\cdot}\gamma{\cdot}H^2$, where K = 0.5 for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of thewall, where:

$h = \{P_O{\cdot}(H/3)+\Delta P_{AE}{\cdot}(0.6{\cdot}H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads shouldbe factored as live loads, as per OBC 2012.

5.7 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 5. The flexible pavement structure presented in Tables 6 and 7 should be used for surface parking and at grade access lanes and heavy loading parking areas.

Table 5 - Recommended Rigid Pavement Structure - Lower Parking Level	
Thickness (mm)	**Material Description**
150	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

Table 6 – Recommended Pavement Structure – Car Only Parking Areas	
Thickness (mm)	**Material Description**
50	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or concrete fill.	

Table 7 – Recommended Pavement Structure – Access Lanes and Heavy Truck Parking Areas	
Thickness (mm)	**Material Description**
40	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course –** HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or concrete fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material.

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.

Due to the impervious nature of the silty clay deposit, where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. The subdrain inverts should be approximately 300 mm below subgrade level and run longitudinal along the curb lines. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

For the proposed underground parking levels, it is understood that the building foundation walls may be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation walls be blind poured against a drainage system and waterproofing system fastened to the shoring system.

Waterproofing of the foundation walls is recommended and the membrane is to be installed from 2 m below finished grade, extending down the foundation walls to the founding elevation (underside of raft or footing).

It is also recommended that a composite drainage system, such as Delta Drain 6000 or equivalent, be installed between the waterproofing membrane and the foundation wall and extend from the finished grade to the founding elevation. The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit.

It is recommended that 150 mm diameter sleeves at 5 m centres be cast in the foundation wall at the perimeter footing or raft slab interface to allow the infiltration of water to flow to an interior perimeter underfloor drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area. These recommendations are summarized in Figure 3 – Water Suppression System, which is provided in Appendix 2.

A waterproofing system should also be provided for any elevator pits (pit bottom and walls).

Raft Slab Construction Joints

It is expected that the raft slab will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the raft slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Sub-slab Drainage

Sub-slab drainage will be required to control water which infiltrates through the raft foundation, or to control water underlying the basement slab in areas where footings are utilized. For design purposes, we recommend that 150 mm diameter perforated pipes be placed along the interior perimeter of the foundation walls and within the building at approximate 6 m spacing. The spacing of the sub-slab drainage system should be confirmed at the time of backfilling the floor, following completion of the excavation when water infiltration can be better assessed.

Foundation Backfill

Where sufficient space is available, 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, should be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. A minimum of 2.1 m thick soil cover (or equivalent) should be provided for all exterior unheated footings.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations). Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

Unsupported Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Temporary shoring is anticipated to be required to support the overburden soils during the proposed building excavation. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system is recommended to consist of a soldier pile and lagging system which could be cantilevered, anchored or braced.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 5 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 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 compacted to 99% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

The backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. The seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material.
The seals should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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.

Permit to Take Water

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

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

Impacts on Neighbouring Properties

As the proposed multi-storey building will be founded below the long term groundwater level, a groundwater infiltration control system has been recommended to mitigate the effects of groundwater infiltration. Any long term dewatering of the site will be minimal and should have no adverse effects to the surrounding buildings or structures. The short term dewatering during the excavation program will be managed by the excavation contractor, as discussed above.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

The proposed development is located in an area of low to medium sensitive silty clay deposits for tree planting. Based on our review of the subsurface profile below the subject site, the underlying silty clay deposit is relatively dry and designated as a very stiff to firm silty clay. Therefore, the proposed development is considered to be located within an area of low sensitive silty clay deposits for tree planting.

Tree Planting Restrictions

Based on the results of the representative soil samples, the subject site is considered as a **low**/**medium** sensitivity area for tree planting according to the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines)

Since the modified plasticity limit (PI) generally does not exceed 40%, large trees (mature height over 14 m) can be planted at the subject site 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).

Based on our testing results, tree planting setback limits should be 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the following conditions 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 of 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 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).

7.0 Recommendations

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

- Grading plan review from a geotechnical perspective, once the final grading plan is available.
- Review of the contractor's design of the temporary shoring system.
- Observation of all bearing surfaces prior to the placement of concrete.
- Inspection of the foundation waterproofing and all foundation drainage systems
- 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 Landric Homes 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
November 5-2021
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Owen Canton, E.I.T. Joey R. Villeneuve, M.A.Sc., P.Eng

Report Distribution:

- ❑ Landric Homes
- ❑ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN-SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMIT TESTING RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 87 & 100 Bearbrook Rd.
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5883**

REMARKS

HOLE NO. **BH 1-21**

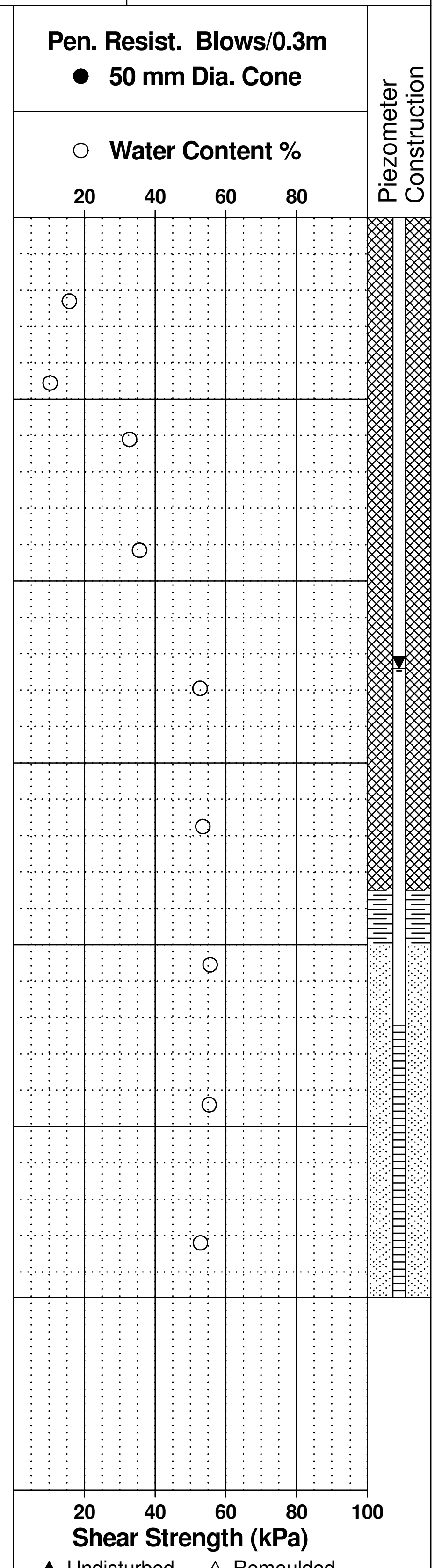
BORINGS BY CME-55 Low Clearance Drill

DATE July 9, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	74.65
TOPSOIL 0.15							
FILL: Brown silty sand, trace clay		AU	1				
1.07		SS	2	75	13	1	73.65
Very stiff to stiff, brown **SILTY CLAY**		SS	3	75	16	2	72.65
		SS	4	75	5		
- stiff to firm and grey by 3.0m depth						3	71.65
		SS	5	83	5		
		SS	6	83	P	4	70.65
		SS	7	83	P	5	69.65
		SS	8	83	3		
5.94							

Dynamic Cone Penetration Test commenced at 5.94m depth. Cone pushed to 29.34m depth, no DCPT refusal encountered, borehole terminated at 29.34m depth.

(GWL @ 2.48m - Sept. 1, 2021)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 87 & 100 Bearbrook Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

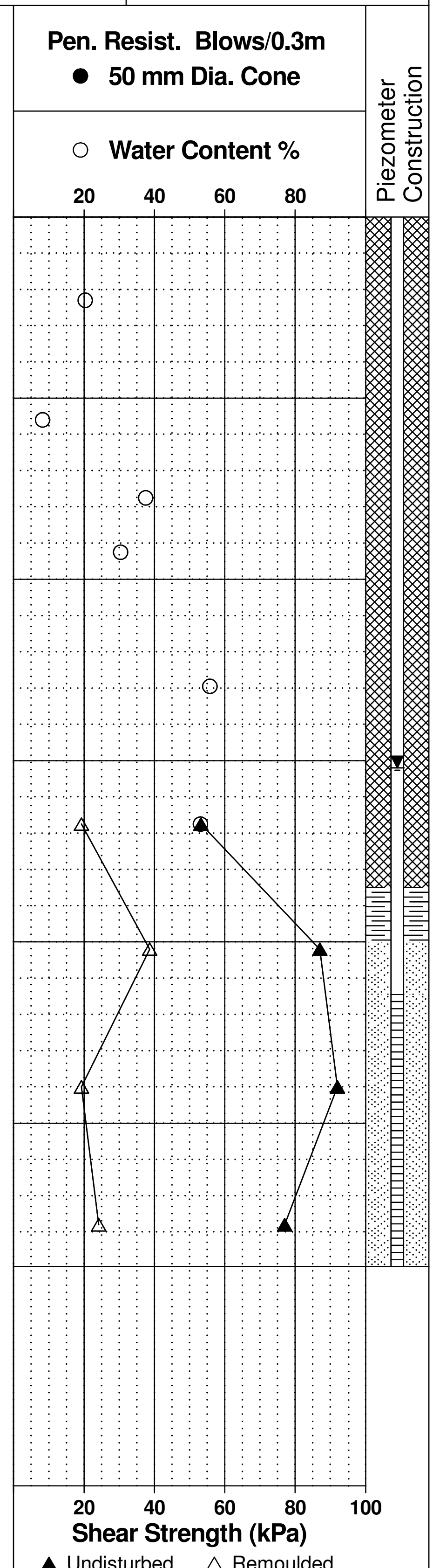
DATE July 9, 2021

FILE NO. PG5883

HOLE NO. BH 2-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.15				
FILL: Brown silty sand with some to trace clay	1.37	AU	1		
		SS	2	75	15
Very stiff to stiff, brown **SILTY CLAY** - grey by 3.0m depth	5.79	SS	3	75	16
		SS	4	92	3
		SS	5	92	P
End of Borehole (GWL @ 3.04m - Sept. 1, 2021)					

DEPTH (m)	ELEV. (m)
0	74.64
1	73.64
2	72.64
3	71.64
4	70.64
5	69.64



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 87 & 100 Bearbrook Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

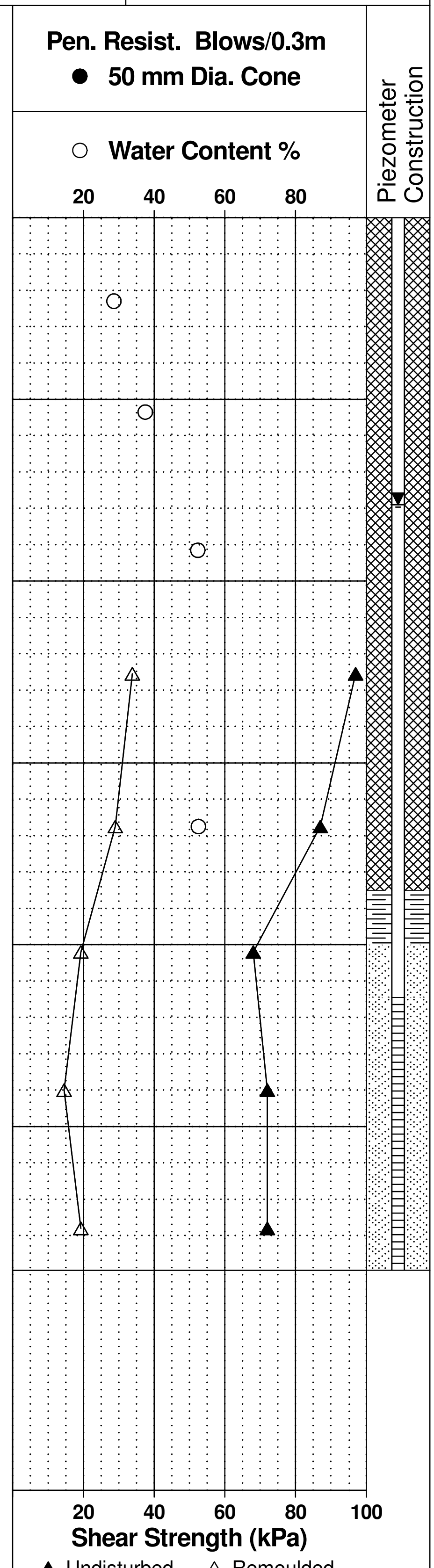
BORINGS BY CME-55 Low Clearance Drill

DATE July 9, 2021

FILE NO. PG5883

HOLE NO. BH 3-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	74.42
TOPSOIL 0.15							
FILL: Brown silty sand, trace clay		AU	1				
0.71							
		SS	2	83	15	1	73.42
		SS	3	83	7		
Very stiff to stiff, brown **SILTY CLAY**						2	72.42
- grey by 2.8m depth						3	71.42
		SS	4	83	P		
						4	70.42
						5	69.42
5.79							
End of Borehole							
(GWL @ 1.58m - Sept. 1, 2021)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 87 & 100 Bearbrook Rd.
Ottawa, Ontario

DATUM Geodetic

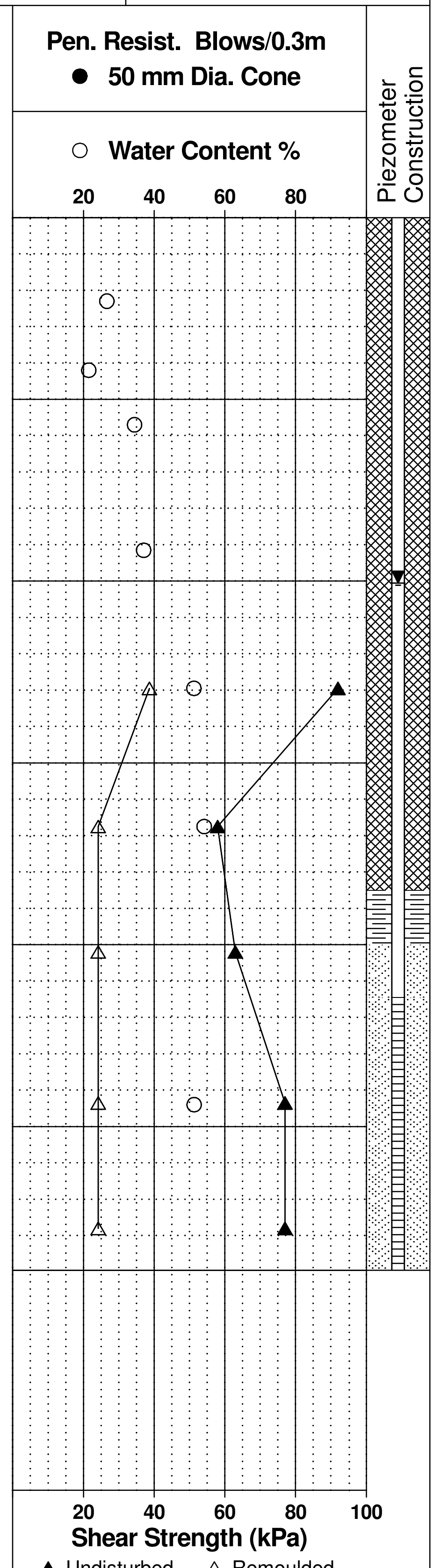
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** July 9, 2021

FILE NO. PG5883

HOLE NO. BH 4-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	74.58
TOPSOIL 0.25	AU	1				
FILL: Brown silty sand with clay, some topsoil 0.91						
	SS	2	75	8	1	73.58
	SS	3	83	9		
					2	72.58
Very stiff to stiff, brown **SILTY CLAY**	SS	4	83	P		
					3	71.58
	SS	5	33	P		
- grey by 3.5m depth						
					4	70.58
	SS	6	83	P		
5.79					5	69.58
End of Borehole						
(GWL @ 2.01m - Sept. 1, 2021)						



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

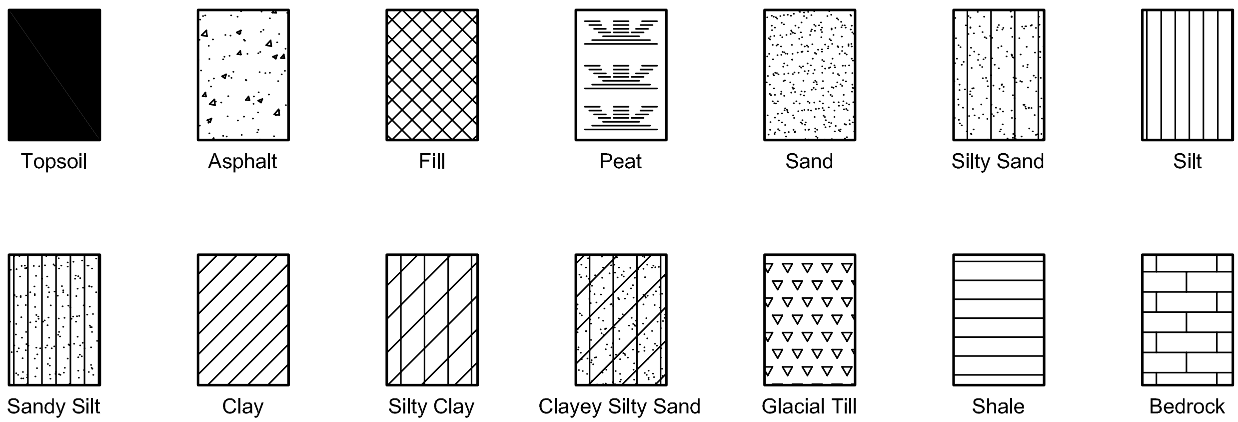
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

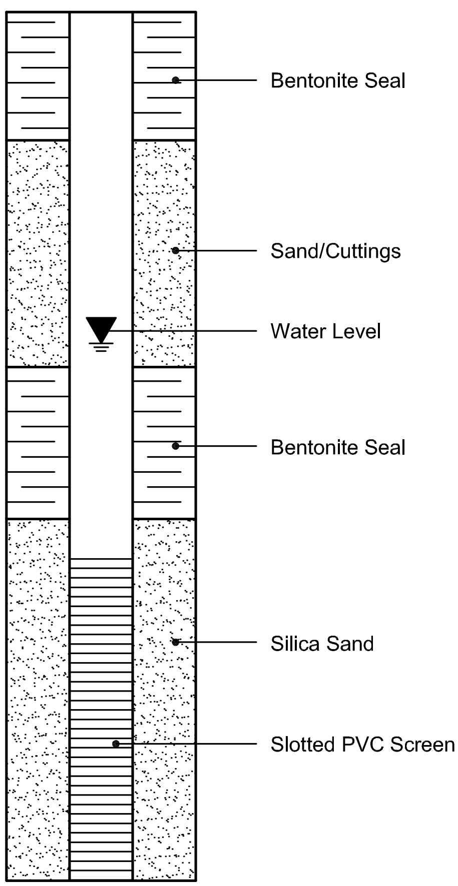
SYMBOLS AND TERMS (continued)

STRATA PLOT

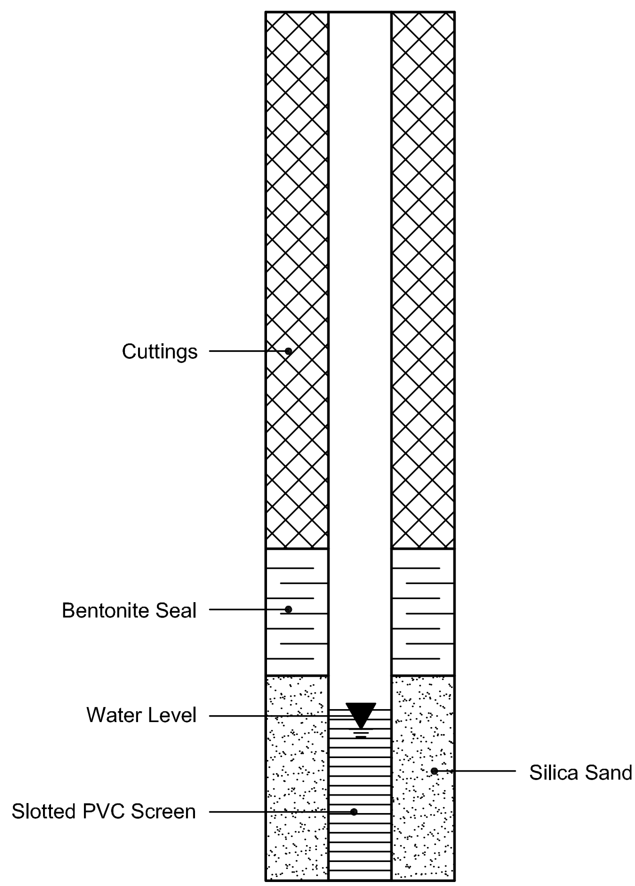


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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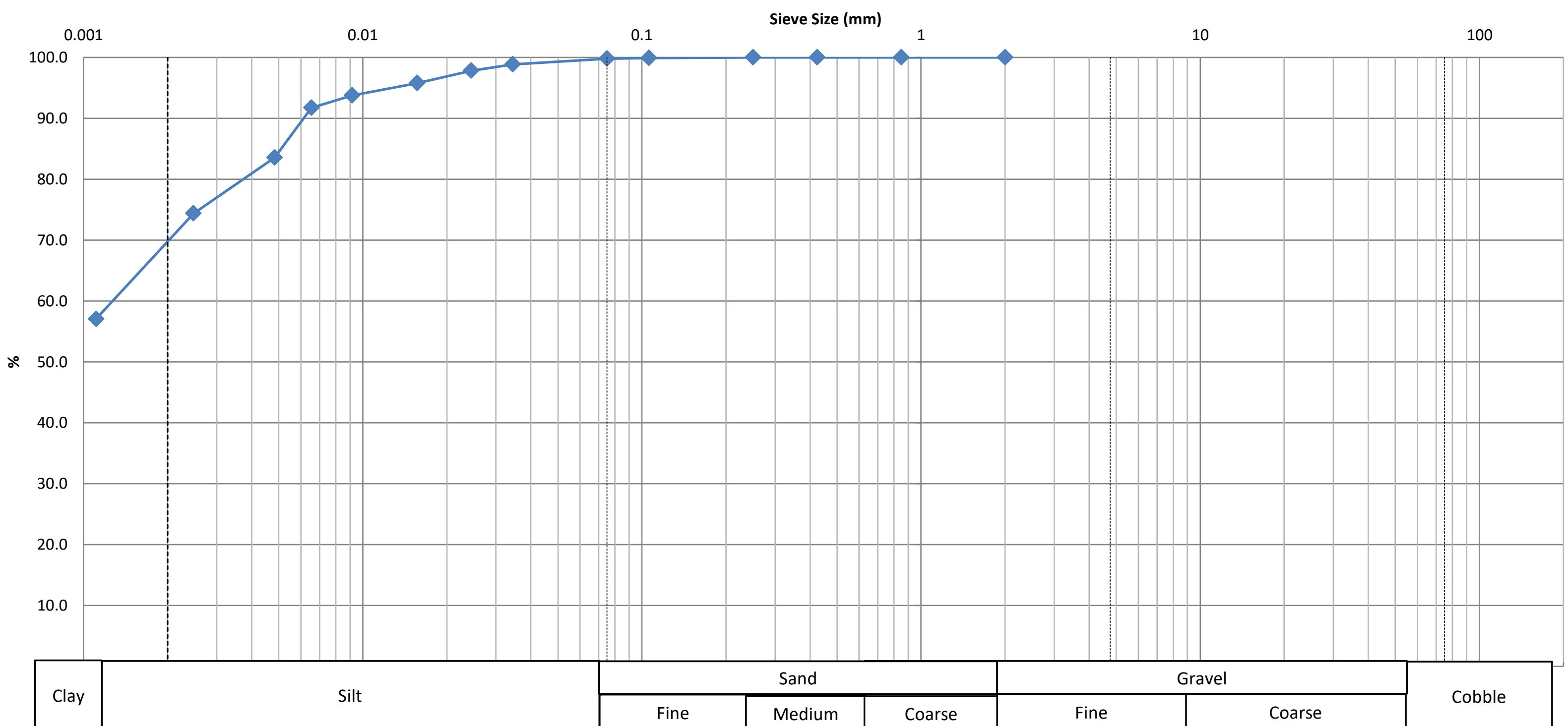
SIEVE ANALYSIS
ASTM C136

CLIENT:	Landric Homes	DEPTH:	7' 6" - 9' 6"	FILE NO:	PG5883
CONTRACT NO.:		BH OR TP No.:	BH4-21 SS4	LAB NO:	26207
PROJECT:	Bearbrook Homes			DATE RECEIVED:	12-Jul-21
				DATE TESTED:	14-Jul-21
DATE SAMPLED:	9-Jul-21			DATE REPORTED:	19-Jul-21
SAMPLED BY:	I. Khan			TESTED BY:	DB



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					21.4					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)	Clay (%)	
					0.0	0.2		30.3	69.5	

Comments:

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

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HYDROMETER
LS-702 ASTM-422

CLIENT:	Landric Homes	DEPTH:	7' 6" - 9' 6"	FILE NO.:	PG5883
PROJECT:	Bearbrook Homes	BH OR TP No.:	BH4-21 SS4	DATE SAMPLED	9-Jul-21
LAB No. :	26207	TESTED BY:	DB	DATE RECEIVE	12-Jul-21
SAMPLED BY:	I. Khan	DATE REPT'D:	19-Jul-21	DATE TESTED:	14-Jul-21

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
104.7	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	44.65	TARE WEIGHT	50.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	0.15	AIR DRY	150.00	100.00
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	139.30	89.30
		CORRECTED	0.893	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75			
2.0	**0.0**	0.0	**100.0**
Pan	**104.7**		
0.850	**0.00**	0.0	**100.0**
0.425	**0.00**	0.0	**100.0**
0.250	**0.00**	0.0	**100.0**
0.106	**0.04**	0.1	**99.9**
0.075	**0.11**	0.2	**99.8**
Pan	**0.15**		
SIEVE CHECK 0.0	MAX = 0.3%		

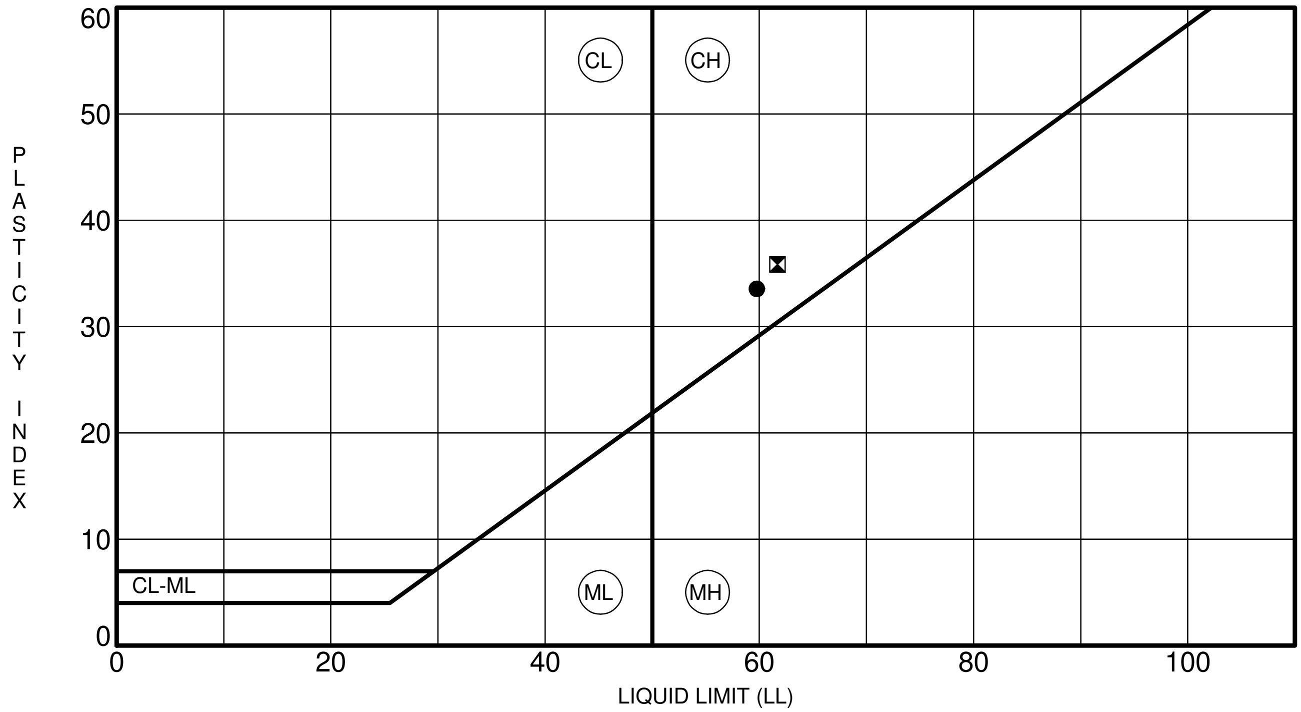
HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	9:20	54.5	6.0	23.0	0.0344	98.8	**98.8**
2	9:21	54.0	6.0	23.0	0.0245	97.8	**97.8**
5	9:24	53.0	6.0	23.0	0.0157	95.8	**95.8**
15	9:34	52.0	6.0	23.0	0.0092	93.8	**93.8**
30	9:49	51.0	6.0	23.0	0.0065	91.7	**91.7**
60	10:19	47.0	6.0	23.0	0.0048	83.6	**83.6**
250	1:29	42.5	6.0	23.0	0.0025	74.4	**74.4**
1440	9:19	34.0	6.0	23.0	0.0011	57.1	**57.1**

COMMENTS:

Moisture = 21.4%

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



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 2-21**	**SS 4**	**60**	**26**	**34**		**CH - Inorganic clays of high plasticity**
⧗	**BH 3-21**	**SS 3**	**62**	**26**	**36**		**CH - Inorganic clays of high plasticity**

CLIENT **Landric Homes**

FILE NO. **PG5883**

PROJECT **Geotechnical Investigation - Prop. Residential Development - 87 & 100 Bearbrook Rd.**

DATE **9 Jul 21**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 32462

Report Date: 15-Jul-2021

Order Date: 13-Jul-2021

Project Description: PG5883

	Client ID:	BH4-21 SS3	-	-	-
	Sample Date:	09-Jul-21 09:00	-	-	-
	Sample ID:	2129270-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	72.6	-	-	-
General Inorganics					
pH	0.05 pH Units	6.96	-	-	-
Resistivity	0.10 Ohm.m	149	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	14	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – WATER SUPPRESION SYSTEM

DRAWING PG5883-1 – TEST HOLE LOCATION PLAN

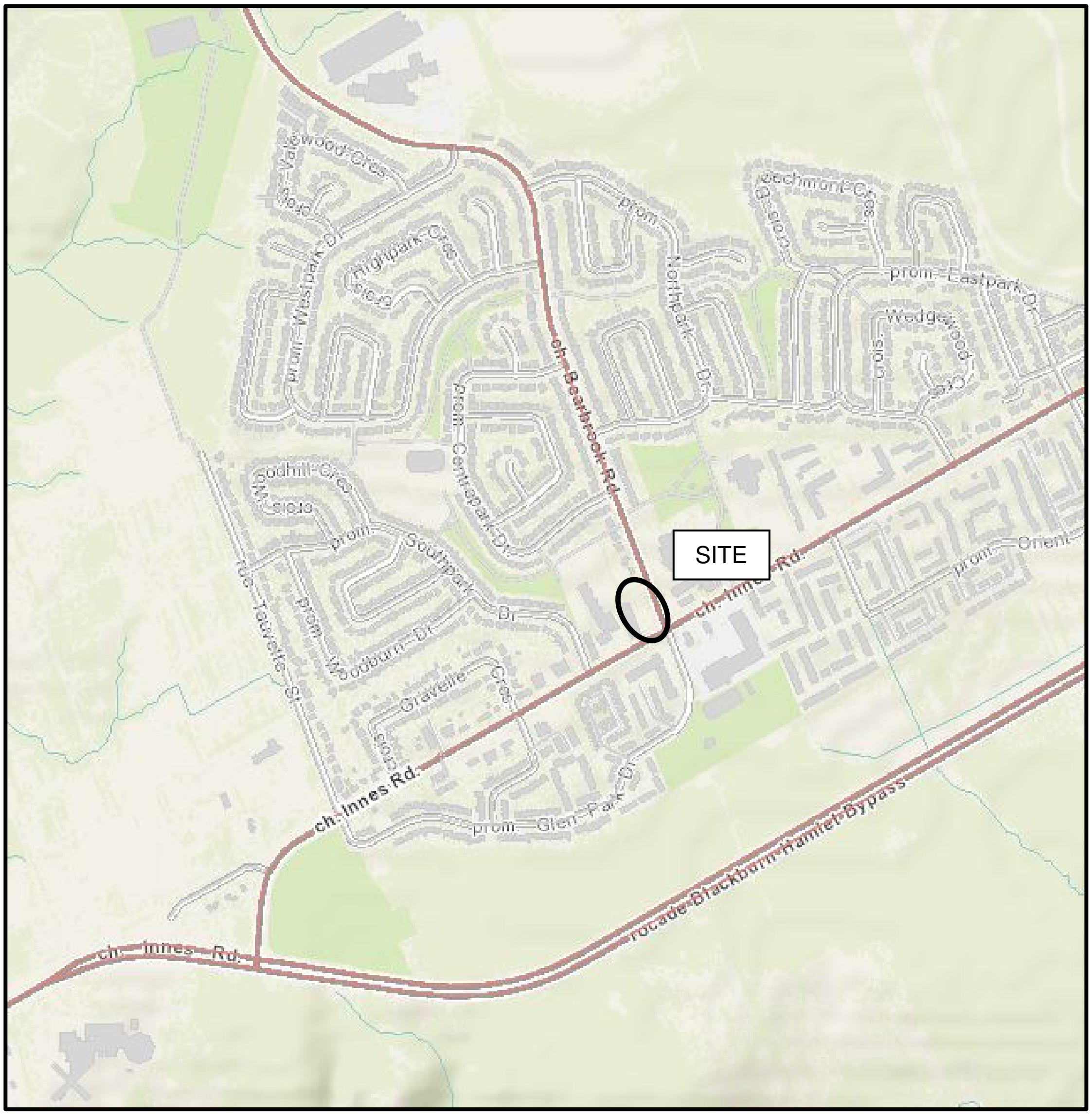


FIGURE 1

KEY PLAN

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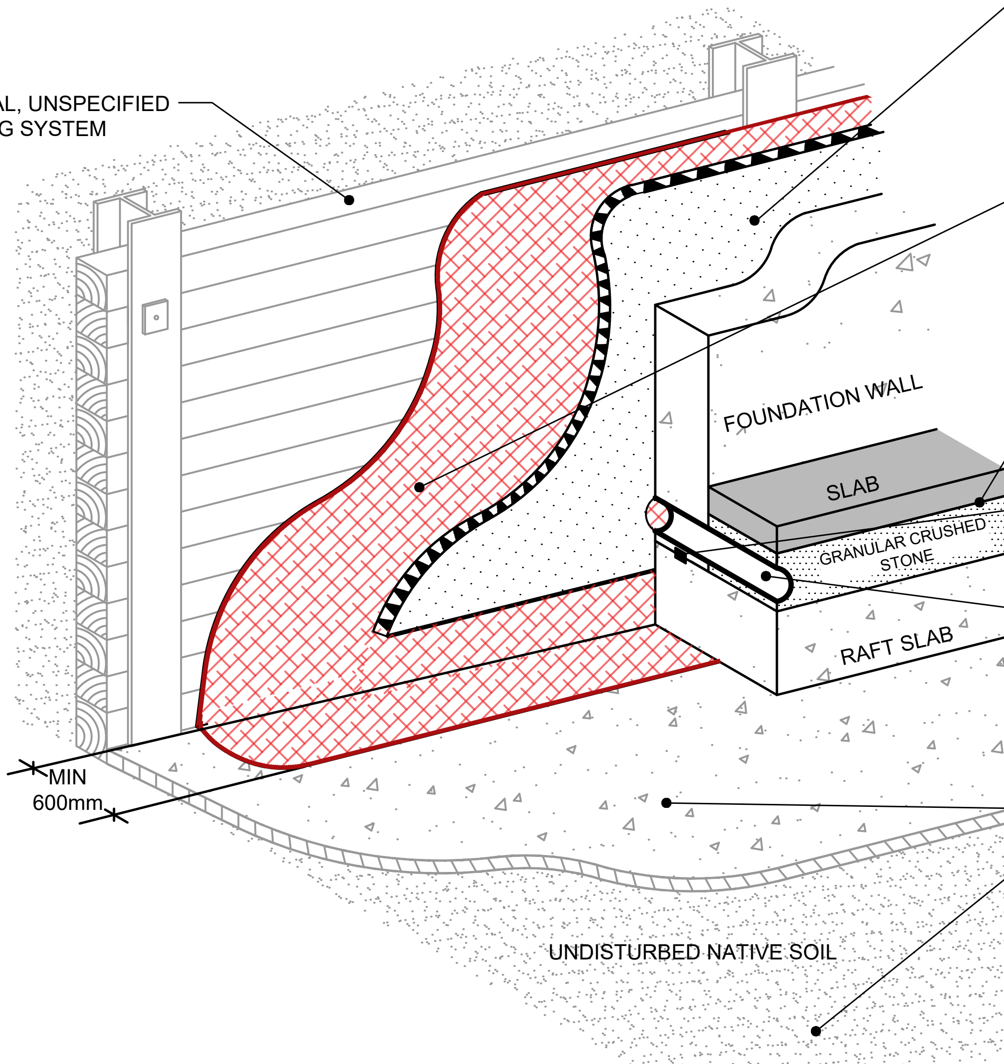
GENERAL, UNSPECIFIED SHORING SYSTEM



COMPOSITE FOUNDATION DRAINAGE MEMBRANE 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH 3M TAPE OR EQUIVALENT.

150miL GRANULAR BENTONITE SURFACING LAMINATED TO 20miL THICK HDPE MEMBRANE. INSTALL IN HORIZONTAL LIFTS TO MANUFACTURERS SPECIFICATIONS IN SHINGLE FASHION WITH THE HDPE SIDE FACING APPLICATOR TO AN ADEQUATELY PREPARED SUBSTRATE SURFACE. WATER PROOFING MEMBRANE TO BE INSTALLED FROM 2m BELOW FINISHED GRADE, EXTENDING DOWN THE FOUNDATION WALLS TO THE FOUNDING ELEVATION (UNDERSIDE OF RAFT SLAB)

OPSS GRANULAR "A" COMPACTED TO 100% OF THE MATERIALS STANDARD PROCTOR MAXIMUM DRY DENSITY (SPMDD) PLACED IN MAXIMUM 300mm THICK LIFTS

BENTONITE WATERSTOP SUCH AS WATERSTOP RX OR EQUIVALENT

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE GRANULAR A LAYER AT APPROXIMATELY 5m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE. THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER INTERIOR SUBFLOOR DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT.

MINIMUM 75mm THICK, 15 MPa CONCRETE MUD SLAB

UNDISTURBED STIFF SILTY CLAY AND/OR GLACIAL TILL APPROVED BY THE GEAOTECHNICAL CONSULTANT

IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE AREA.

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LANDRIC HOMES
PROPOSED HI-RISE BUILDING
98 & 100 BEARBROOK ROAD
OTTAWA, ONTARIO

Title:
WATER SUPPRESSION SYSTEM

Scale: NTS	Date: 10/2021
Drawn by: NFRV	Report No.: PG5883-1
Checked by: OC	Drawing No.: FIG. 2
Approved by: JV	Revision No.:

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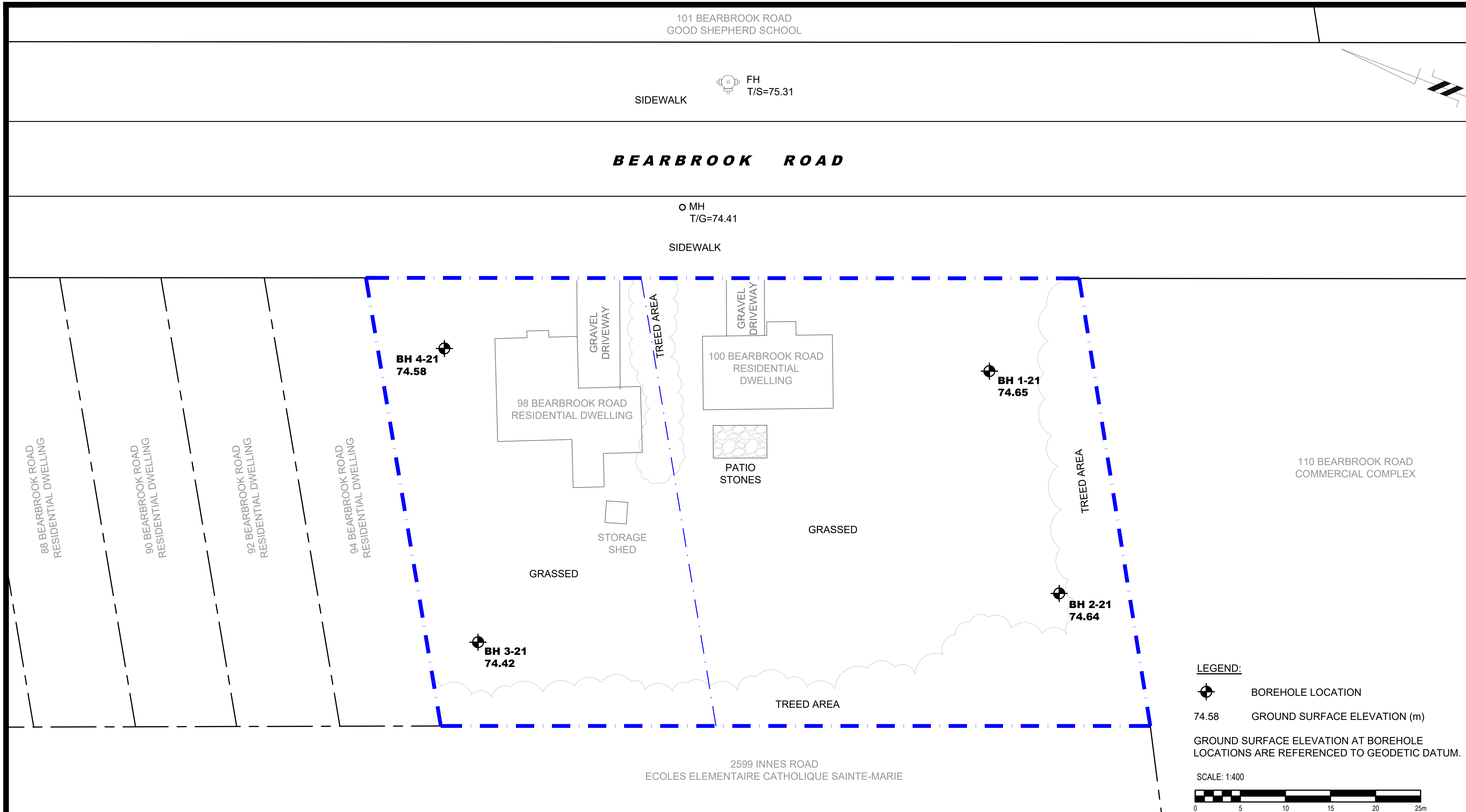

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NO.	REVISIONS	DATE	INITIAL

LANDRIC HOMES

GEOTECHNICAL INVESTIGATION
98 AND 100 BEARBROOK ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 07/2021
Drawn by: YA	Report No.: PG5883-1
Checked by: OC	Dwg. No.: PG5883-1
Approved by: SD	Revision No.:

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