Geotechnical
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Geotechnical Investigation

Proposed Residential Development
2275 Mer Bleue Road
Ottawa, Ontario

Prepared For

Caivan Communities

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Report: PG5521-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Caivan Communities to conduct a geotechnical investigation for the proposed residential development to be located at 2275 Mer Bleue Road in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2). The objective of the investigation was to:

❑ determine the subsurface soil and groundwater conditions by means of boreholes and monitoring well program.

❑ provide geotechnical recommendations for the foundation design of the proposed buildings and provide geotechnical construction precautions which may affect the design.

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

2.0 Proposed Development

It is understood that the proposed development will consist of low-rise residential wood-framed structures as well as a mixed-use density block consisting of low- to mid-rise structures. It is anticipated the proposed residential structures will be provided basements and attached garage structures. Local roadways, residential driveways and landscaped areas are also anticipated for the proposed development. It is further anticipated that the site will be serviced by future municipal services.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was carried out on December 11, 2020. At that time, 5 boreholes were advanced to a maximum depth of 5.9 m below the existing ground surface. The test hole locations were placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations for the current investigation are presented on Drawing PG5521-2 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track mounted drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler, or from the auger flights. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the samples were recovered from the split-spoon and auger samples recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented 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 15 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 3. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the 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.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations

Groundwater

All boreholes were fitted with a flexible polyethylene standpipe to allow groundwater level monitoring. The groundwater level reading were obtained after a suitable stabilization period subsequent to the completion of the field investigation. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All samples from the current investigation will be stored in the laboratory for a period of one 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 determined by Paterson personnel and surveyed in the field by Paterson. The test hole elevation are referred to the geodetic datum. The locations of the boreholes are presented on Drawing PG5521-2 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were visually examined in our laboratory to review the results of the field logging. A total of 1 shrinkage test, 2 grain size distribution analyses and 5 Atterberg limits tests were completed on selected soil samples. The results of our testing are presented in Subsection 4.2 and on Grain Size Distribution and Hydrometer Testing and Atterberg Limit's 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. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site currently consists of undeveloped agricultural land. The subject site is relatively flat and slightly below grade of surrounding roadways. Approximately 1 m deep ditches were noted along the west and north property boundaries. The subject site is bordered to the north by Brian Coburn Boulevard East, to the east a residential subdivision to the east, to the south by several one storey residential dwellings and to the west by Mer Bleue Road.

4.2 Subsurface Profile

Overburden

Generally, the subsurface soil profile encountered at the test hole locations consisted of a topsoil layer underlain by a deep deposit of silty clay. A hard to stiff brown silty clay crust was observed within the upper 2.7 to 3.0 m below the ground surface. The weathered silty clay crust was observed to be underlain by a firm to stiff layer of unweathered grey silty clay.

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

A DCPT was completed at BH 3 with practical refusal encountered at a depth of 23.9 m below the existing ground surface. Based on available geological mapping, the bedrock in the area is part of the Billings formation, which consists of shale with an overburden drift thickness ranging between 25 to 50 m.

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 tests 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 (%)	w (%)	Classification
BH 1	0.76 -1.37	67	29	38	34.4	CH
BH 2	0.76 -1.37	65	27	38	36.1	CH
BH 3	1.52 - 2.13	63	28	35	54.4	CH
BH 4	0.76 -1.37	71	32	39	41.7	CH
BH 5	0.76 -1.37	68	29	39	41	CH
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content; CH: Inorganic Clay of High Plasticity						

The results of the shrinkage limit test indicate a shrinkage limit of 23% and a shrinkage ratio of 1.74.

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was also completed on two (2) selected soil samples. The results of the grain size analysis are summarized in Table 2 and presented on the Grain-Size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 2 - Summary of Grain Size Distribution Analysis					
Test Hole	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 2	SS 2	0	0.6	25.9	73.5
BH 5	SS 2	0	0.4	29.8	69.8

4.3 Groundwater

Groundwater level readings were recorded on December 17, 2020. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1. It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Long-term groundwater level can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is expected between 2.5 to 3.5 m depth. 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 residential development. It is expected that the proposed buildings will be founded over conventional shallow footings placed mainly on an undisturbed very stiff to stiff silty clay crust layer.

Due to the presence of the sensitive silty clay deposit, the proposed development will be subjected to grade raise restrictions. Permissible grade raise recommendations have been designed for the subject site. The recommended permissible grade raise areas are presented in Drawing PG5521-3 - Permissible Grade Raise Plan in Appendix 2. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Although significant slopes were not noted at the time of our field investigation, the sloped ditches observed between the subject site and the western property boundaries are considered to be stable based on our observations. A global slope stability factor of safety of greater than 1.5 should be considered appropriate for the existing slopes within the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant amounts of organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb subgrade soils during site preparation activities.

Fill Placement

Fill used 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 or approved alternative. Granular material should be tested and approved prior to delivery to the site. The fill should be placed in loose lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness.

Fill placed beneath the building areas should be compacted to at least 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 and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of the SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

In-filling operations should be completed in a stepped fashion within the lateral support of the proposed buildings. The fill should consist of clean imported granular fill, such as OPSS Granular A or OPSS Granular B Type II material. The steps should have a minimum horizontal length of 1.5 m and minimum vertical height of 500 mm and should be compacted using suitable compaction equipment to a minimum 98% of the material's SPMDD.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, very stiff to stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **120 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **180 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed on an undisturbed, firm grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **70 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **100 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

The bearing resistance values at SLS for shallow footing bearing on the above-noted soils will be subjected to potential post-construction total and differential settlements of 25 and 15 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 a stiff silty clay above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Permissible Grade Raise Recommendations

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site and our knowledge of the silty clay deposit throughout the general area of the site, **the recommended permissible grade raise areas for buildings are defined in Drawing PG5521-3 - Permissible Grade Raise Areas** in Appendix 2.

Where proposed grade raises exceed our permissible grade raise recommendations, several options could be considered for the foundation support of the proposed buildings:

Scenario A

Where the grade raise is close to, but below, the maximum permissible grade raise, consideration should be given to using more reinforcement in the design of the foundation (footings and walls) to reduce the risks of cracking in the concrete foundation. The use of control joints within the brick work between the garage and basement area should also be considered.

Scenario B

Where the grade raise cannot be accommodated with soil fill, the following options could be used alone or in combination.

Option 1 - Use of Lightweight Fill

Lightweight fill (LWF) can be used, consisting of EPS (expanded polystyrene) Type 12 or 15 blocks or other light weight materials which allow for raising the grade without adding a significant load to the underlying soils.

However, these materials are expensive and, in the case of the EPS, are more difficult to use under the groundwater level, as they are buoyant, and must be protected against potential hydrocarbon spills. Use lightweight fill within the interior of the garage and porch areas to reduce the fill-related loads.

Option 2 - Preloading or Surcharging

It is possible to preload or surcharge the proposed site in localized areas provided sufficient time is available to achieve the desired settlements based on theoretical values from the settlement analysis. If this option is considered, a monitoring program using settlement plates will have to be implemented. This program will determine the amount of settlement in the preloaded or surcharged areas. Preloading to proposed finished grades will allow for consolidation of the underlying clays over a longer time period. Surcharging the site with additional fill above the proposed finished grade will add additional load to the underlying clays accelerating the consolidation process and allowing for accelerated settlements. Once the desired settlements are achieved, the site can be unloaded and the fill can be used elsewhere on site.

Once the required grade raises are established, the above options could be further discussed along with further recommendations on specific requirements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class E** for the foundations bearing on a is applicable for design of the proposed buildings bearing over a deep silty clay deposit throughout the subject site. 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 fill, containing deleterious or organic materials, the native soil or existing fill, approved by the geotechnical consultant at the time of excavation, will be considered to be an acceptable subgrade surface on which to commence backfilling for basement floor slab. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab for this purpose.

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 to placed 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, in our opinion, 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 dry unit weight of 20 kN/m^3.

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

Static Conditions

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

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

Seismic Conditions

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

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/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. The vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions could be calculated using

$P_o = 0.5\ K_o \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions presented above.

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

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

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

5.7 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of driveways, local residential streets and roadways with bus traffic. It should be noted that for residential driveways and car only parking areas, an Ontario Traffic Category A is applicable. For local roadways an Ontario Traffic Category B should be used for design purposes.

Table 4 - Recommended Pavement Structure - Driveways

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

Notes:
1-SUBGRADE - Either in situ soils or OPSS Granular B Type I or II material placed over in situ soil
2- Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this Pavement Structure.

Table 5 - Recommended Pavement Structure - Local Residential Roadways

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
400	**SUBBASE** - OPSS Granular B Type II

Notes:
1-SUBGRADE - Either in situ soils or OPSS Granular B Type I or II material placed over in situ soil
2- Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this Pavement Structure.

Table 6 - Recommended Pavement Structure - Arterial Roadways with Bus Traffic	
Thickness (mm)	**Material Description**
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** - Superpave 19.0 Asphaltic Concrete
50	**Lower Binder Course -** Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
600	**SUBBASE** - OPSS Granular B Type II

Notes:
1-SUBGRADE - Either in situ soils or OPSS Granular B Type I or II material placed over in situ soil
2- Minimum Performance Graded (PG) 64-34 asphalt cement should be used for this Pavement Structure.

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. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for driveways and local roadways and PG 64-34 asphalt cement should be used for roadways with bus traffic. 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 vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter foundation drainage system is recommended for the proposed structures. The system should consist of a 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structures. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a composite drainage system, such as Delta Drain 6000 or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection Against Frost Action

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

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

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavations to be undertaken by open-cut methods (i.e. unsupported excavations).

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.

Excavation Base Stability

The base of supported excavations can fail by three (3) general modes:

- ❑ Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,
- ❑ Piping from water seepage through granular soils, and
- ❑ Heave of layered soils due to water pressures confined by intervening low permeability soils.

Shear failure of excavation bases is typically rare in granular soils if adequate lateral support is provided. Inadequate dewatering can cause instability in excavations made through granular or layered soils. The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems.

The factor of safety with respect to base heave, FS_b, is:

$$FS_b = N_b s_u / \sigma_z$$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.
s_u - undrained shear strength of the soil below the base level
σ_z - total overburden and surcharge pressures at the bottom of the excavation

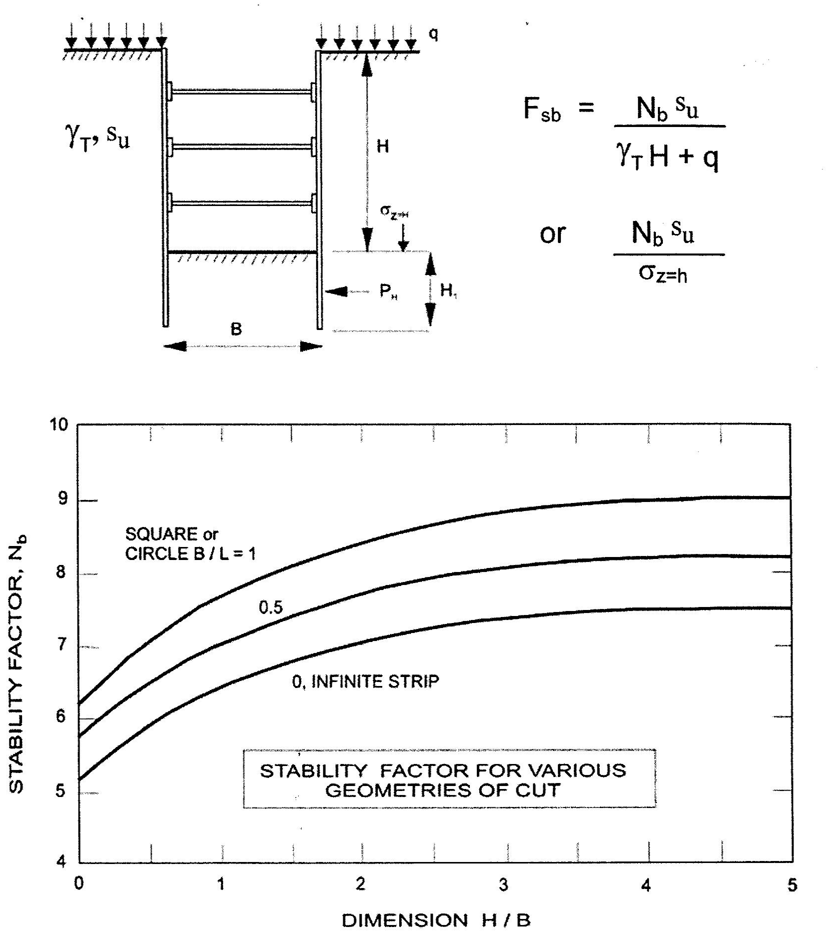


Figure 1 - Stability Factor for Various Geometries of Cut

In the case of soft to firm clays, a factor of safety of 2 is recommended for base stability.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & 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 layer should be increased to a minimum of 300 mm where the subgrade consists of a firm grey silty clay. 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 SPMDD.

Generally, it should be possible to re-use the 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.

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) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 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. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers 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

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low to moderate 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 of 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, 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 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.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Restrictions

Paterson completed a soils review of the site to determine applicable tree planting setbacks, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines) for trees planted within a public right-of-way (ROW). Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. Grain size distribution and hydrometer testing was also completed on selected soil samples. The above-noted test results were completed on samples taken at depths between the anticipated underside of footing elevation and a 3.5 m depth below finished grade. The results of our testing are presented in Tables 1 and 2 in Subsection 4.2 and in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples. In addition, based on the clay content found in the clay samples from the grain size distribution test results, moisture levels and consistency, the silty clay across the subject site is considered low to medium sensitivity clay and cannot be designated as sensitive marine clays.

The following tree planting setbacks are recommended for the low to medium sensitivity silty clay deposit throughout the subject site where silty clay was encountered (permissible grade raise restriction area) as shown in Drawing PG5521-4 - Tree Planting Setback Plan.

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

- The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.

- A small tree must be provided with a minimum 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 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 surround the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree).

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

Aboveground Swimming Pools, Hot Tubs, Decks and Additions

The in-situ soils are considered to be acceptable for in-ground swimming pools. Above ground swimming pools must be placed at least 5 m away from the residence foundation and neighboring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer`s requirements.

Additional grading around the hot tub should not exceed permissible grade raises. Otherwise, hot tub construction is considered routine, and can be constructed in accordance with the manufacturer’s specifications.

Additional grading around proposed deck or addition should not exceed permissible grade raises. Otherwise, standard construction practices are considered acceptable.

7.0 Recommendations

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

- ❑ Grading plan review from a geotechnical perspective, once the final grading plan is available.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to 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 request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with Paterson's present understanding of the project. Paterson requests permission to review the grading plan once available. Paterson's recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole log are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

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 to be notified immediately in order to permit reassessment of the recommendations.

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 Caivan Communities or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Dec.23-2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ❏ Caivan Communities (1 digital copy)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN-SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMIT TESTING RESULTS

ANALYTICAL TEST RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2275 Mer Bleue Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

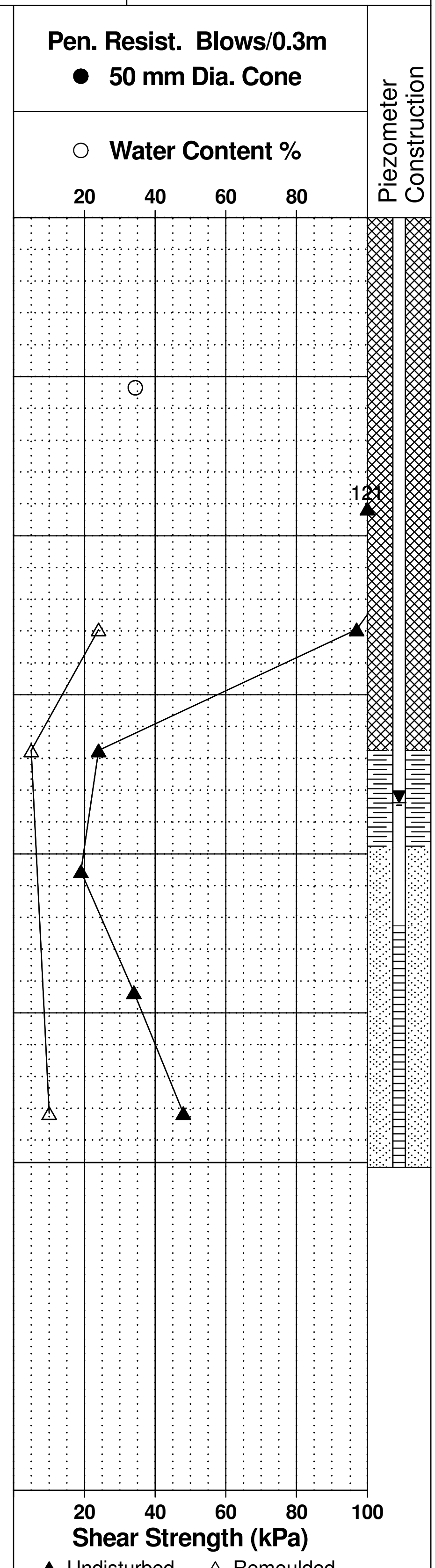
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 December 11

FILE NO. PG5521

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	87.28
TOPSOIL (0.15)		AU	1				
Very stiff to stiff, brown **SILTY CLAY** -Soft to firm and grey by 3.0 m depth -Firm to stiff by 4.5 m depth		SS	2	58	10	1	86.28
						2	85.28
						3	84.28
						4	83.28
						5	82.28
(5.94) End of Borehole (GWL @ 3.68 m December 17, 2020)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2275 Mer Bleue Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

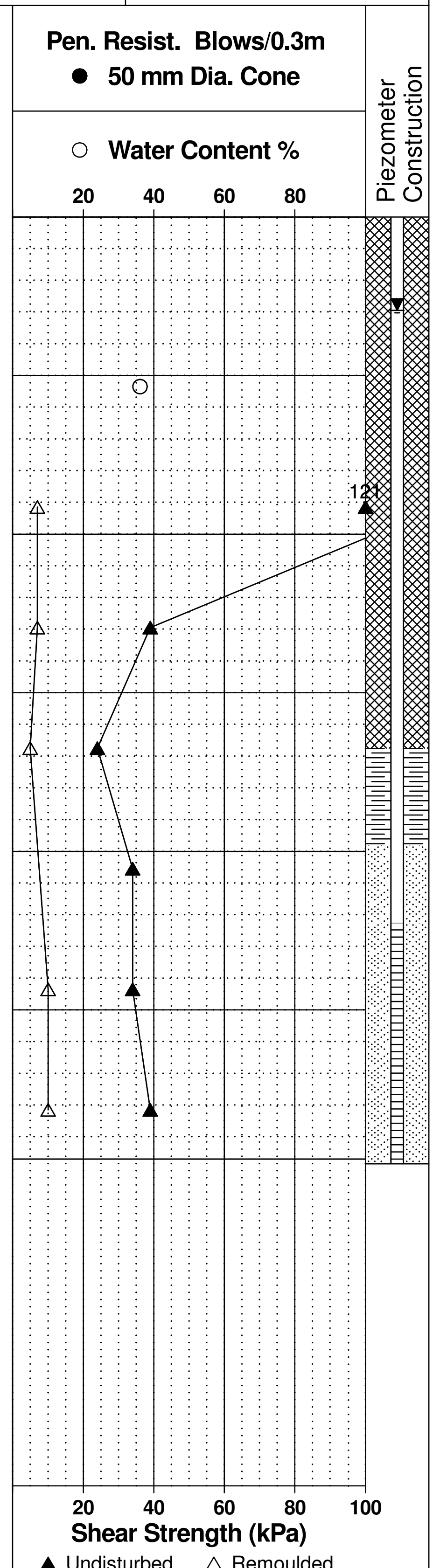
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 December 11

FILE NO. PG5521

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	87.22
TOPSOIL 0.15		AU	1				
Stiff to very stiff brown **SILTY CLAY**		SS	2	75	6	1	86.22
- Firm and grey by 2.9 m depth						2	85.22
						3	84.22
						4	83.22
						5	82.22
5.97							
End of Borehole							
(GWL @ 0.59 m December 17, 2020)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2275 Mer Bleue Rd.
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5521**

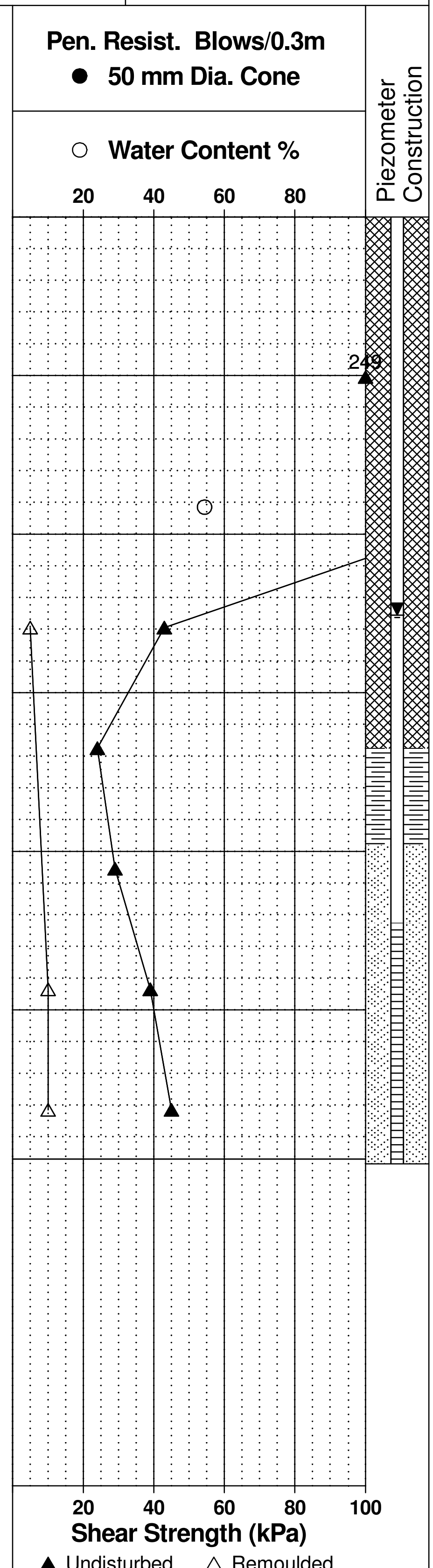
REMARKS

HOLE NO. **BH 3**

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 December 11

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.98
TOPSOIL 0.15		AU	1				
Hard to stiff, brown **SILTY CLAY**						1	85.98
- firm by 2.3 m depth		SS	2	100	4	2	84.98
- firm and grey by 3.0 m depth						3	83.98
						4	82.98
Dynamic Cone Penetration Test commenced at 6.10 m depth. Cone pushed to 23.90 m depth.						5	81.98
5.94							
End of Borehole							
Practical DCPT refusal at 23.90 m depth							
(GWL @ 2.51 m December 17, 2020)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2275 Mer Bleue Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

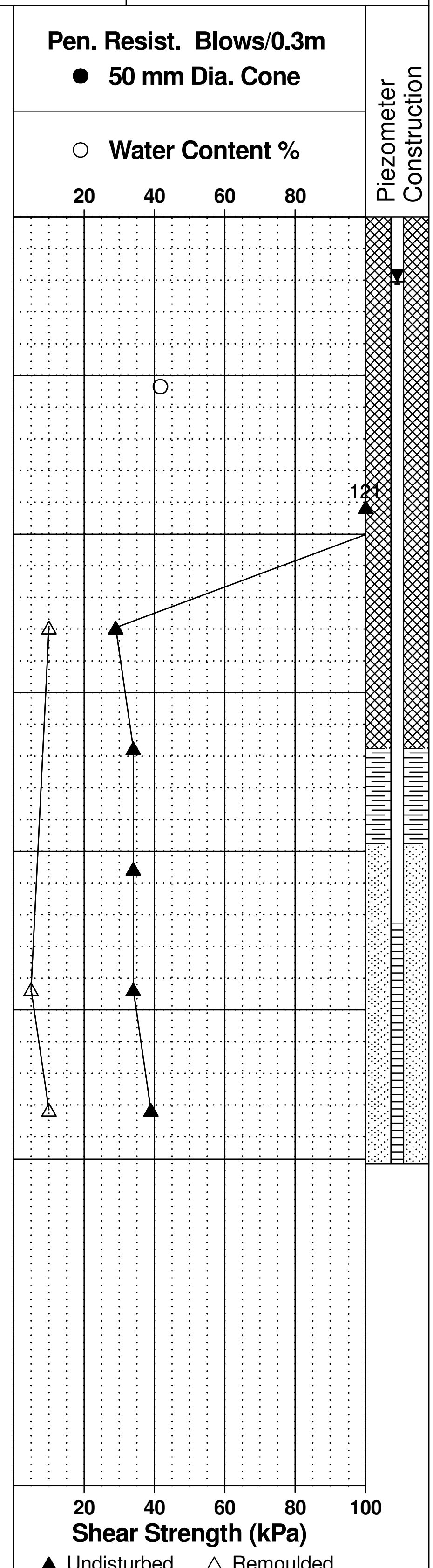
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 December 11

FILE NO. PG5521

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.96
TOPSOIL 0.15							
Very stiff, brown **SILTY CLAY**		AU	1				
- firm and grey by 2.7 m depth		SS	2	67	7	1	85.96
						2	84.96
						3	83.96
						4	82.96
						5	81.96
5.94							
End of Borehole							
(GWL @ 0.41 m December 17, 2020)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 2275 Mer Bleue Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

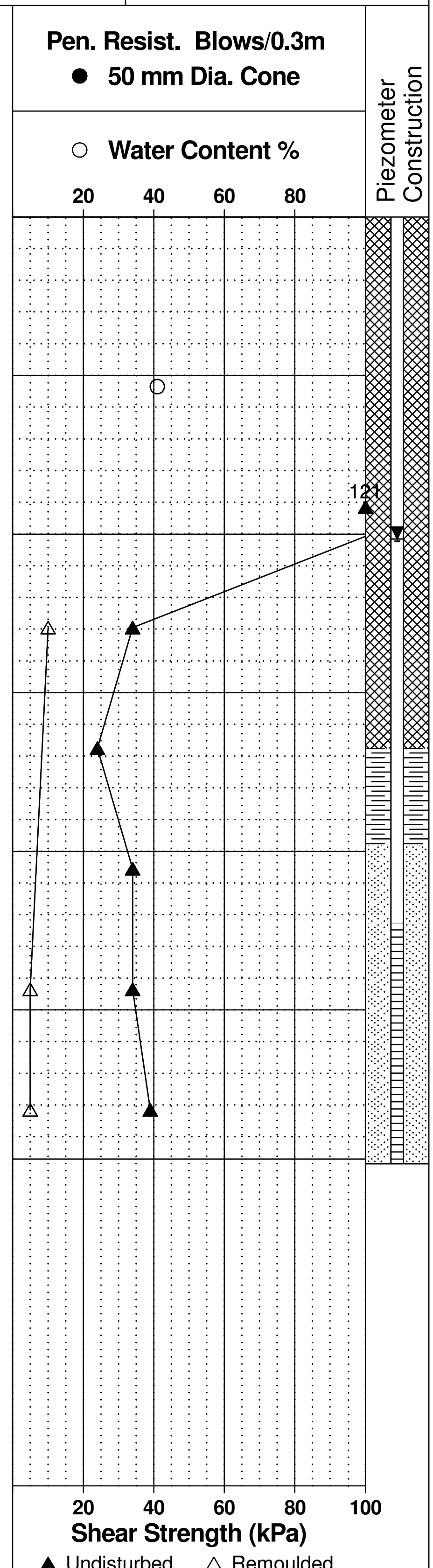
BORINGS BY CME-55 Low Clearance Drill

DATE 2020 December 11

FILE NO. PG5521

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.87
TOPSOIL 0.15		AU	1				
Very stiff, brown **SILTY CLAY** - firm by 2.5 m depth - firm and grey by 2.7 m depth		SS	2	75	8	1	85.87
						2	84.87
						3	83.87
						4	82.87
5.94						5	81.87
End of Borehole (GWL @ 2.03 m December 17, 2020)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

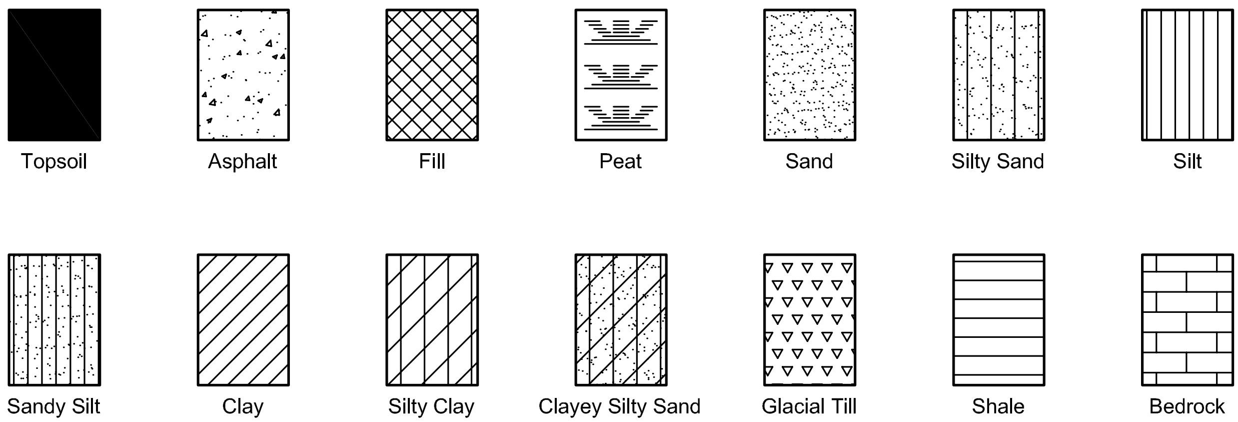
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

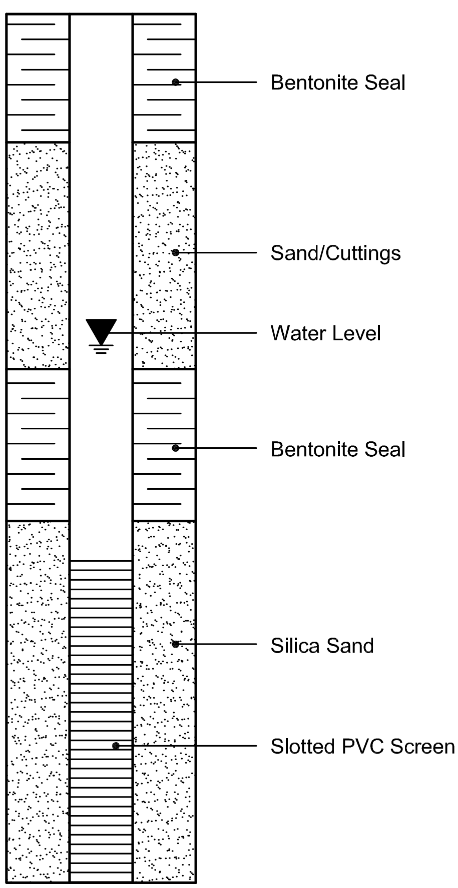
SYMBOLS AND TERMS (continued)

STRATA PLOT

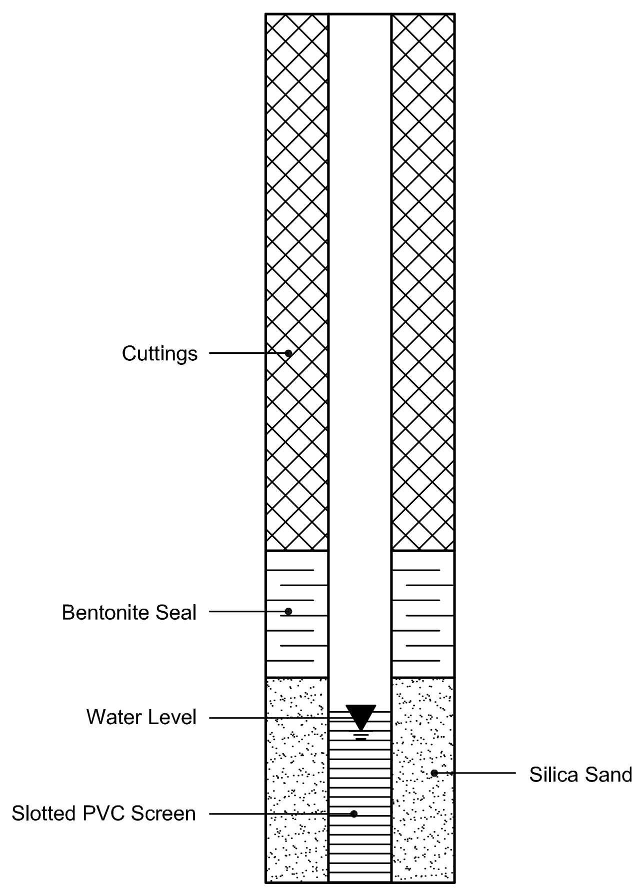


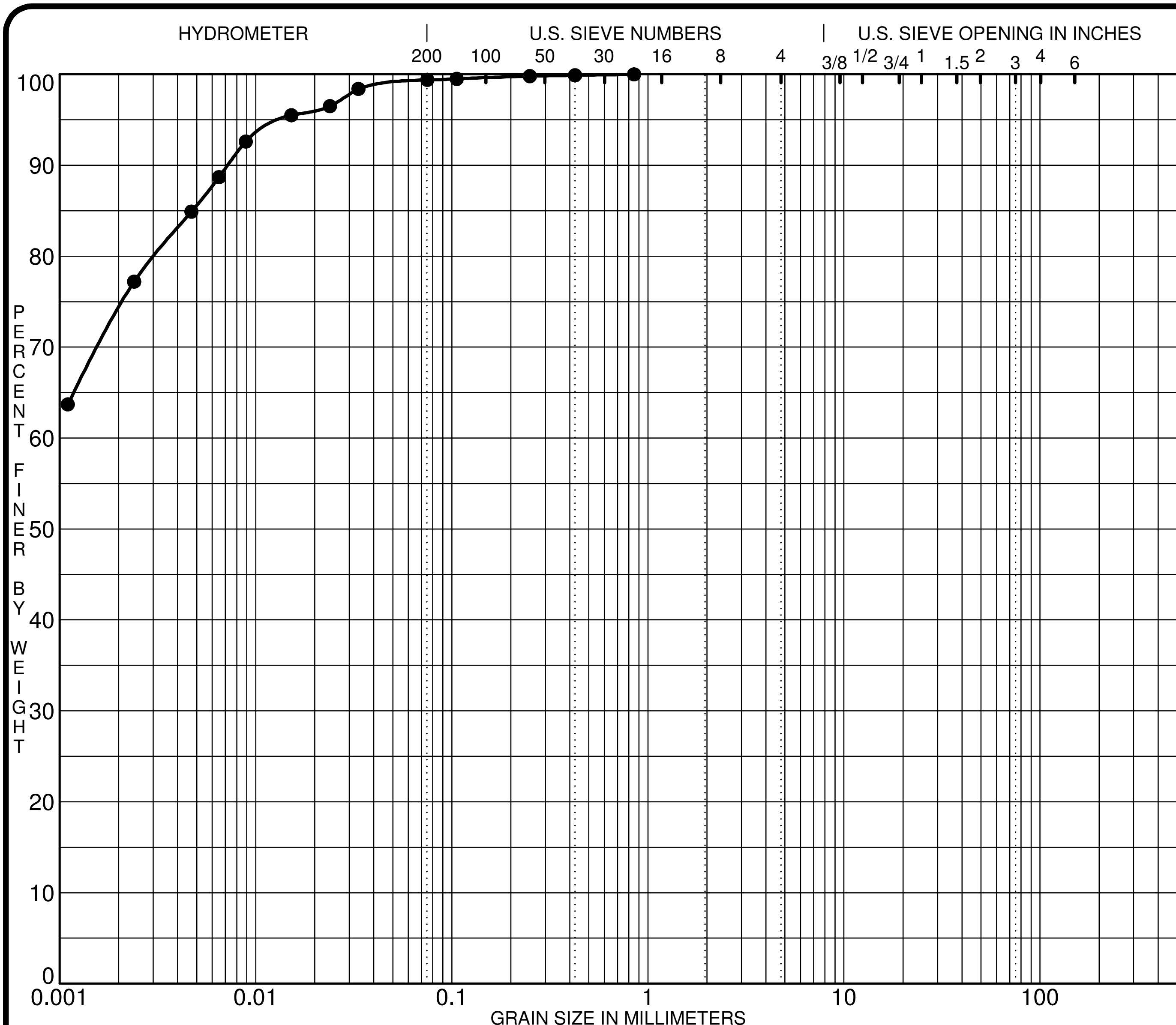
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 2 SS2**	**CH - Inorganic Clay of High Plasticity**	**36.1%**	**65**	**27**	**39**		

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 2 SS2**	**0.85**				**0.0**	**0.6**	**99.4**	

CLIENT **Caivan Communities**

FILE NO. **PG5521**

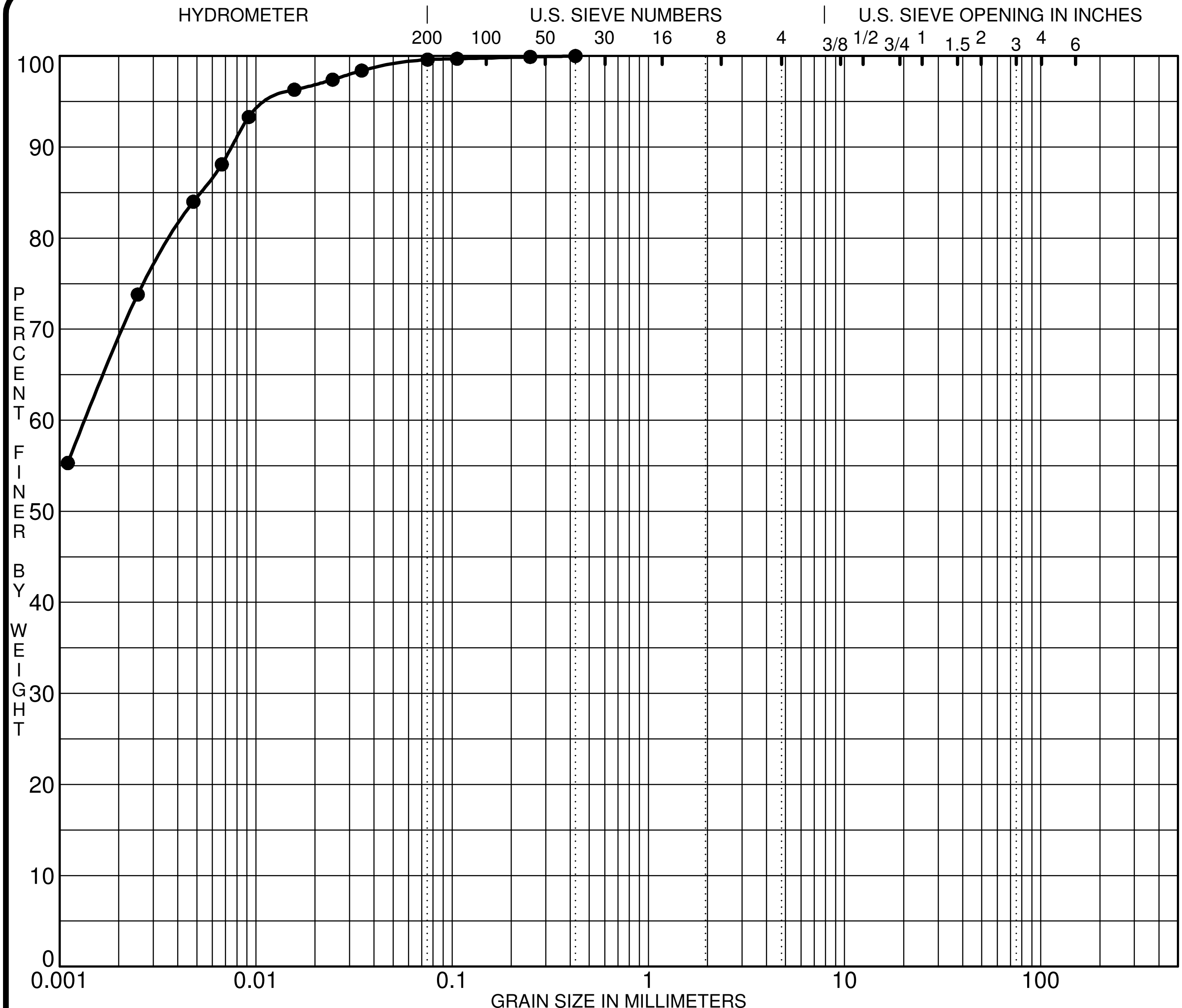
PROJECT **Geotechnical Investigation - Proposed Residential Development - 2275 Mer Bleue Rd.**

DATE **11 Dec 20**

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 5 SS2**	**CH - Inorganic Clay of High Plasticity**	**41%**	**68**	**29**	**39**		

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 5 SS2**	**0.43**	**0.00**			**0.0**	**0.4**	**99.6**	

CLIENT **Caivan Communities**

FILE NO. **PG5521**

PROJECT **Geotechnical Investigation - Proposed Residential Development - 2275 Mer Bleue Rd.**

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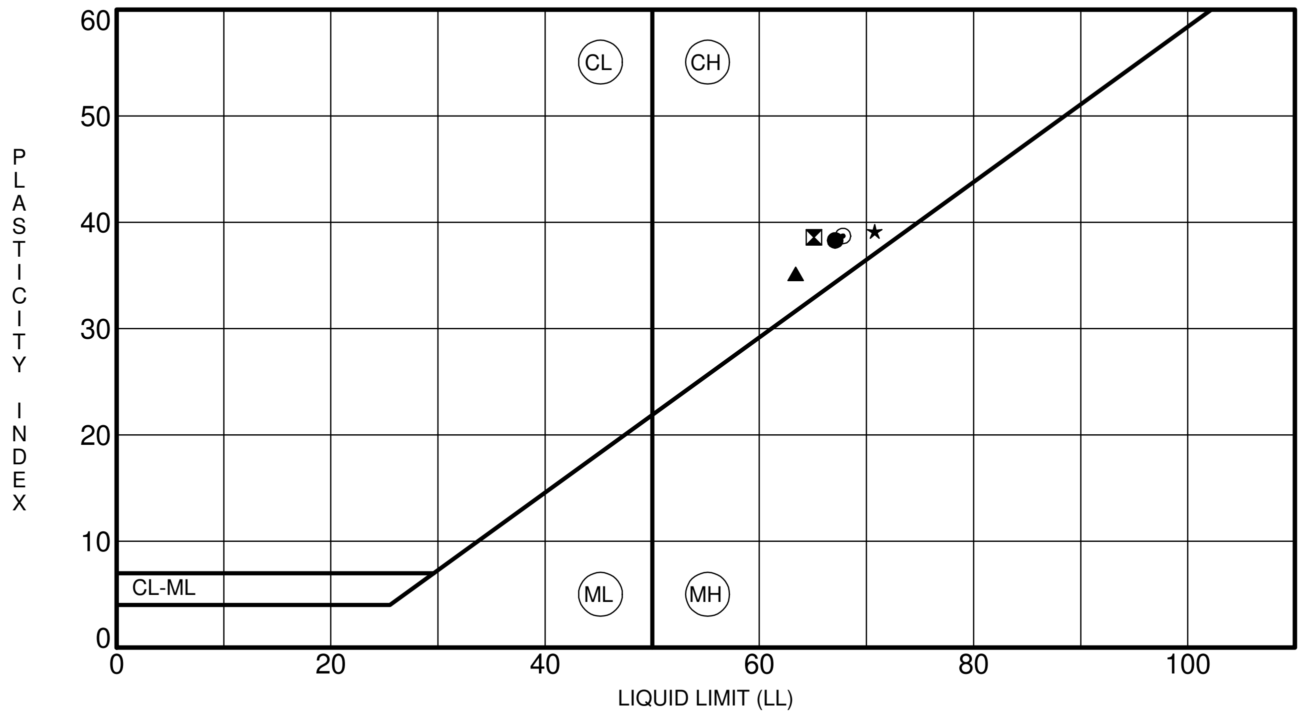
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 1 SS2**	**67**	**29**	**38**		**CH: Inorganic Clay of High Plasticity**
⧗	**BH 2 SS2**	**65**	**27**	**39**	**99.4**	**CH: Inorganic Clay of High Plasticity**
▲	**BH 3 SS2**	**63**	**28**	**35**		**CH: Inorganic Clay of High Plasticity**
★	**BH 4 SS2**	**71**	**32**	**39**		**CH: Inorganic Clay of High Plasticity**
⊙	**BH 5 SS2**	**68**	**29**	**39**	**99.6**	**CH: Inorganic Clay of High Plasticity**

CLIENT **Caivan Communities** FILE NO. **PG5521**

PROJECT **Geotechnical Investigation - Proposed Residential Development - 2275 Mer Bleue Rd.** DATE **11 Dec 20**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 31369

Report Date: 17-Dec-2020

Order Date: 11-Dec-2020

Project Description: PG5521

	Client ID:	BHC-20 SS2	-	-	-
	Sample Date:	11-Dec-20 15:00	-	-	-
	Sample ID:	2050622-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	62.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.61	-	-	-
Resistivity	0.10 Ohm.m	23.0	-	-	-
Anions					
Chloride	5 ug/g dry	258	-	-	-
Sulphate	5 ug/g dry	65	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5521-2 - TEST HOLE LOCATION PLAN

DRAWING PG5521-3 - PERMISSIBLE GRADE RAISE PLAN

DRAWING PG5521-4 - TREE PLANTING SETBACK PLAN

FIGURE 1

KEY PLAN

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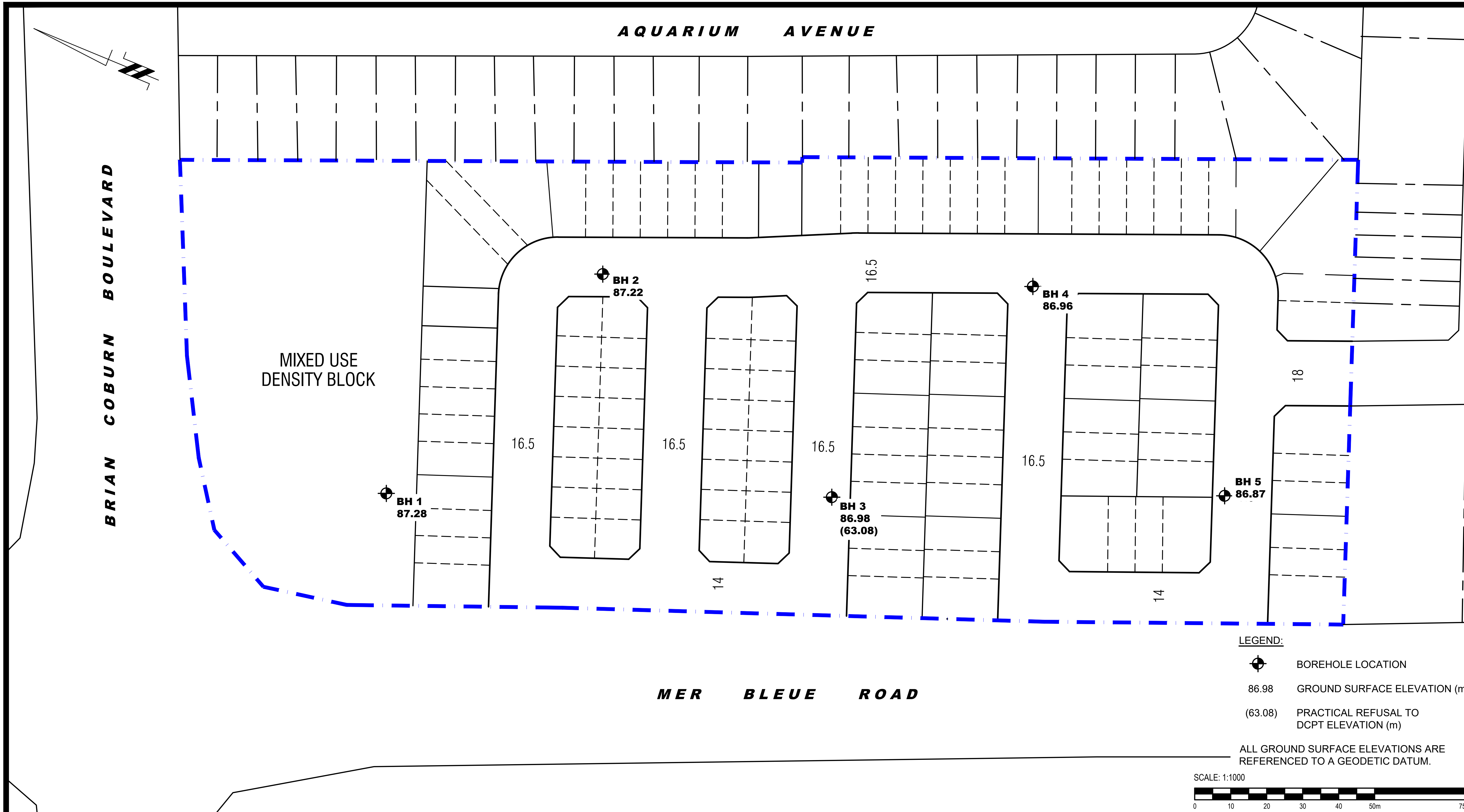

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

CAIVAN COMMUNITIES

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - 2275 MER BLEUE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 12/2020
Drawn by: NFRV	Report No.: PG5521-1
Checked by: DP	Dwg. No.: **PG5521-2**
Approved by: DJG	Revision No.:

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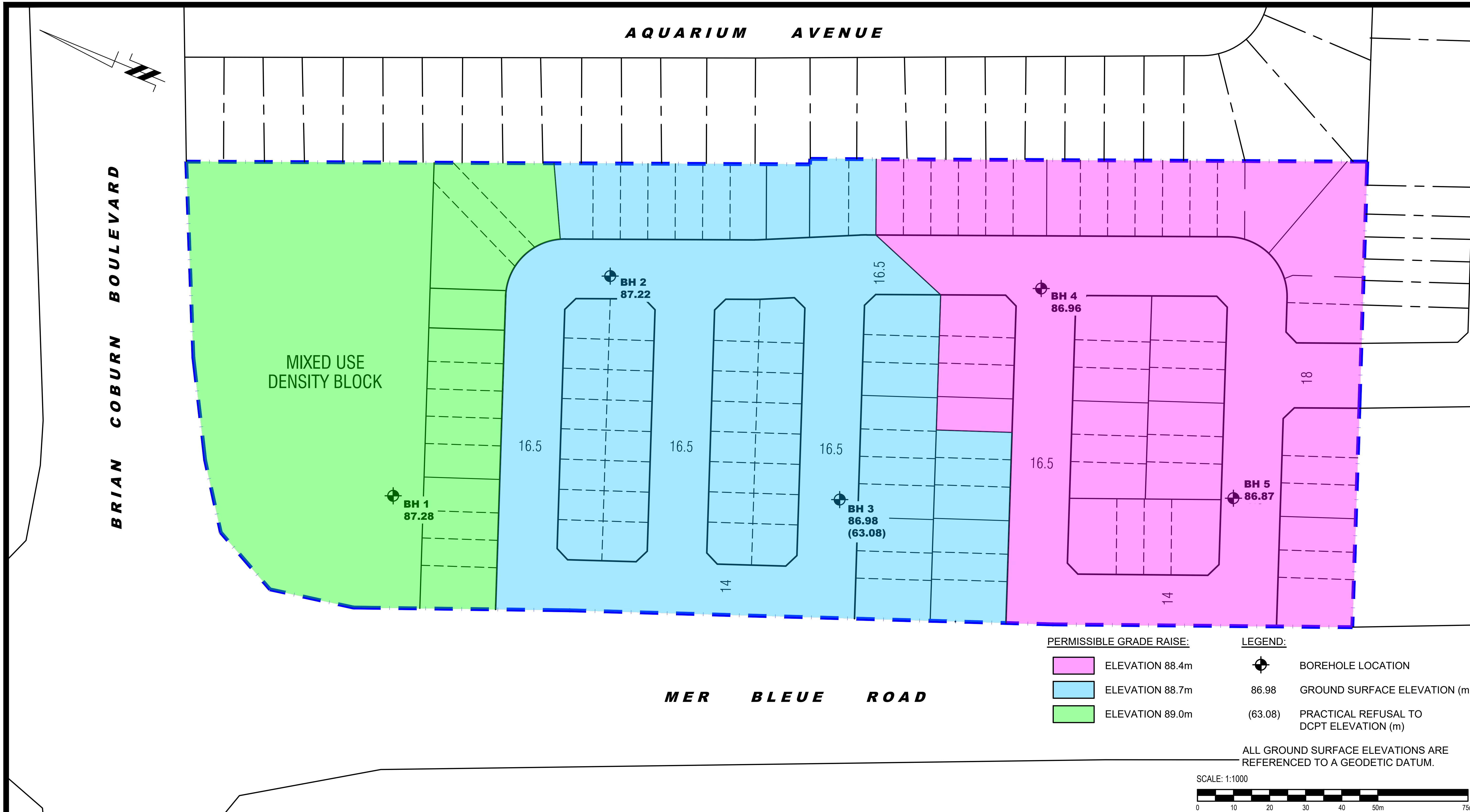

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NO.	REVISIONS	DATE	INITIAL

CAIVAN COMMUNITIES

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - 2275 MER BLEUE ROAD
OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:1000	Date: 12/2020
Drawn by: NFRV	Report No.: PG5521-1
Checked by: DP	Dwg. No.: **PG5521-3**
Approved by: DJG	Revision No.:

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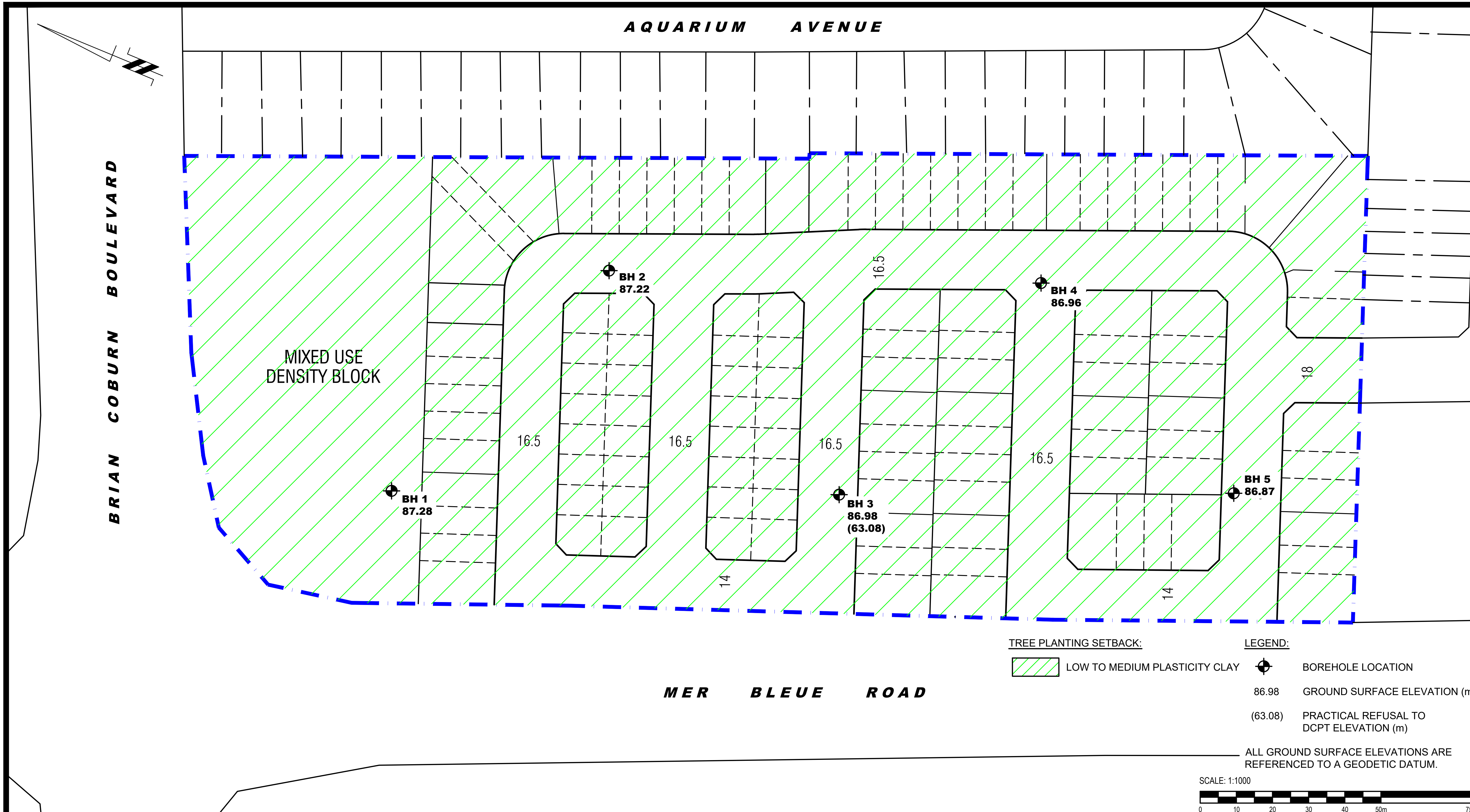

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NO.	REVISIONS	DATE	INITIAL

CAIVAN COMMUNITIES

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - 2275 MER BLEUE ROAD

OTTAWA, ONTARIO

Title:

TREE PLANTING SETBACK PLAN

Scale: 1:1000	Date: 12/2020
Drawn by: NFRV	Report No.: PG5521-1
Checked by: DP	Dwg. No.: **PG5521-4**
Approved by: DJG	Revision No.:

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