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Geotechnical Investigation

Proposed Residential Building
2458 Cleroux Crescent
Ottawa, Ontario

Prepared For

Melmar Group

April 1, 2022

Report: PG5973-1 Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Melmar Group to conduct a geotechnical investigation for the proposed residential building site to be located at 2458 Cleroux Crescent 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, it is understood that the proposed development will consist of a multi-storey residential building. It is also understood that up to one level of underground parking will be provided in the basement level. Associated access lanes, walkways, and landscaped areas are also anticipated as part of the development. It is expected that the proposed building will be municipally serviced.

It is expected that the existing building will be demolished in support of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on September 22, 2021 and consisted of advancing a total of three (3) boreholes to a maximum depth of 5.8 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 PG5973-1 - 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 boreholes BH 2-21. 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 PG5973-1 - 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. 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 subject site is currently occupied by a two-storey residential dwelling with an associated garage, paved driveway and two sheds. An approximately 1.8 m high retaining wall supporting the neighboring property was observed along the eastern property boundary. The majority of the subject site was covered with grass and plants. However, the rear portion of the subject site is densely covered by trees and vegetation. The ground surface across the subject site slopes downward in a north to south direction. An approximately 4 m high slope was observed along the rear portion of the subject site with the bottom of the slope located along the rear-property line.

The site is bordered by Cleroux Crescent followed by residential dwellings to the north, residential dwellings to the east and west, and vacant land to the south. The existing ground surface fronting Cleroux Crescent is at grade with the adjacent right-of-way.

4.2 Subsurface Profile

Generally, the soil profile encountered at the test hole locations consisted of topsoil underlain by silty sand in BH 2-21 and BH 3-21, and further by a deposit of silty clay at all boreholes. A silty sand deposit was encountered below the brown silty clay layer at BH 2-21 at an approximate depth of 1.6 m. The silty sand deposit was generally observed to consist of loose, reddish brown to brown silty sand. The silty sand was underlain by a stiff, grey silty clay deposit at an approximate depth of 3.3 m.

The silty clay was generally observed to consist of a hard to very stiff, brown weathered crust to depths ranging between 1.6 and 5.7 m below ground surface. The brown silty clay was observed to be underlain by a stiff to firm grey silty clay at BH 1-21.

A DCPT at borehole BH 2-21 commenced at 5.7 m below ground surface and was terminated at 30.5 m below ground surface. Refusal to DCPT was not encountered at the time of the investigation.

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 interbedded limestone and shale of the Lindsay 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
BH1-21 SS4	2.3 – 2.9	78	33	45	CH
BH2-21 SS6	3.8 – 4.4	53	27	26	CH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index;
CH: Inorganic Clay of High Plasticity

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

The results of the shrinkage limit test indicate a shrinkage limit of 25.22 and a shrinkage ratio of 1.650.

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 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 (%)
BH3-21	SS5	0.0	2.2	55.3	42.5

4.3 Groundwater

Groundwater levels were measured on September 27, 2021 within the installed polytube piezometers. The measured groundwater levels are presented in Table 3 below.

Table 3 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21	81.49	2.04	79.45	September 27, 2021
BH 2-21	81.45	4.15	77.30	
BH 3-21	80.66	4.33	76.33	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using 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 3.5 to 4.5 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. The proposed building may be founded on conventional spread footings placed on an undisturbed, hard to very stiff silty clay bearing medium.

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.

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.

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 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.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, very stiff brown silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **350 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.

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 bearing medium 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

Based on the undrained shear strength testing carried out within the silty clay layer, a permissible grade raise restriction of **2.0 m** above the existing ground surface may be considered for grading throughout 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 in-situ silty sand or silty clay bearing surface will be considered an acceptable subgrade upon which to commence backfilling for basement slab construction.

The recommended pavement structures noted in Subsection 5.7 will be applicable for the founding level of the proposed parking garage structure. 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 may consist of clear crushed stone.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Types I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab (outside the zones of influence of the footings).

All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

A sub-slab drainage system consisting of lines of perforated drainage pipes connected to a sump pump located within the lowest basement level. The spacing and layout of the sub-slab drainage pipes should be provided by the geotechnical consultant once the foundation layout has been finalized. The spacing may be subject to change as based on groundwater conditions encountered at the time of construction and reviewed by the geotechnical consultant. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for 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 expected (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 corresponding parameters are presented below.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/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 or 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Design

Car only parking areas and access lanes are expected as part of the proposed development. The proposed pavement structures are presented in Tables 4 and 5.

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. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terrafix 200W or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Table 4 – Recommended Pavement Structure – Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II
SUBGRADE – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, or concrete fill.	

Table 5 – Recommended Pavement Structure – Access Lanes

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, 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.

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

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 100 to 150 mm diameter perforated, corrugated plastic pipe which is surrounded on all sides by 150 mm of 19 mm clear crushed stone and is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. 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 drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Sub-Slab Drainage

Subfloor drainage is recommended to control water infiltration for the proposed structure to the buildings sump pit. For design purposes, a minimum of two sets each of north-south and east-west running lines of 150 mm diameter perforated, corrugated pipes should connect to the buildings sump pit. The layout of this subfloor system should be provided by the geotechnical consultant once design drawings for the lowest basement level which indicate the location of elevator shafts, sump pits and footings have been finalized and may be subsequently reviewed.

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 reviewed and approved by the geotechnical consultant) 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.

The foundation of the underground parking level is expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided at entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided to these areas and the perimeter of the building foundation.

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.

Excavation side slopes around the building excavation should be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes or other means along their footprint. Efforts should also be made to maintain dry surfaces at the bottom of excavation footprints and along the bottom of side slopes. Additional measures may be recommended at the time of construction by the geotechnical consultant.

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.

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.

The pipe bedding for sewer and water pipes placed on a relatively dry, undisturbed subgrade surface should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. 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 and silty sand 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.

Clay Seals

Where silty clay is encountered, 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

Groundwater Control for Building Construction

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

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 moderate to slightly aggressive corrosive environment.

6.8 Slope Stability Analysis

The rear portion of the subject site is bordered by a gradual slope. The slope conditions were reviewed by Paterson field personnel as part of the geotechnical investigation on September 22, 2021. The slope was noted to be in stable condition, covered with trees and with no signs of active erosion or movement. No watercourses or ponding water were observed at the bottom of the slope at the time of our visit. The slope was observed to be between 5 to 6 m high and appeared to have an approximate 2.5H:1V inclination across its surface.

A slope stability analysis was carried out to determine the required geotechnical setback from the top of slope. One (1) slope cross-section was studied as the worst-case scenario. The cross-section location is presented on Drawing PG5973-1 – Test Hole Location Plan in Appendix 2.

Slope Stability Analysis

The analysis of the stability of the slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's simplified method, which is a widely used and accepted analysis method.

The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is marginally stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable.

A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

The cross-section was analyzed based on the proposed conditions taking into consideration the existing conditions and features observed during our site visit, our review of the available geotechnical information, and our experience in the general area.

Subsoil conditions at the cross-section were inferred based on the subsoil information recovered during the geotechnical investigation and general knowledge of the geology of the area. The effective strength soil parameters used for static analysis are presented in Table 6 in the following page.

Table 6 – Effective Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay Crust	17	33	5
Grey Silty Clay	16	33	10
Concrete	20	Infinite Strength	

The total strength parameters for seismic analysis were chosen based on the in situ undrained shear strengths recovered within the boreholes completed at the time of the geotechnical investigation and based on our general knowledge of the geology of the area. The strength parameters used for the seismic analysis at the slope cross-sections are presented in Table 7 below.

Table 7 – Effective Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
Brown Silty Clay Crust	17	-	150
Grey Silty Clay	16	-	50
Glacial Till	20	Infinite Strength	

Static Loading Analysis

The results of the static analysis for the proposed conditions are shown on Figure 2A in Appendix 2. The results indicate a slope with factors of safety exceeding 1.5 beyond the top of slope at the analyzed section.

Based on these results, the slope and proposed dwelling are considered stable under static loading conditions.

Seismic Loading Analysis

An analysis considering seismic loading for the proposed site conditions was also completed at Section A. A horizontal ground acceleration of 0.16 g was considered for the slopes. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the seismic analysis for proposed conditions are shown on Figure 2B. The results indicate that the factor of safety is greater than 1.1 under seismic conditions. Based on these results, the slope and proposed structure are considered stable under seismic loading.

Geotechnical Setback – Limit of Hazard Lands

As noted above, no signs of active erosion were observed, and a watercourse is not present at the toe of slope. Therefore, a limit of hazard lands designation line is not required for the subject slope.

Conclusions and Recommendations

Based on these results, the existing slope located within the subject site is considered stable from a geotechnical perspective. A suitable factor of safety is present under static conditions and when considering seismic loading such that the proposed dwelling and associated development will not detrimentally impact the existing slope. It is also recommended that the future roof drains do not discharge storm water toward the existing slope.

6.9 Landscaping Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. A shrinkage limit test and sieve analysis testing were also completed on selected soil samples.

The shrinkage limit testing indicates a shrinkage limit of 25.2% with a shrinkage ratio of 1.65. The results of our Atterberg limit and sieve testing are presented in Appendix 1.

Based on our testing results, tree planting setback limits should be 7.5 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 tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

- The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

- Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree).

It should be noted that plants as shrubs and bushes in which root growth is typically limited to the upper 2 m of the overburden soils, may be planted within the 7.5 m setback.

6.10 Low-Impact Development Considerations

Should consideration be given to the implementation of low-impact development (LID) strategies, recommendations have been provided herein with regards to preliminary considerations for the subject site. These recommendations should not be considered as a substitute to site-specific infiltration and hydraulic conductivity testing of the subsoils prior to carrying out detailed design that would impact LID's. Infiltration rates provided herein are based on both published literature and experience at sites with similar soils in the Ottawa region, however, should only be considered theoretical from a design perspective.

Therefore, the geotechnical and/or hydrogeological consultant should advise on suitable LID strategies during the early stages of design of the future development. This would mitigate incorporating strategies that may be incompatible with the soils encountered at the subject site prior to a detailed design stage.

Based on the hydraulic conductivity estimates obtained from previous studies and published literature, typical infiltration rates for the in-situ silty clay soils encountered throughout the subject site is anticipated to range between <5 to 10 mm/hr. Based on this information, the silty clay overburden is generally considered to act as an aquitard layer.

It is our interpretation that groundwater will generally flow laterally and across the surface of the weathered brown silty clay and unweathered, saturated, grey silty clay layers, as opposed to vertically upwards or downwards through overburden soils with higher hydraulic conductivity. While small amounts of groundwater recharge and discharge could potentially take place on a localized scale where overburden thickness is minimal and/or contains notable fissures and seams of silt and sand, the geological conditions of the silty clay deposit are not suitable for recharge or discharge to be occurring on a large scale throughout the subject site.

Based on the results of the investigation, implementation of LID techniques consisting of draining stored water and run-off into the clay subsoils to promote groundwater recharge are not considered suitable for the subject site, from a geotechnical perspective. As previously discussed, the clay subsoil conditions at the subject site currently allow for only minimal volumes of recharge to occur. As such, the applicability of infiltration techniques is considered limited to LID measures such as perforated pipe-rear yard catch basins, bioretention and bioswales outletting directly to the storm sewer system and amended topsoil finishes used in conjunction with soak-away pits due to the presence of the less permeable clay soil.

Further and as discussed in Section 4.3 of this report, the long-term groundwater table can be expected at approximately 3.5 to 4.5 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

6.11 Neighboring Structures

Due to the presence of the neighboring retaining wall and its proximity to the future building excavation, it is recommended to confirm the founding conditions and construction of the retaining structure during the design stage of the proposed development and once the founding elevations of the proposed building have been finalized. The geotechnical consultant may then carry out an assessment of suitable measures that may be considered to maintain the existing retaining wall at the time of construction. Consideration may be given to undertaking an underpinning program and suitable lowering of the founding medium at the time of the building excavation.

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.
- Implementation of foundation drainage systems such as composite foundation drainage board.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

All excess soils must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

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 Melmar Group 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.

Drew Petahtegoose B.Eng.

LICENSED PROFESSIONAL ENGINEER
April 1, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

David J. Gilbert, P. Eng.

Report Distribution:

☐ Melmar Group (e-mail copy)
☐ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS
SYMBOLS AND TERMS
GRAIN-SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS
ATTERBERG LIMIT TESTING RESULTS
SHRINKAGE 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 - 2458 Cleroux Cres.
Ottawa, Ontario

DATUM Geodetic

REMARKS

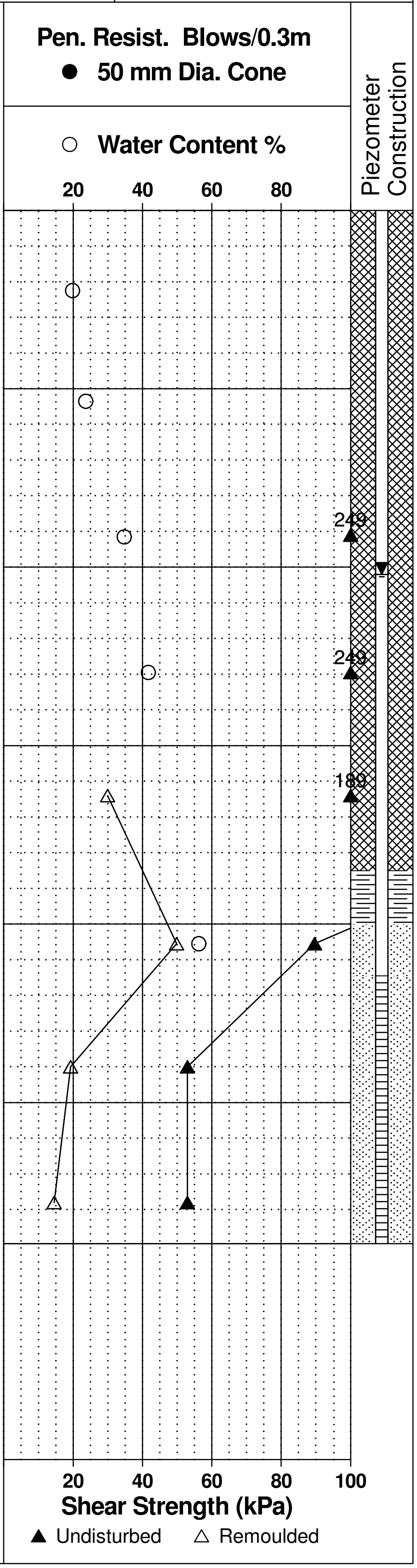
BORINGS BY CME-55 Low Clearance Drill

DATE September 22, 2021

FILE NO. PG5973

HOLE NO. BH 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	81.49
TOPSOIL 0.30							
		AU	1				
		SS	2	83	17	1	80.49
		SS	3	83	P	2	79.49
Hard to very stiff, reddish brown **SILTY CLAY**							
		SS	4	83	P	3	78.49
- stiff and grey by 4.0m depth		SS	5	75	P	4	77.49
						5	76.49
5.79 End of Borehole (GWL @ 2.04m - Sept. 27, 2021)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 2458 Cleroux Cres.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE September 22, 2021

FILE NO. PG5973

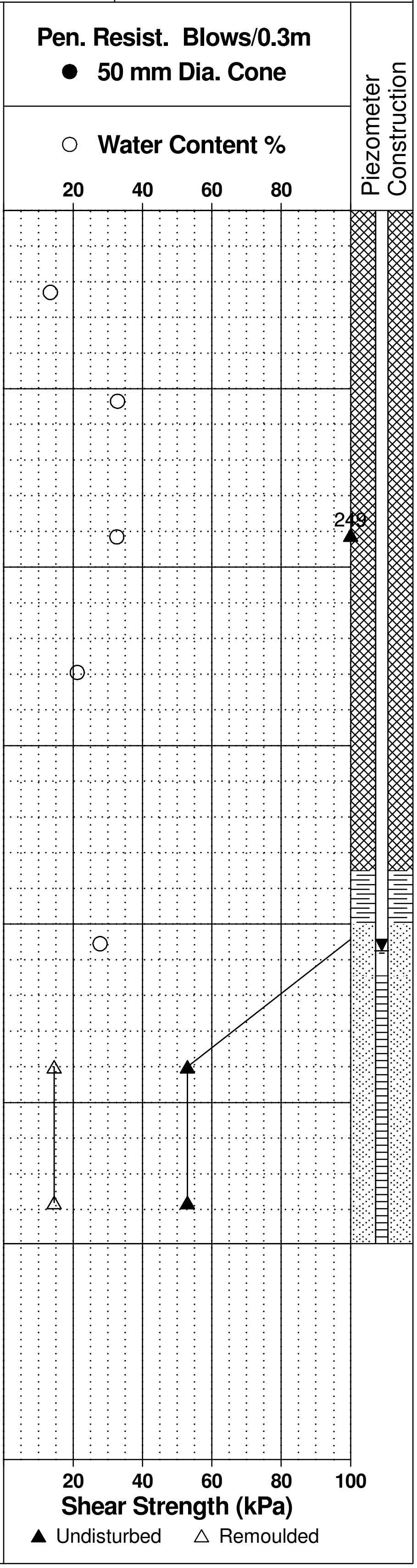
HOLE NO. BH 2-21

SOIL DESCRIPTION GROUND SURFACE		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
TOPSOIL	0.30				
Compact, brown **SILTY SAND**	0.60	AU	1		
Hard to very stiff, reddish brown **SILTY CLAY**	1.68	SS	2	83	16
Loose, reddish brown to brown **SILTY SAND**		SS	3	50	P
		SS	4	50	7
	3.35	SS	5	0	2
Stiff, grey **SILTY CLAY**	5.79	SS	6	83	2

Dynamic Cone Penetration Test commenced at 5.79m depth. Cone pushed to 30.5m depth, no cone refusal encountered, borehole terminated.

(GWL @ 4.15m - Sept. 27, 2021)

DEPTH (m)	ELEV. (m)
0	81.45
1	80.45
2	79.45
3	78.45
4	77.45
5	76.45



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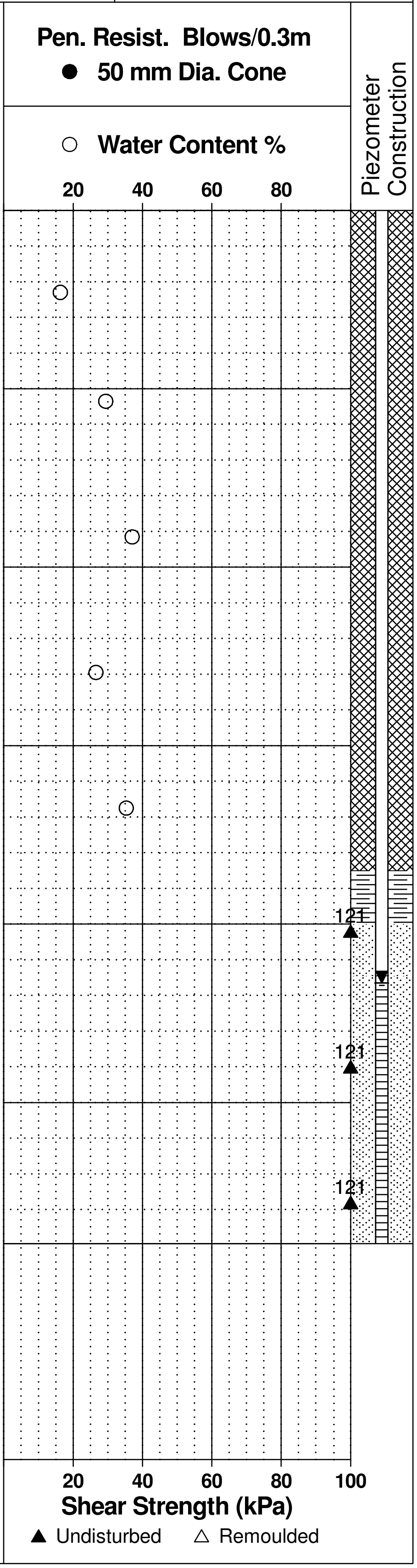
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 2458 Cleroux Cres.
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE September 22, 2021
FILE NO. PG5973
HOLE NO. BH 3-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	80.66
TOPSOIL 0.23							
Compact, brown **SILTY SAND** 0.60		AU	1				
		SS	2	83	16	1	79.66
		SS	3	75	19	2	78.66
		SS	4	83	17		
						3	77.66
		SS	5	83	17		
Very stiff, reddish brown **SILTY CLAY**						4	76.66
						5	75.66
5.79 End of Borehole							
(GWL @ 4.33m - Sept. 27, 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 strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x 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)

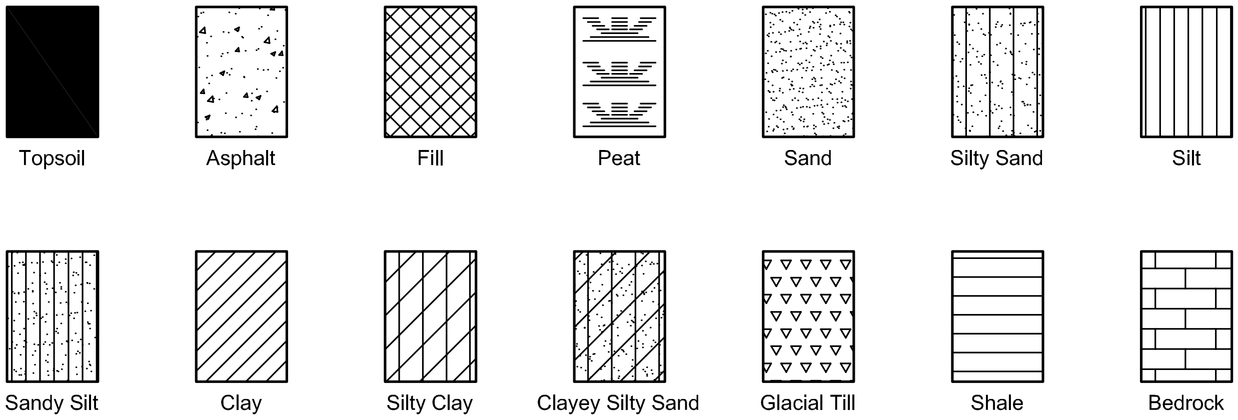
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

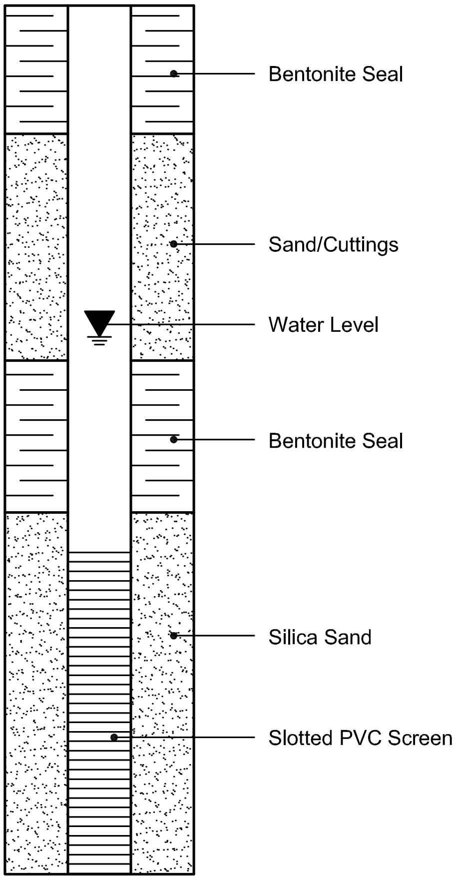
SYMBOLS AND TERMS (continued)

STRATA PLOT

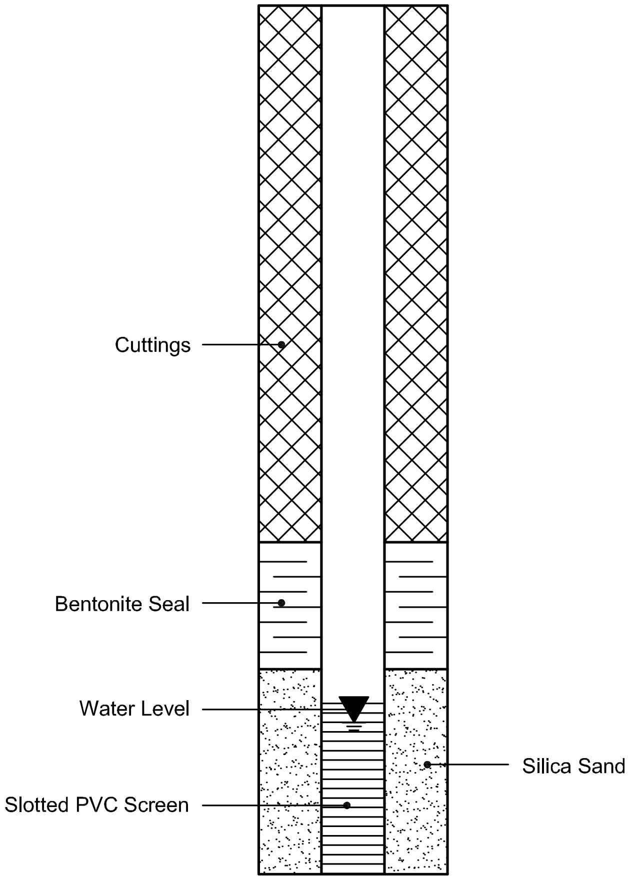


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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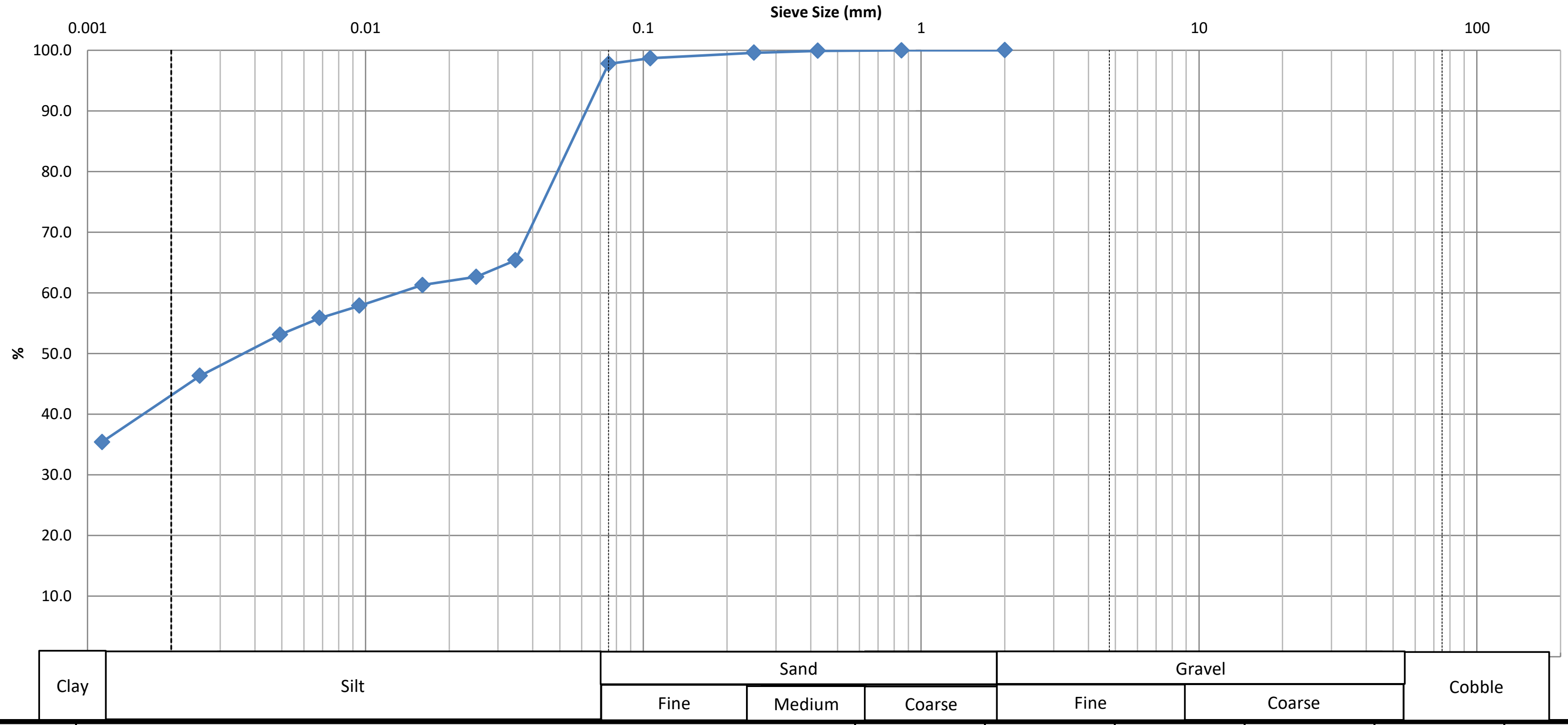
SIEVE ANALYSIS
ASTM C136

CLIENT:	Melmar Group	DEPTH:	3.0 - 3.6m	FILE NO:	PG5973
CONTRACT NO.:		BH OR TP No.:	BH3-21 SS5	LAB NO:	29215
PROJECT:	2458 Cleroux Cres.			DATE RECEIVED:	22-Sep-21
				DATE TESTED:	27-Sep-21
DATE SAMPLED:	22-Sep-21			DATE REPORTED:	1-Oct-21
SAMPLED BY:	I. Khan			TESTED BY:	CS



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		36.2					

	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
					0.0	2.2	55.3	42.5

Comments:

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

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consulting engineers

HYDROMETER
LS-702 ASTM-422

CLIENT:	Melmar Group	DEPTH:	3.0 - 3.6m	FILE NO.:	PG5973
PROJECT:	2458 Cleroux Cres.	BH OR TP No.:	BH3-21 SS5	DATE SAMPLED	22-Sep-21
LAB No. :	29215	TESTED BY:	CS	DATE RECEIVE	22-Sep-21
SAMPLED BY:	I. Khan	DATE REPT'D:	1-Oct-21	DATE TESTED:	27-Sep-21

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
155.0	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	72.61	TARE WEIGHT	50.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	1.13	AIR DRY	122.30	72.30
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	155.00	105.00
		CORRECTED	1.452	

GRAIN SIZE ANALYSIS

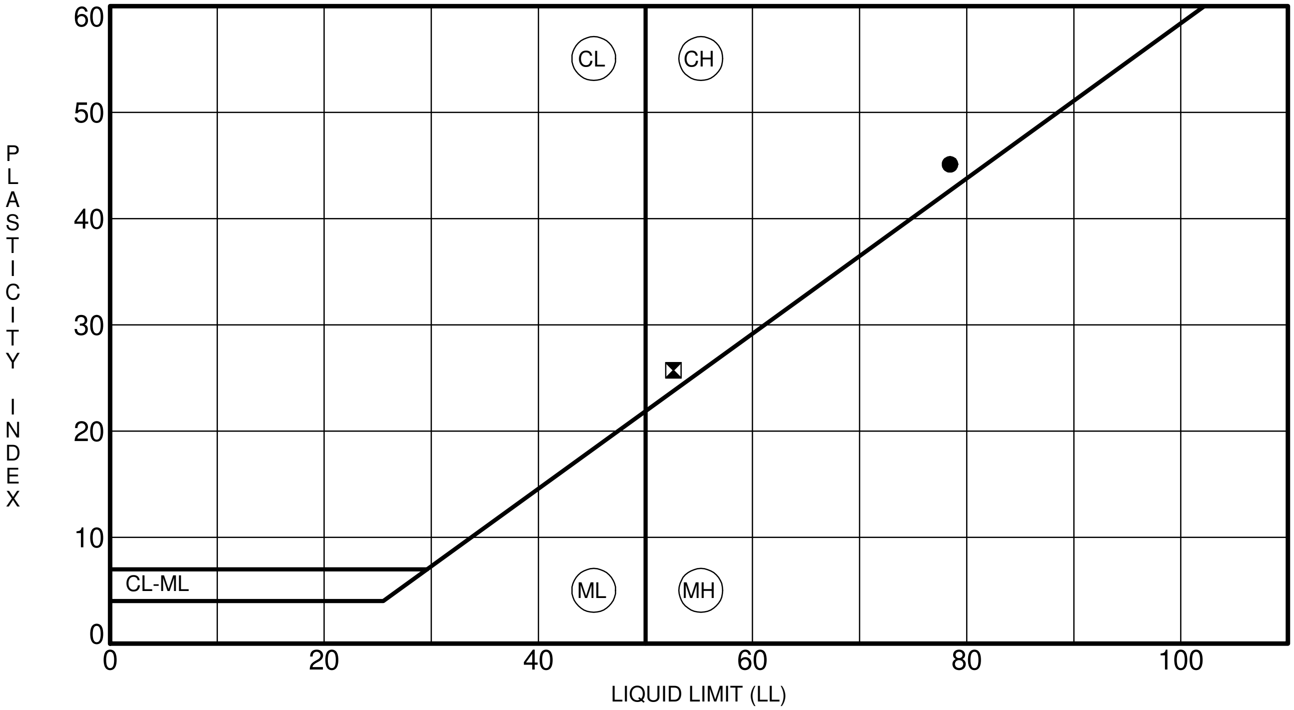
SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75			
2.0	**0.0**	0.0	**100.0**
Pan	**155.0**		
0.850	**0.01**	0.0	**100.0**
0.425	**0.05**	0.1	**99.9**
0.250	**0.20**	0.4	**99.6**
0.106	**0.66**	1.3	**98.7**
0.075	**1.11**	2.2	**97.8**
Pan	**1.13**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (°C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	8:32	54.0	6.0	23.0	0.0346	65.4	**65.4**
2	8:33	52.0	6.0	23.0	0.0251	62.6	**62.6**
5	8:36	51.0	6.0	23.0	0.0160	61.3	**61.3**
15	8:46	48.5	6.0	23.0	0.0095	57.9	**57.9**
30	9:01	47.0	6.0	23.0	0.0068	55.8	**55.8**
60	9:31	45.0	6.0	23.0	0.0049	53.1	**53.1**
250	12:41	40.0	6.0	23.0	0.0025	46.3	**46.3**
1440	8:31	32.0	6.0	23.0	0.0011	35.4	**35.4**

Moisture = 36.19%

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



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1-21**	**SS4**	**78**	**33**	**45**		**CH - Inorganic clays of high plasticity**
⧗	**BH 2-21**	**SS6**	**53**	**27**	**26**		**CH - Inorganic clays of high plasticity**

CLIENT **Melmar Group**

FILE NO. **PG5973**

PROJECT **Geotechnical Investigation - Prop. Residential Development - 2458 Cleroux Cres.**

DATE **22 Sep 21**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

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Linear Shrinkage
ASTM D4943-02

CLIENT:	Melmar Group	DEPTH	3.8m to 4.4m	FILE NO.:	PG5973
PROJECT:	2458 Clearoux Crescent	BH OR TP No:	BH1-21 SS5	DATE SAMPLED	22-Sep
LAB No:	29213	TESTED BY:	DJ / CS	DATE RECEIVED	23-Sep
SAMPLED BY:	IK	DATE REPORTED:	01-Oct-21	DATE TESTED	30-Sep

LABORATORY INFORMATION & TEST RESULTS

Moisture	No. of Blows(6)	Calibration (Two Trials)	Tin NO.(x33)	
Tare	4.66	Tin	4.48	4.48
Soil Pat Wet + Tare	62.36	Tin + Grease	4.71	4.72
Soil Pat Wet	57.7	Glass	48.97	48.97
Soil Pat Dry + Tare	36.13	Tin + Glass + Water	91.04	91.07
Soil Pat Dry	31.47	Volume	37.36	37.38
Moisture	**83.35**	**Average Volume**	**37.37**	

Soil Pat + String	31.61
Soil Pat + Wax + String in Air	33.75
Soil Pat + Wax + String in Water	12.27
Volume Of Pat (Vdx)	21.48

RESULTS:

Shrinkage Limit	**25.22**
Shrinkage Ratio	**1.650**
Volumetric Shrinkage	**95.906**
Linear Shrinkage	**20.079**

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

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 24535

Report Date: 30-Sep-2021

Order Date: 24-Sep-2021

Project Description: PG5973

	Client ID:	BH1-21 / SS3	-	-	-
	Sample Date:	22-Sep-21 09:00	-	-	-
	Sample ID:	2139526-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	70.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.17	-	-	-
Resistivity	0.10 Ohm.m	51.5	-	-	-
Anions					
Chloride	5 ug/g dry	24	-	-	-
Sulphate	5 ug/g dry	42	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – SLOPE STABILITY ANALYSIS CROSS-SECTIONS

DRAWING PG5973-1 – TEST HOLE LOCATION PLAN

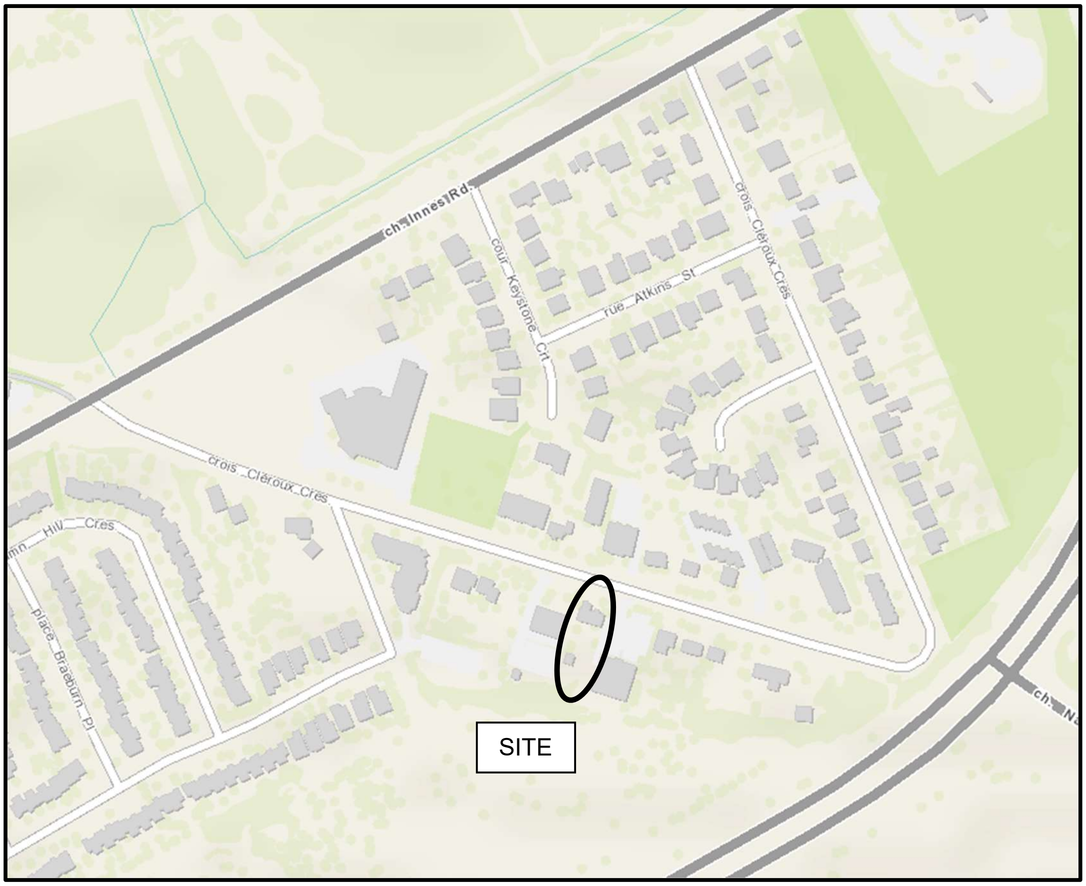


FIGURE 1

KEY PLAN

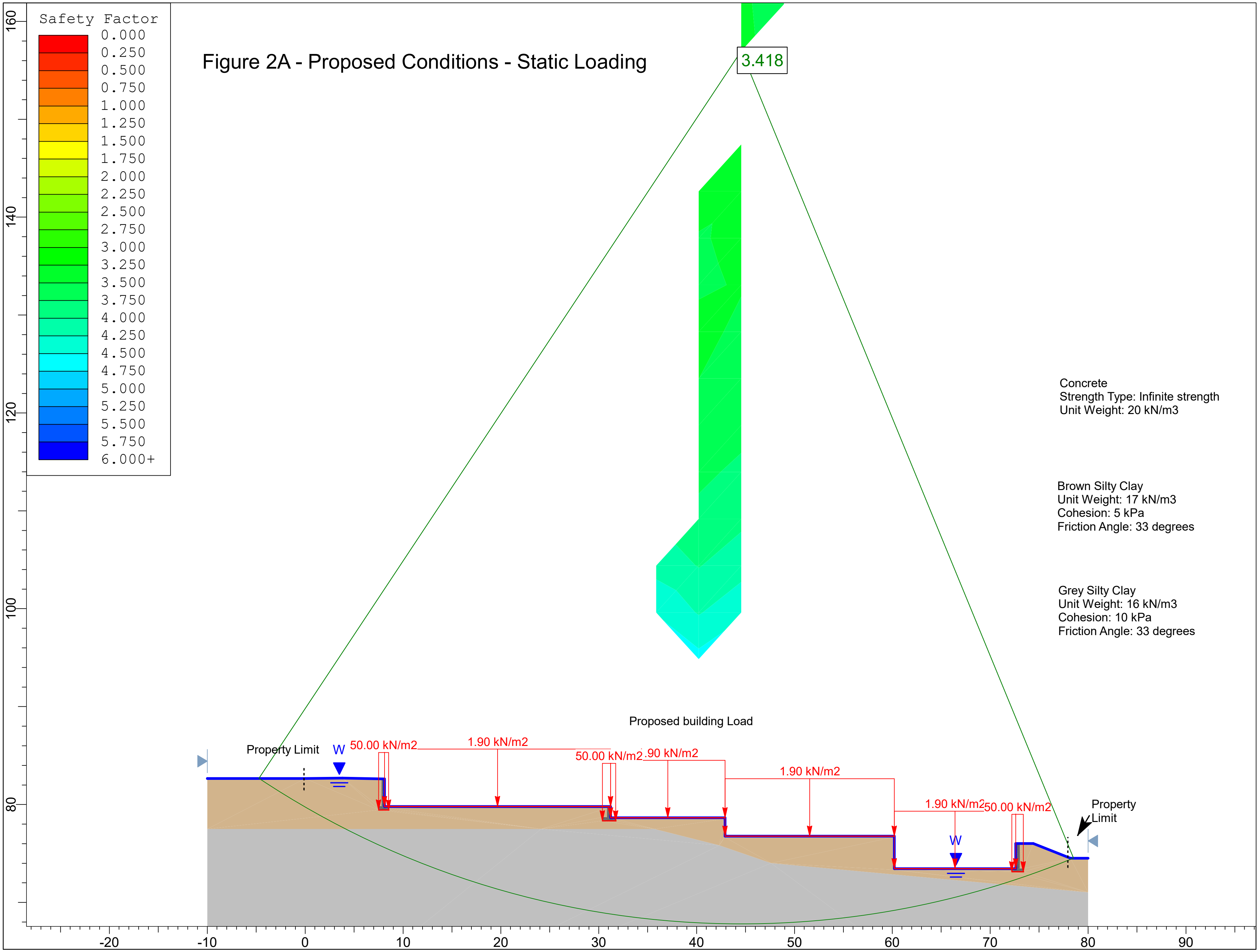


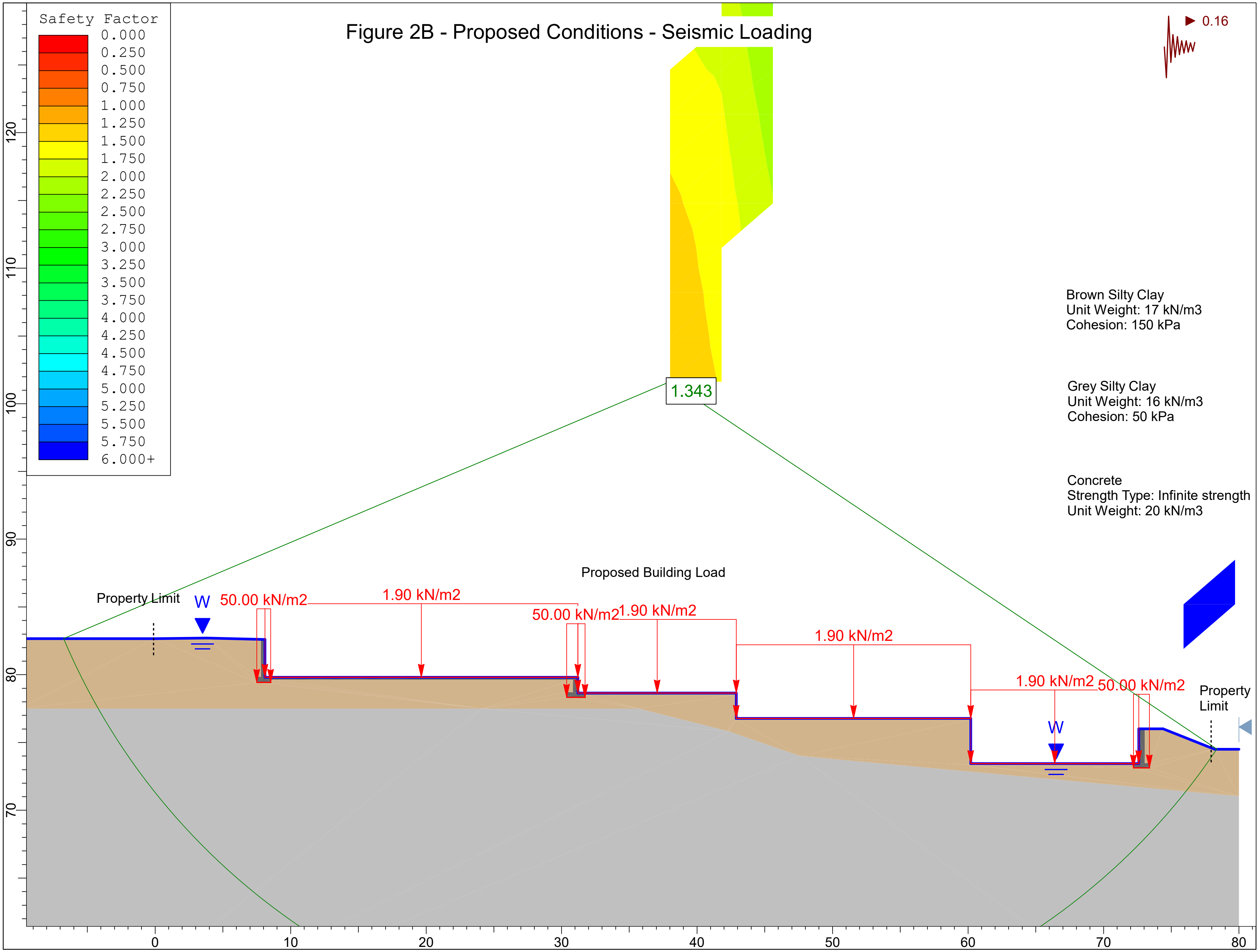
Figure 2A - Proposed Conditions - Static Loading

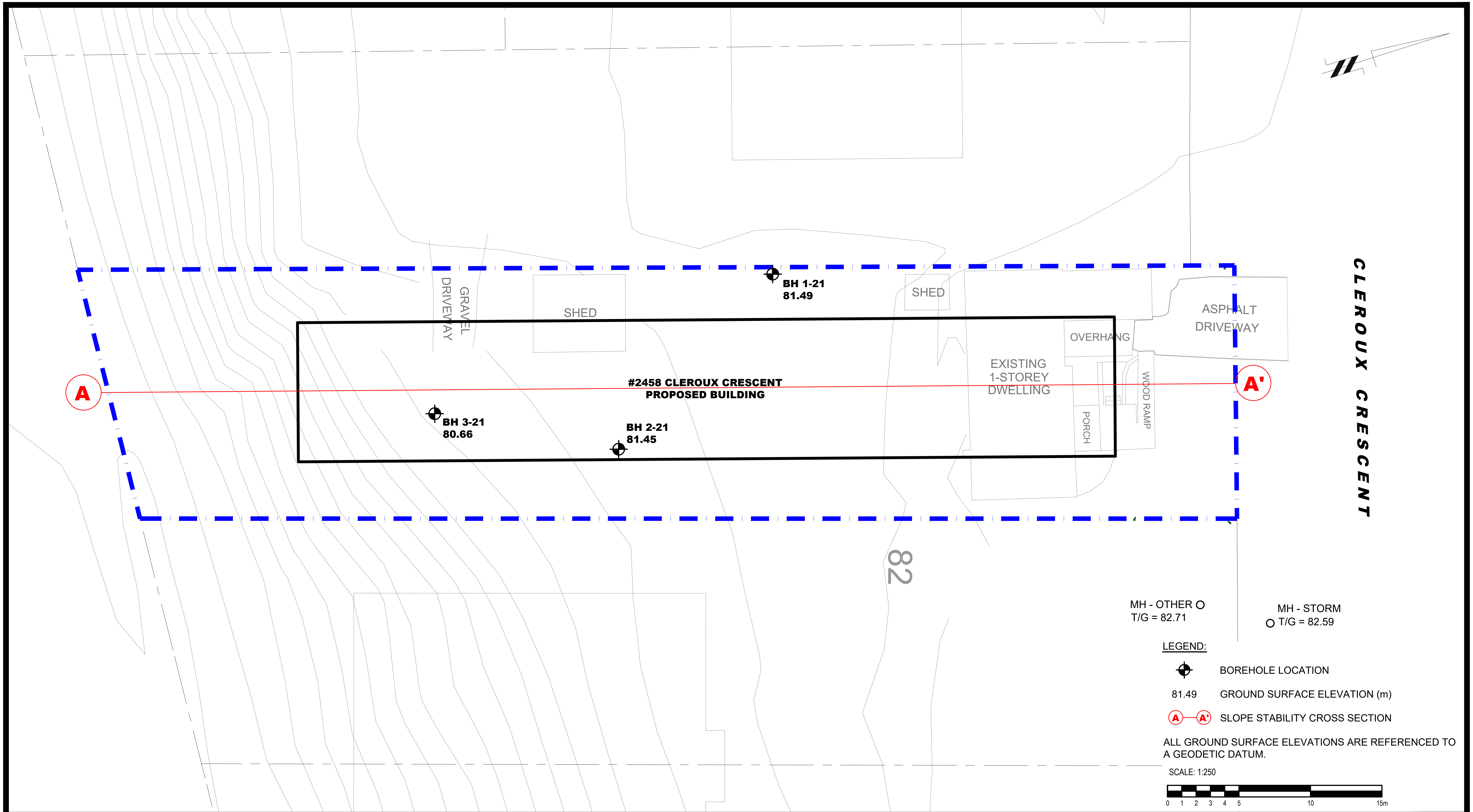
Concrete
Strength Type: Infinite strength
Unit Weight: 20 kN/m3

Brown Silty Clay
Unit Weight: 17 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees

Grey Silty Clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees

Figure 2B - Proposed Conditions - Seismic Loading





ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

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NO.	REVISIONS	DATE	INITIAL

MELMAR GROUP
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
2458 CLEROUX CRESCENT
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 03/2022
Drawn by: NFRV
Report No.: PG5973-1
Checked by: DP
Dwg. No.: PG5973-1
Approved by: DJG
Revision No.:

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