Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa, Ontario
Canada K2E 7J5

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

patersongroup

Geotechnical Investigation

Proposed St. Faustina Parish
2835 Old Montreal Road
Ottawa, Ontario

Prepared For

Mr. Ned Lathrop

March 22, 2022

Report: PG4197-1 Revision 1

Table of Contents

Page

1.0 Introduction . . . 1

2.0 Proposed Development . . . 1

3.0 Method of Investigation . . . 2
3.1 Field Investigation . . . 2
3.2 Field Survey . . . 3
3.3 Laboratory Review and Testing . . . 3

4.0 Observations
4.1 Surface Conditions . . . 4
4.2 Subsurface Profile . . . 4
4.3 Groundwater . . . 5

5.0 Discussion
5.1 Geotechnical Assessment . . . 6
5.2 Site Grading and Preparation . . . 6
5.3 Foundation Design . . . 7
5.4 Design for Earthquakes . . . 8
5.5 Basement Slab . . . 8
5.6 Basement Wall . . . 9
5.7 Pavement Structure . . . 10

6.0 Design and Construction Precautions
6.1 Foundation Drainage and Backfill . . . 12
6.2 Protection of Footings Against Frost Action . . . 13
6.3 Excavation Side Slopes . . . 13
6.4 Pipe Bedding and Backfill . . . 14
6.5 Groundwater Control . . . 14
6.6 Winter Construction . . . 15
6.7 Slope Stability Assessment . . . 15
6.8 Landscaping Considerations . . . 18

7.0 Recommendations . . . 20

8.0 Statement of Limitations . . . 21

Appendices

Appendix 1 Borehole Logs by Others
Laboratory Test Results by Others

Appendix 2 Figure 1 - Key Plan
Figures 2 & 3 - Slope Stability Cross Sections
Drawing PG4197-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Mr. Ned Lathrop to conduct a geotechnical investigation for the proposed St. Faustina Parish to be located on 2835 Old Montreal Road in Cumberland, in the City of Ottawa (reference should be made to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of this geotechnical investigation report were to:

- ❏ Determine the subsoil and groundwater conditions at this site by means of test holes.

- ❏ Provide geotechnical recommendations pertaining to the design of the proposed building 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

It is our understanding that the proposed development will consist of a one-storey church building with a basement level and a bell tower. Associated at-grade parking areas, access lanes and landscaped areas are also anticipated as part of the proposed development. It is anticipated the subject site will privately serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

A field investigation program was completed at the subject site by others on March 13, 2017 and April 21, 2017. At that time, a total of five (5) boreholes and ten (10) test pits were completed across the subject site to a maximum depth of 10.1 m below the ground surface. The test hole locations are shown on Drawing PG4197-1 - Test Hole Location Plan included in Appendix 2.

Sampling and In Situ Testing

It is understood that soil samples were recovered using a 51 mm open diameter split-spoon sampler. The depths at which the split-spoon samples were recovered from the boreholes are shown as SS on the Borehole Logs by Others in Appendix 1.

It is understood that a Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “blows/0.3m” values on the Borehole Logs by others in Appendix 1. The overburden thickness was evaluated using a dynamic cone penetration test (DCPT) completed at boreholes BH 17-2, BH 17-3, and BH 17-5.

It is further understood that undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

The soil profiles are logged on the Borehole Logs by Others presented in Appendix 1 of this report.

Groundwater

Borehole BH 17-2 was fitted with a flexible standpipe piezometer to allow for groundwater level monitoring. Groundwater level observations are discussed in Section 4.3 and are presented in the Borehole Logs by Others in Appendix 1.

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by others. It is understood that the elevations at each test hole location were surveyed using a laser level and were referenced to the base of a hydro pole in the middle of the north property lines with an assumed elevation of 100.00 m. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG4197-1 Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

It is understood that selected samples of soil were tested for water content, grain size distribution, and Atterberg limits. The results of the testing are presented in Subsection 4.2 and on Laboratory Test Results by Others in Appendix 1.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and generally vacant. The existing ground surface slopes down from southwest to northeast, towards the Ottawa River.

The site is bordered to the north by Regional Road 174 followed by the Ottawa River, to the east and west by residential properties and to the south by Old Montreal Road. The existing ground surface across the subject site is relatively flat and at grade with surrounding roadways and properties. The area following Regional Road 174 and along Ottawa River is composed of a sloped terrain and valley corridors.

Due to the presence of the slopes bordering the Ottawa River and their proximity to the subject site, a slope stability assessment was carried out considering the slope conditions present in the subject site and described above. The results of the slope stability assessment are discussed further in Subsection 6.7 of this report.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consisted of a thin layer of topsoil overlying a silty clay deposit. The topsoil was underlain by a layer of silty sand at boreholes BH 17-1, BH 17-3, and BH 17-5.

The silty sand deposit generally consisted of loose brown sand. The deposit was observed to extend to approximate depths ranging from 1.2 m to 5.8 m below ground surface.

The silty clay deposit generally consisted of a stiff brown weathered crust and/or a firm grey silty clay. The weathered crust was observed at boreholes BH 17-2 and BH 17-4 extending to depths ranging between 1.8 and 2.1 m below ground surface. The firm grey silty clay deposit was observed at all boreholes.

Practical refusal to DCPT was encountered at an approximate depth of 16 m, 16.1 m and 17.4 m below existing ground surface at the location of boreholes BH 17-2, BH 17-3 and BH 17-5, respectively.

Reference should be made to the Borehole Logs by Others presented in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in the subject site consists of Paleozoic Dolomite of the Oxford formation with an overburden drift thickness of 10 to 50 m depth.

4.3 Groundwater

Groundwater levels were measured by others in the installed piezometer at borehole BH 17-2 approximately 3 weeks after completion of the drilling program. Observations are presented in the Borehole Logs by Others in Appendix 1 and in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)	
		Depth	Elevation
BH 17-2	101.30	2.00	99.30

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 levels can also be estimated based on the observed color, moisture levels and consistency of the recovered samples. Based on our interpretation of the observations by other, the long-term groundwater level is expected between 3.0 and 5.0 m depth. However, it should be noted that groundwater levels are subject to seasonal fluctuations and, therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. The proposed structure may be founded on conventional shallow spread footings bearing on an undisturbed stiff silty clay or silty sand deposit.

Due to the presence of the silty clay deposit, the subject site will have a permissible grade raise restriction. The permissible grade raise recommendations are discussed in Subsection 5.3

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 organics, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities.

Fill Placement

Fill used for grading beneath the building footprints, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The material should be tested and approved prior to delivery to the site.

The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. 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 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 their SPMDD.

If excavated silty clay, free of organics and deleterious materials, is to be used to build up the subgrade level for areas to be paved, it is recommended that the material be compacted by several passes of a suitably sized vibratory sheepfoot roller (i.e.- 5 to 6 passes and as deemed appropriate by Paterson personnel at the time of construction). Should the material consist of site-generated silty sand, the material should be compacted using a suitably sized smooth drum vibratory roller when considered for placement. All material should be placed in dry and above-freezing conditions. Frozen material may not be considered for this purpose. This process should be reviewed and approved by Paterson field personnel upon completion of each lift.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

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, stiff, brown silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

Conventional shallow footings placed on an undisturbed, compact silty sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

Where the silty sand bearing surface is found to be in a loose state of compactness, the area should be proof-rolled using a vibratory compactor and approved by the geotechnical consultant prior to the placement of footings. This process should be verified and approved by the geotechnical consultant at the tie of construction.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings designed using the bearing resistance values at SLS given above will be subjected to potential post construction 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 a stiff silty clay or silty sand 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 Restriction

It is recommended that a permissible grade raise restriction of **1.5 m** above the existing ground surface be considered for grading within 5 m of the proposed structure. A permissible grade raise restriction of **2.0 m** may be considered throughout areas located 5 m beyond the proposed structure. Proposed grading should be reviewed by the geotechnical consultant during the design phase of the proposed development.

If higher than permissible grade raises are required, preloading with or without 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 proposed site can be taken as seismic site response **Class D** as defined in Table 4.1.8.4.A of the Ontario Building Code 2012 (OBC 2012) for foundations considered at this site. The soils underlying the proposed shallow foundations are not considered susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious materials, within the footprint of the proposed building, the native soil, approved by Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab. The upper 200 mm of sub-slab fill should consist of 19 mm clear crushed stone.

All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

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 applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. 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 Design

Car only parking areas and access lanes are anticipated at this site. The proposed pavement structures are shown in Tables 2 and 3.

Table 2 - 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 fill	

Table 3 - Recommended Pavement Structure - Entrance and Access Lanes	
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
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 fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

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

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, noting that excessive vibration could lead to subgrade softening.

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.

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. These drains should extend in four orthogonal directions or longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines or the pipe, should be wrapped with suitable filter cloth. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines. Discharge of the subdrains should be directed by gravity to storm sewers or deeper drainage ditches.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded by a geosock on all sides and by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or be mechanically connected to a sump pump within the proposed structure.

Foundation Backfill - Basements Unequipped With Sump Pump Systems

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.

Foundation Backfill - Basements Equipped with Sump Pump Systems

Backfill against the exterior sides of the foundation walls should consist of a workable, brown silty clay extending a minimum of 1.5 m away from and along the perimeter of the foundations. The clay backfill must be implemented 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, are not recommended to be used for this purpose where sump pump systems are considered.

Sump Pump Systems - Additional Considerations

Service trenches for service lateral extending between private service alignments and the proposed structure should be provided with a clay seal. The clay seal should be installed in accordance with City of Ottawa Standard Detail Drawing S8 - Clay Seals for Pipe Trenches. The clay seal should extend a minimum of 300 mm above pipes within the service trench. The placement of clay seals should be reviewed and approved at the time of placement by the geotechnical consultant as part of site servicing reviews.

Reference should be made to the latest revision of the *Ottawa Design Guidelines - Sewer, Second Edition* dated 2012, and the latest revision to *Drawing P01 - Standard Sump Pump Configuration Greenfield Subdivisions with Clay Soils and Full Municipal Services* and the associated specifications.

6.2 Protection of Footings 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 soil and fill 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 excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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.

6.4 Pipe Bedding and Backfill

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

At least 300 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. 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. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 99% of the material's SPMDD.

Generally, it should be possible to re-use the moist, not wet, clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet 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.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that excavations extending below the sand-clay interface may result in initially high levels of sidewall infiltration considering the downward sloping ground surface throughout the subject site. Once the groundwater infiltration volumes stabilize to steady state, 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, 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 Slope Stability Assessment

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 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 favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is 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 than 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.

Subsoil conditions at the cross-sections were inferred based on the existing boreholes. The slope stability analysis was completed at each slope cross-section under worst-case scenario by assigning cohesive soils under fully saturated conditions. In addition, the silty clay deposit was noted to exit at the toe of slope and across the river.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation. The effective strength soil parameters used for static analysis are presented in Table 4 below.

Table 4 - Effective Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (KPa)
Topsoil	16	33	5
Silty Sand	20	35	0
Brown Silty Clay	17	33	5
Grey Silty Clay	16	33	10
Glacial Till	20	33	2

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strengths and based on our general knowledge of the areas geology. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 5 below.

The location of the two cross sections analyzed are presented on Drawing PG4197-1 - Test Hole Location Plan in Appendix 2.

Table 5 - Total Stress Soil and Material Parameters (Seismic Analysis)			
Soil Layer	**Unit Weight (kN/m^3)**	**Friction Angle (degrees)**	**Cohesion (KPa)**
Topsoil	**16**	**33**	**5**
Silty Sand	**20**	**35**	**0**
Brown Silty Clay	**17**	**-**	**120**
Grey Silty Clay	**16**	**-**	**70**
Glacial Till	**20**	**33**	**2**

Static Loading Analysis

The results of the static analysis at the subject site are shown in Figures 2A and 3A in Appendix 2. The minimum analyzed slope stability factor of safety under fully saturated conditions (worst case- scenario) were calculated to range between 1.58 and 1.63.

As a result, the two slope cross-sections analyzed exceeded the recommended Factor of Safety of 1.5 and are considered stable under static conditions.

Seismic Loading Analysis

An analysis considering seismic loading was also completed as part of our slope stability assessment. A horizontal seismic acceleration of 0.16g was considered for the analyzed section. A factor of safety of 1.1 is considered to be satisfactory for stability analysis including seismic loading.

The results of the analysis including seismic loading are shown in Figures 2B and 3B in Appendix 2. The overall slope stability factor of safety at the two slope cross-sections when considering seismic loading was found to be greater than 1.1 which is considered to be stable under seismic loading.

Stable Slope Allowance

The stable slope limit is usually defined by the extent of the lowest slip circle (failure slip plains) analyzed behind the top of slope where the minimum factor of safety calculated is less than 1.5. The minimum factor of safety was calculated for the two slope section analyzed to be above the recommended 1.5 under static conditions and above the recommended factor of safety of 1.1 under seismic loading. Therefore, the slope is defined as stable and no stable slope allowance is required from a geotechnical perspective.

Toe Erosion Allowance

The toe erosion allowance for the slope are based on the cohesive nature of the top layers of the subsoils, the observed current erosional activities, and the width and location of the current watercourse. Since the toe of slope located along the Ottawa River is protected from erosion by the use of shoreline protection measures, it is our opinion that no toe erosion allowance setback is required form a geotechnical perspective.

Erosion Access Allowance

Based on the City of Ottawa guidelines for slope stability, where the development precludes an access for construction equipment such as a parking lot, access lanes, etc, a 6 m erosion access allowance should be provided. Therefore, an erosion access allowance of 6 m has been recommended from the top of slope.

Limit of Hazard Lands

Based on the above, a setback taken from the top of the current slope has been provided based on the above-noted observations and analysis. Reference should be made to Drawing PG4197-1 - Test Hole Location Plan for the proposed Limit of Hazard Lands setback for development considerations at the subject site. The existing vegetation on the slope faces and shoreline protection along the bank of the Ottawa River should not be removed as it contributes to the stability of the slope and mitigates the potential for impacts to the slopes stability by the means of toe erosion.

6.8 Landscaping Considerations

Paterson completed a soils review of the laboratory testing of soils smaples by others for 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 in proximity to building foundations. Atterberg limits testing and Sieve analysis testing was completed by others and are presented in Appendix 1.

Based on these results, the modified plasticity index generally exceeds 40% in these areas. The following tree planting setbacks are recommended for the subject site. Large trees (mature height over 14 m) can be planted within these areas provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits are 7.5 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 following conditions are met:

- ❑ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade 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.

- ❑ A small tree must be provided with a minimum of 25 m^3 of available soils 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), as noted on the subdivision Grading Plan.

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.

- ❑ Review of the grading plan from a geotechnical perspective.
- ❑ 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.

All excess soil 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 made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

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 Mr. Ned Lathrop or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.



Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ❑ Mr. Ned Lathrop (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

BOREHOLE LOGS BY OTHERS

LABORATORY TEST RESULTS BY OTHERS

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINT AND BEDDING:

SPACING	50mm	50 300mm	0.3m 1m	1m 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
Φ'	$^{-o}$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	$^{-o}$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = ($W_L - W_L$)	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = ($W - W_P$)/ I_P	i	1	HYDAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = ($W_L - W$) / 1_P	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERED SOIL	$e_{,max}$	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

McIntosh Perry Consulting Engineers Ltd
1-1329 Gardiners Rd
Kingston, Ontario K7P 0L8

RECORD OF BOREHOLE No 17-1

1 OF 1

METRIC

DATE 13/03/2017 | LOCATION Old Montreal Rd, Cummberland | ORIGINATED BY MG

ID CP-17-0077 | COORDINATE lat: 45.518563 , lon: -75.395127 | COMPILED BY MG

CLIENT Archdiocese Ottawa | DATUM Assumed | CHECKED BY NT

ELEVATION 103.68

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES (REC)	* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
103.7	**SNOW**										
103.7 / 0.0											
			1	SS	7		103				
	SAND, loose, wet, brown		2	SS	9		102				
			3	SS	5		101				
100.8 / 2.9			4	SS	2				o		
	CLAY, firm, wet, grey		5	SS	1		100		o		
99.6 / 4.1	**END OF BOREHOLE** *water in sand layer										

$+^{3}$, $\times^{3}$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

McIntosh Perry Consulting Engineers Ltd
1-1329 Gardiners Rd
Kingston, Ontario K7P 0L8

RECORD OF BOREHOLE No 17-2

1 OF 2

METRIC

DATE 10/03/2017
ID CP-17-0077
CLIENT Archdiocese Ottawa
ELEVATION 101.33

LOCATION Old Montreal Rd, Cummberland
COORDINATE lat: 45.519338 , lon: -75.395526
DATUM Assumed

ORIGINATED BY GR
COMPILED BY MG
CHECKED BY NT

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES (REC)	* GROUND WATER CONDITIONS
101.3	**SNOW**					
101.1 0.0	**TOPSOIL**					
0.2	**CLAY,** trace sand, trace silt, stiff, desiccated, moist, brown		1	SS	4	
99.5 1.8			2	SS	2	▼
	CLAY, firm, wet, grey		3	TW		
			4	SS	0	
95.7 5.6	Probable **CLAY**					

ELEVATION SCALE: 101, 100, 99, 98, 97, 96, 95, 94, 93, 92

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L
WATER CONTENT (%)
25 50 75

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

+ 5
+ 6
+ 10
+ 8

Continued Next Page

+ ^{3}, × ^{3}: Numbers refer to Sensitivity
○ $^{3\%}$ STRAIN AT FAILURE

McIntosh Perry Consulting Engineers Ltd
1-1329 Gardiners Rd
Kingston, Ontario K7P 0L8

RECORD OF BOREHOLE No 17-2

2 OF 2

METRIC

DATE 10/03/2017
ID CP-17-0077
CLIENT Archdiocese Ottawa
ELEVATION 101.33
LOCATION Old Montreal Rd, Cummberland
COORDINATE lat: 45.519338 , lon: -75.395526
DATUM Assumed
ORIGINATED BY GR
COMPILED BY MG
CHECKED BY NT

SOIL PROFILE			SAMPLES								
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES (REC)	* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Probable **CLAY** *(continued)*						91 / 90 / 89 / 88 / 87 / 86				
86.4 / 14.9	Probable **TILL**										
85.4 / 16.0	**END OF BOREHOLE** Refusal at probable bedrock										

$+^3$, $\times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

McIntosh Perry Consulting Engineers Ltd
1-1329 Gardiners Rd
Kingston, Ontario K7P 0L8

RECORD OF BOREHOLE No 17-3

1 OF 2

METRIC

DATE 10/03/2017
ID CP-17-0077
CLIENT Archdiocese Ottawa
ELEVATION 102.68
LOCATION Old Montreal Rd, Cummberland
COORDINATE lat: 45.518973 , lon: -75.394639
DATUM Assumed
ORIGINATED BY GR
COMPILED BY MG
CHECKED BY NT

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES (REC)	* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
102.7	**SNOW**										
102.5 0.2	**TOPSOIL**										
			1	SS	9	▼	102				
			2	SS	5		101				
			3	SS	6						0 95 (5)
	SAND, loose, wet, brown		4	SS	2		100				0 98 (2)
			5	SS	4		99				
			6	SS	2		98				
			7	SS	2						
96.9 5.8			8	SS	1		97		○		
							96				Vane Refusal - sand in augers
			9	SS	0		95		○		
	CLAY, firm, wet, grey										Vane Refusal - sand in augers
			10	SS	3		94		○		
							93				Vane Refusal - sand in augers

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

McIntosh Perry Consulting Engineers Ltd
1-1329 Gardiners Rd
Kingston, Ontario K7P 0L8

RECORD OF BOREHOLE No 17-3

2 OF 2

METRIC

DATE 10/03/2017
ID CP-17-0077
CLIENT Archdiocese Ottawa
ELEVATION 102.68
LOCATION Old Montreal Rd, Cummberland
COORDINATE lat: 45.518973 , lon: -75.394639
DATUM Assumed
ORIGINATED BY GR
COMPILED BY MG
CHECKED BY NT

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES (REC)	* GROUND WATER CONDITIONS	ELEVATION SCALE
92.6 / 10.1	Probable **CLAY**						92
							91
							90
							89
88.0 / 14.6	Probable **TILL**						88
							87
86.6 / 16.1	**END OF BOREHOLE** Refusal at probable bedrock						

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L
WATER CONTENT (%)
25 50 75

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity
$\circ^{3\%}$ STRAIN AT FAILURE

McIntosh Perry Consulting Engineers Ltd
1-1329 Gardiners Rd
Kingston, Ontario K7P 0L8

RECORD OF BOREHOLE No 17-4

1 OF 1

METRIC

DATE 13/03/2017
ID CP-17-0077
CLIENT Archdiocese Ottawa
ELEVATION 100.78
LOCATION Old Montreal Rd, Cummberland
COORDINATE lat: 45.519466 , lon: -75.394129
DATUM Assumed
ORIGINATED BY MG
COMPILED BY MG
CHECKED BY NT

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES (REC)	* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
100.8	**SNOW**										
100.6 / 0.2	**TOPSOIL**										
	CLAY, stiff, desiccated, moist, brown		1	SS	7		100				
			2	SS	8		99				
98.6 / 2.1			3	SS	0		98	8, 7			
	CLAY, firm, wet, grey										
97.6 / 3.2	**END OF BOREHOLE**										
	*no water observed in open borehole										

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: 20 40 60 80 100
WATER CONTENT (%): 25 50 75

+ ³, × ³: Numbers refer to Sensitivity
○ ³% STRAIN AT FAILURE

McIntosh Perry Consulting Engineers Ltd
1-1329 Gardiners Rd
Kingston, Ontario K7P 0L8

RECORD OF BOREHOLE No 17-5

1 OF 2

METRIC

DATE 13/03/2017
ID CP-17-0077
CLIENT Archdiocese Ottawa
ELEVATION 103.3
LOCATION Old Montreal Rd, Cummberland
COORDINATE lat: 45.518673 , lon: -75.393816
DATUM Assumed
ORIGINATED BY MG
COMPILED BY MG
CHECKED BY NT

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES (REC)	* GROUND WATER CONDITIONS
103.3 / 0.0	**SNOW**					
103.1 / 0.2	**TOPSOIL**					
	SAND, compact, wet, brown		1	SS	10	
102.1 / 1.2			2	SS	0	
	CLAY, firm, wet, grey		3	SS	0	
			4	SS	0	
98.1 / 5.2	Probable **CLAY**					

ELEVATION SCALE: 103, 102, 101, 100, 99, 98, 97, 96, 95, 94

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L
WATER CONTENT (%)
25 50 75

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

McIntosh Perry Consulting Engineers Ltd
1-1329 Gardiners Rd
Kingston, Ontario K7P 0L8

RECORD OF BOREHOLE No 17-5

2 OF 2

METRIC

DATE 13/03/2017
ID CP-17-0077
CLIENT Archdiocese Ottawa
ELEVATION 103.3
LOCATION Old Montreal Rd, Cummberland
COORDINATE lat: 45.518673 , lon: -75.393816
DATUM Assumed
ORIGINATED BY MG
COMPILED BY MG
CHECKED BY NT

SOIL PROFILE			SAMPLES								
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES (REC)	* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Probable **CLAY** *(continued)*						93 92 91 90 89 88 87 86				
87.8 / 15.5	Probable **TILL**										
85.9 / 17.4	**END OF BOREHOLE** *no water observed in open borehole										

$+^3$, $\times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Borehole Records

Site Investigation: April 21st, 2017

Logged By: Mary-Ellen Gleeson

TP1:

0 – 255	Sandy Topsoil, trace organics
255 – 560	Sand, brown
560	Clay, stiff, desiccated, wet

Water infiltrating test pit

TP2:

0 – 255	Sandy Topsoil, trace organics
255 – 1000	Sand, brown

Water infiltrating test pit

TP3:

0 – 455	Sandy Topsoil, trace organics
455 – 585	Sand, brown
585	Clay, stiff, desiccated, wet

Water infiltrating test pit

TP4:

0 – 380	Sandy Topsoil, trace organics
380 – 610	Sand, brown
610	Clay, stiff, desiccated, wet

Water infiltrating test pit

TP5:

0 – 305	Sandy Topsoil, trace organics
305 – 990	Sand, brown
990	Clay, firm, wet, grey blue

Water infiltrating test pit

TP6:

0 – 75	Topsoil, trace organics
75	Clay, stiff, desiccated

TP7:

0 – 75	Topsoil, trace organics
75	Clay, stiff, desiccated

TP8:

0 – 75	Topsoil, trace organics
75	Clay, stiff, desiccated

TP9:

0 – 240	Topsoil, trace organics
240 – 430	Sand, brown
430	Clay, stiff, desiccated

Water infiltrating test pit

TP2:

0 – 230	Sandy Topsoil, trace organics
230 – 1000	Sand, brown

Water infiltrating test pit

Note:
All measurements are in millimeter indicating the depth below existing ground surface.

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

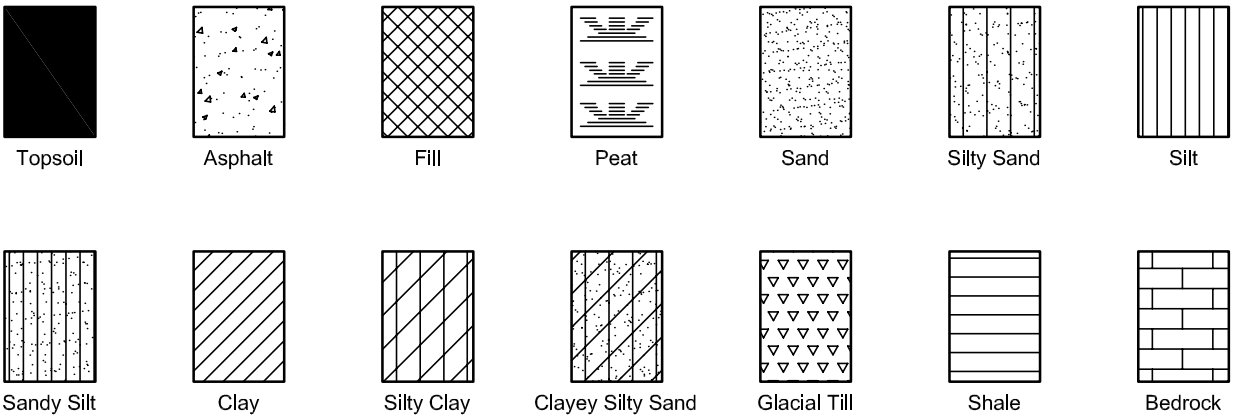
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

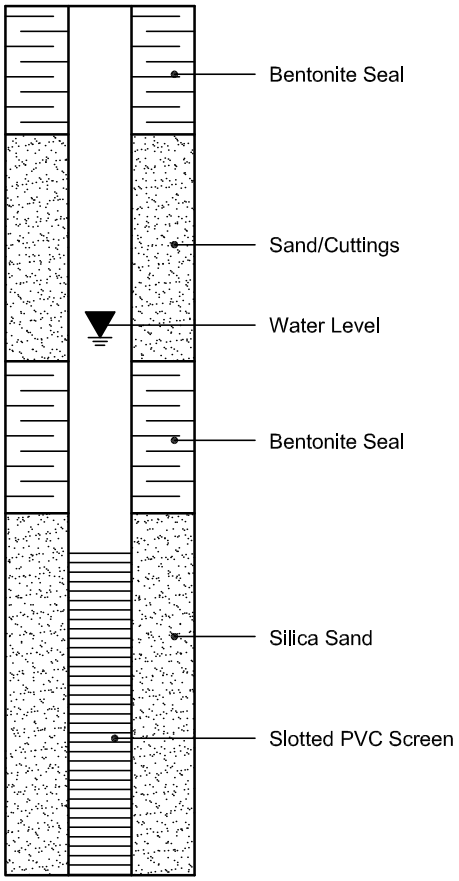
SYMBOLS AND TERMS (continued)

STRATA PLOT

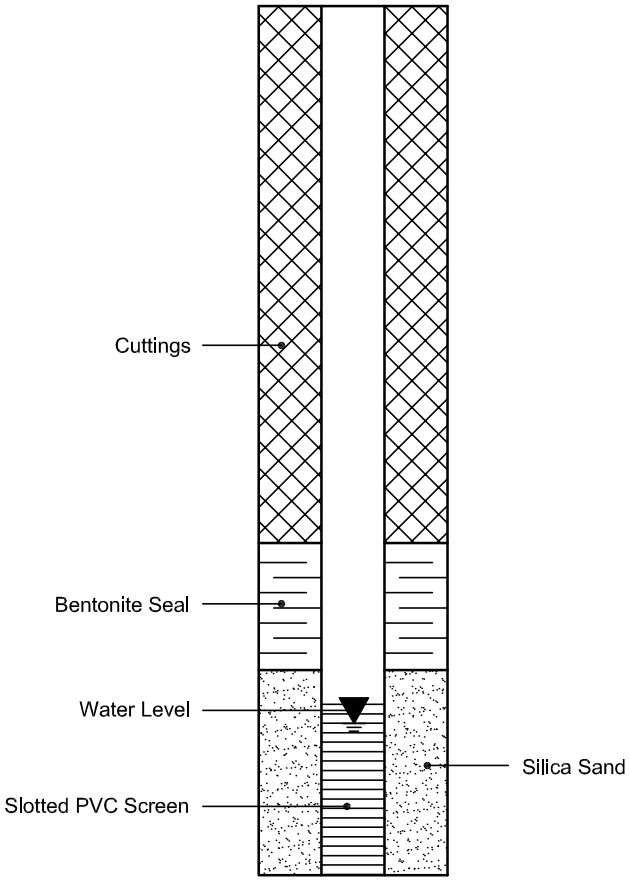


MONITORING WELL AND PIEZOMETER CONSTRUCTION

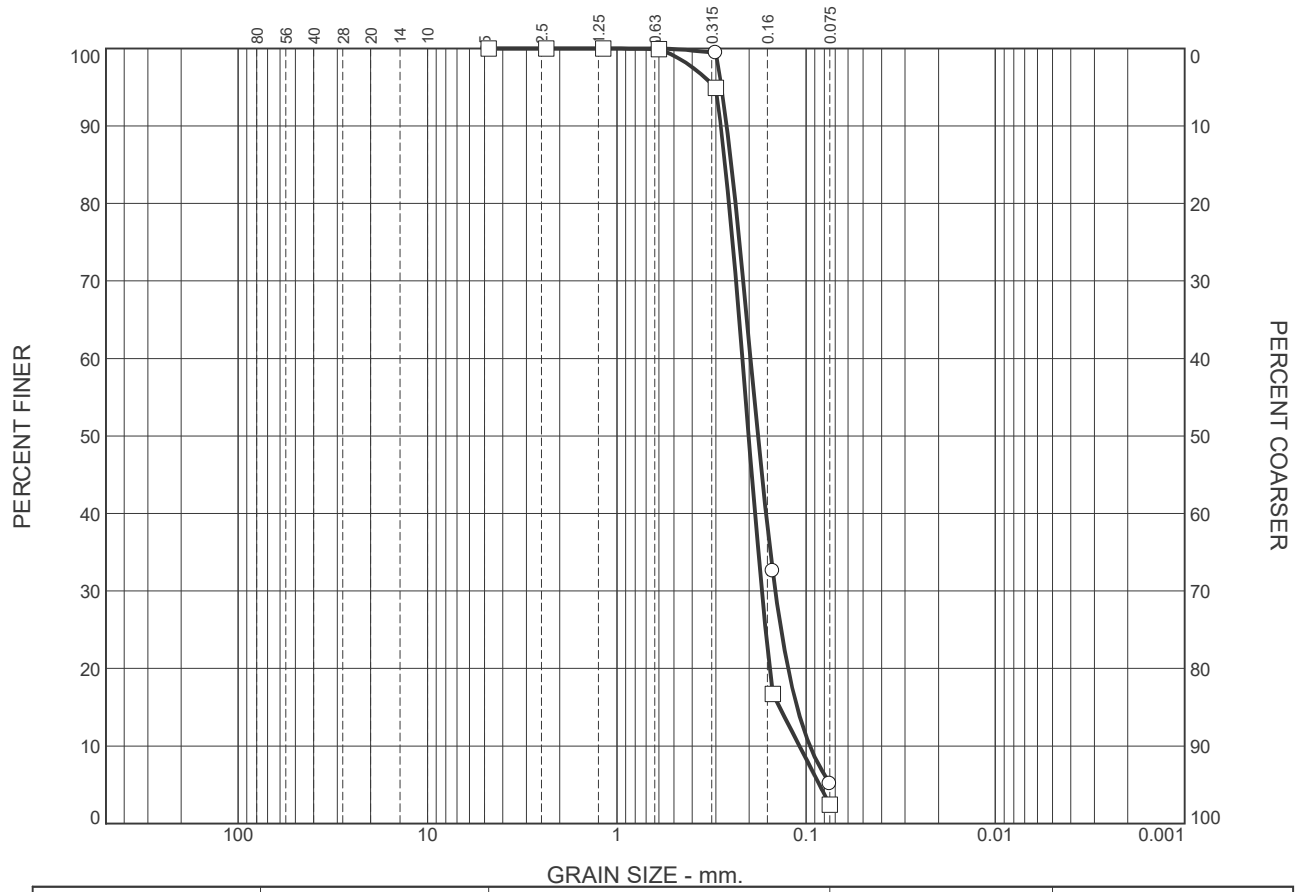
MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Particle Size Distribution Report



	% +3"	% Gravel	% Sand	% Silt	% Clay
○	0.0	0.0	94.9	5.1	
□	0.0	0.0	97.6	2.4	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (m.)	Material Description	USCS
○		BH17-3SS1		Sand trace Silt	
□		BH17-3SS4		Sand trace Silt	SP

AME Materials Engineering

Client: McIntosh Perry

Project: Archdioceses - Old Montreal Rd.

Project No.: 60182.005 (CP-17-0077) **Figure**

Checked By: H.Smith

GRAIN SIZE DISTRIBUTION TEST DATA

03/28/2017

Client: McIntosh Perry
Project: Archdioceses - Old Montreal Rd.
Project Number: 60182.005 (CP-17-0077)
Sample Number: BH17-3SS1
Material Description: Sand trace Silt
Checked by: H.Smith

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer	Percent Retained
177.10	0.00	0.00	4.75 mm	0.00	100.0	0.0
			2.36 mm	0.00	100.0	0.0
			1.18 mm	0.00	100.0	0.0
			.600 mm	0.00	100.0	0.0
			.300 mm	1.00	99.4	0.6
			.150 mm	119.40	32.6	67.4
			.075 mm	168.00	5.1	94.9

Fractional Components

Cobbles	Gravel	Sand	Silt	Clay
0.0	0.0	94.9		

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
	0.0954	0.1117	0.1245	0.1453	0.1628	0.1794	0.1965	0.2365	0.2488	0.2629	0.2800

Fineness Modulus	C_u	C_c
0.68	2.06	1.13

GRAIN SIZE DISTRIBUTION TEST DATA

03/28/2017

Client: McIntosh Perry
Project: Archdioceses - Old Montreal Rd.
Project Number: 60182.005 (CP-17-0077)
Sample Number: BH17-3SS4
Material Description: Sand trace Silt
USCS: SP
Checked by: H.Smith

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer	Percent Retained
223.70	0.00	0.00	4.75 mm	0.00	100.0	0.0
			2.36 mm	0.00	100.0	0.0
			1.18 mm	0.00	100.0	0.0
			.600 mm	0.20	99.9	0.1
			.300 mm	11.40	94.9	5.1
			.150 mm	186.30	16.7	83.3
			.075 mm	218.30	2.4	97.6

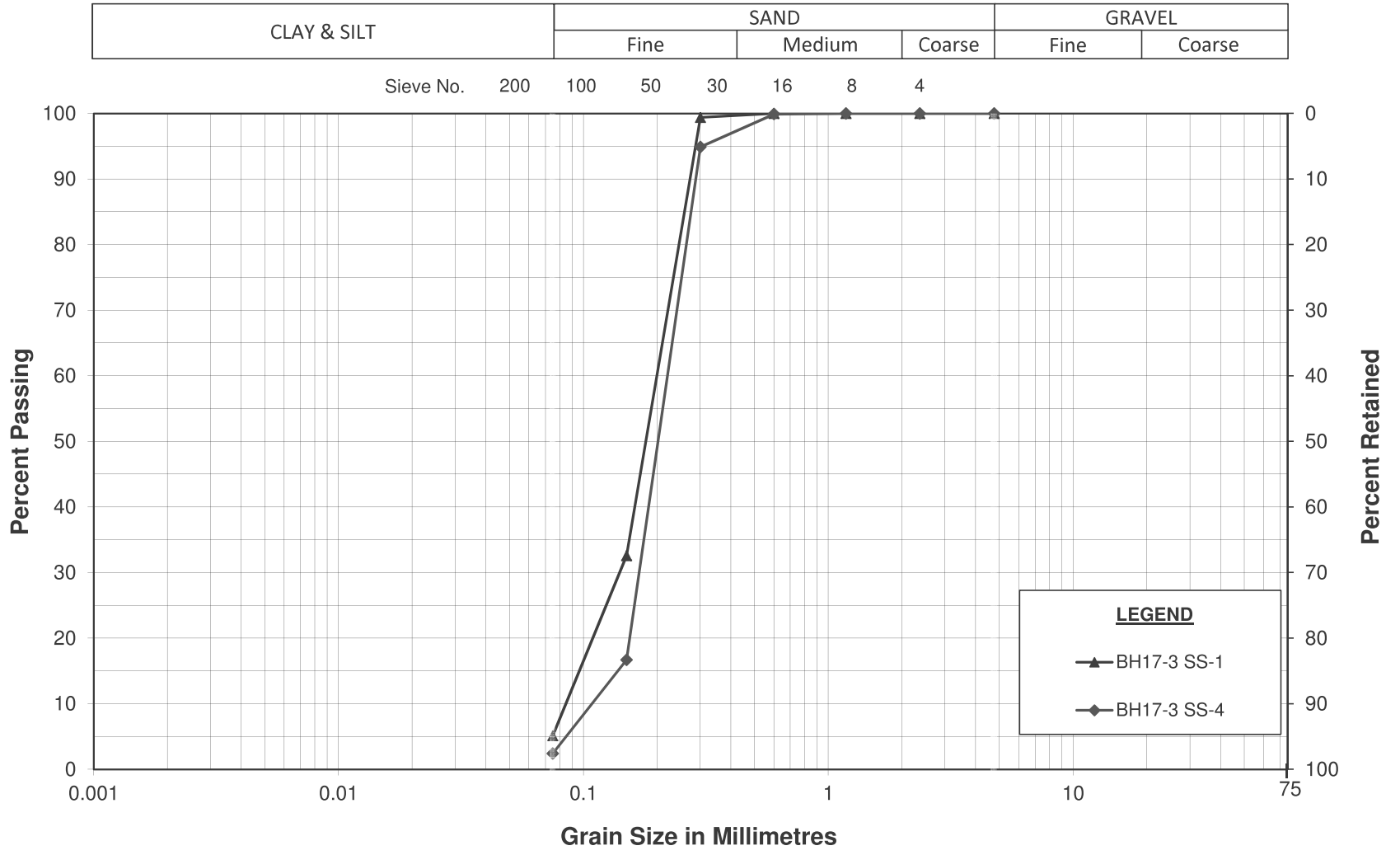
Fractional Components

Cobbles	Gravel	Sand	Silt	Clay
0.0	0.0	97.6		

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.0850	0.1083	0.1380	0.1556	0.1711	0.1860	0.2011	0.2172	0.2558	0.2681	0.2824	0.3027

Fineness Modulus	C_u	C_c
0.88	2.01	1.24

Unified Soil Classification System

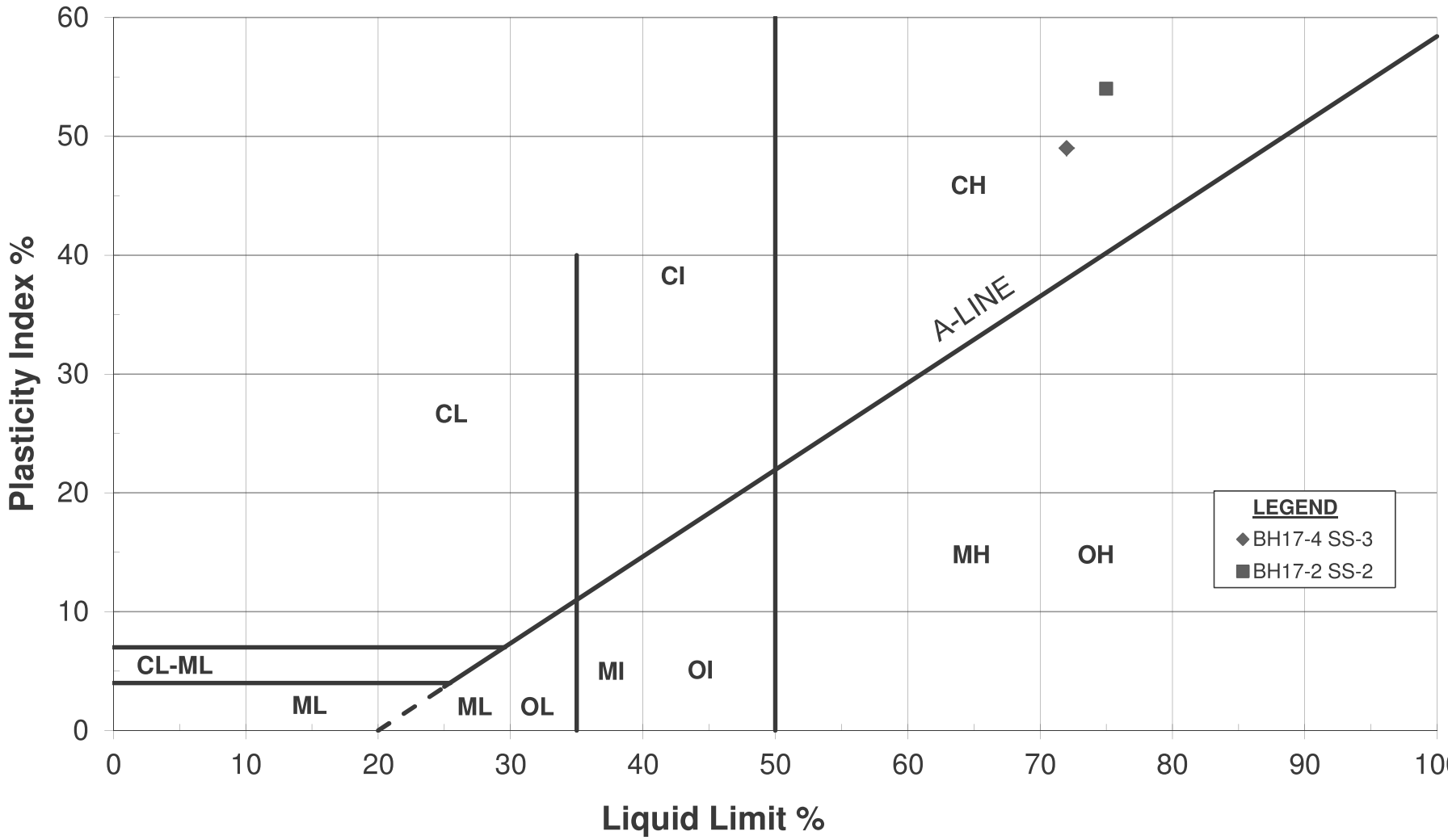


McINTOSH PERRY

Grain Size Distribution
Sand
Archdiocese - Old Montreal Rd

Figure No. 3

CP-17-0077



McINTOSH PERRY

PLASTICITY CHART

CLAY

Archdiocese - Old Montreal Rd

Figure No. 4

CP-17-0077

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SLOPE STABILITY CROSS SECTIONS

DRAWING PG4197 - TEST HOLE LOCATION PLAN

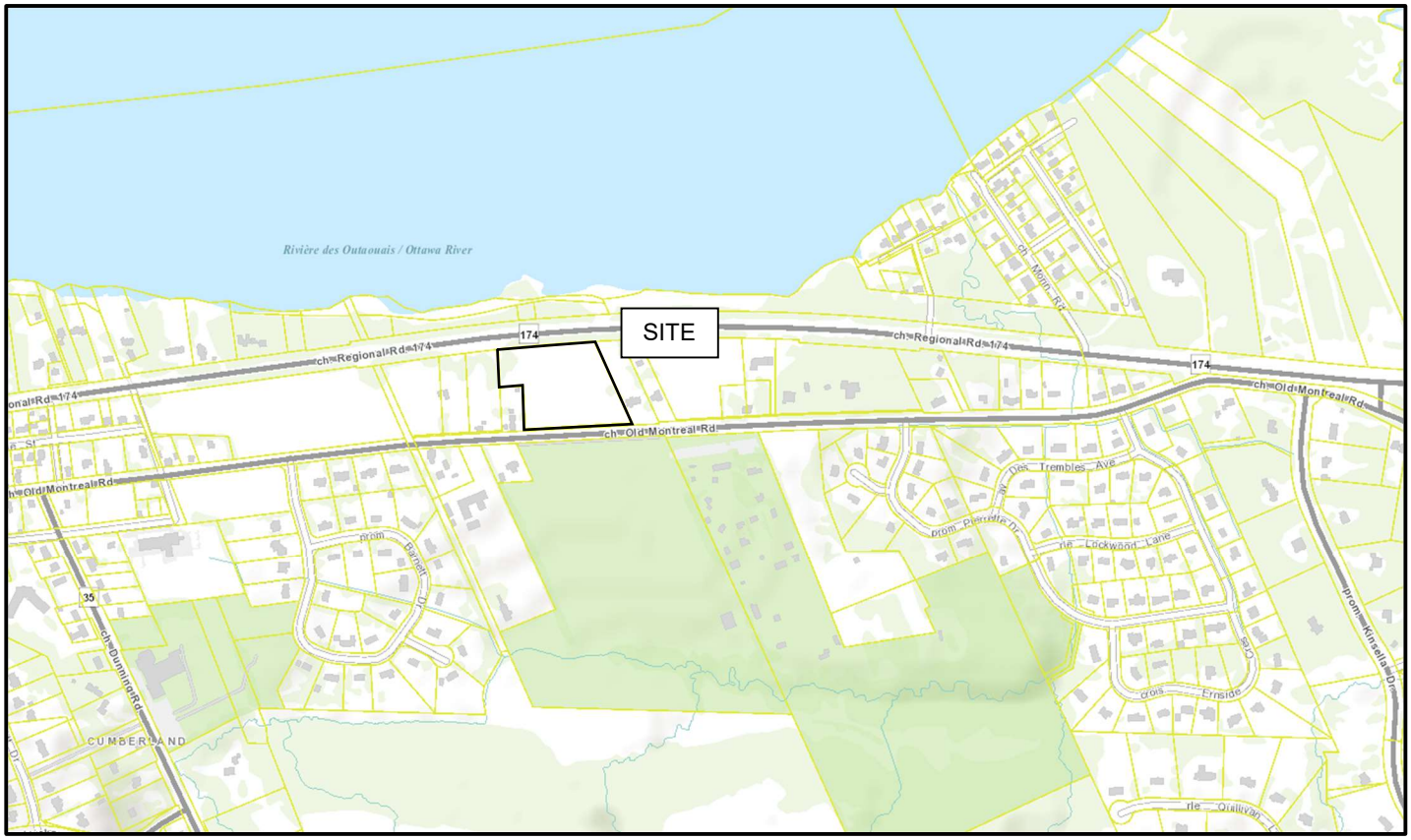
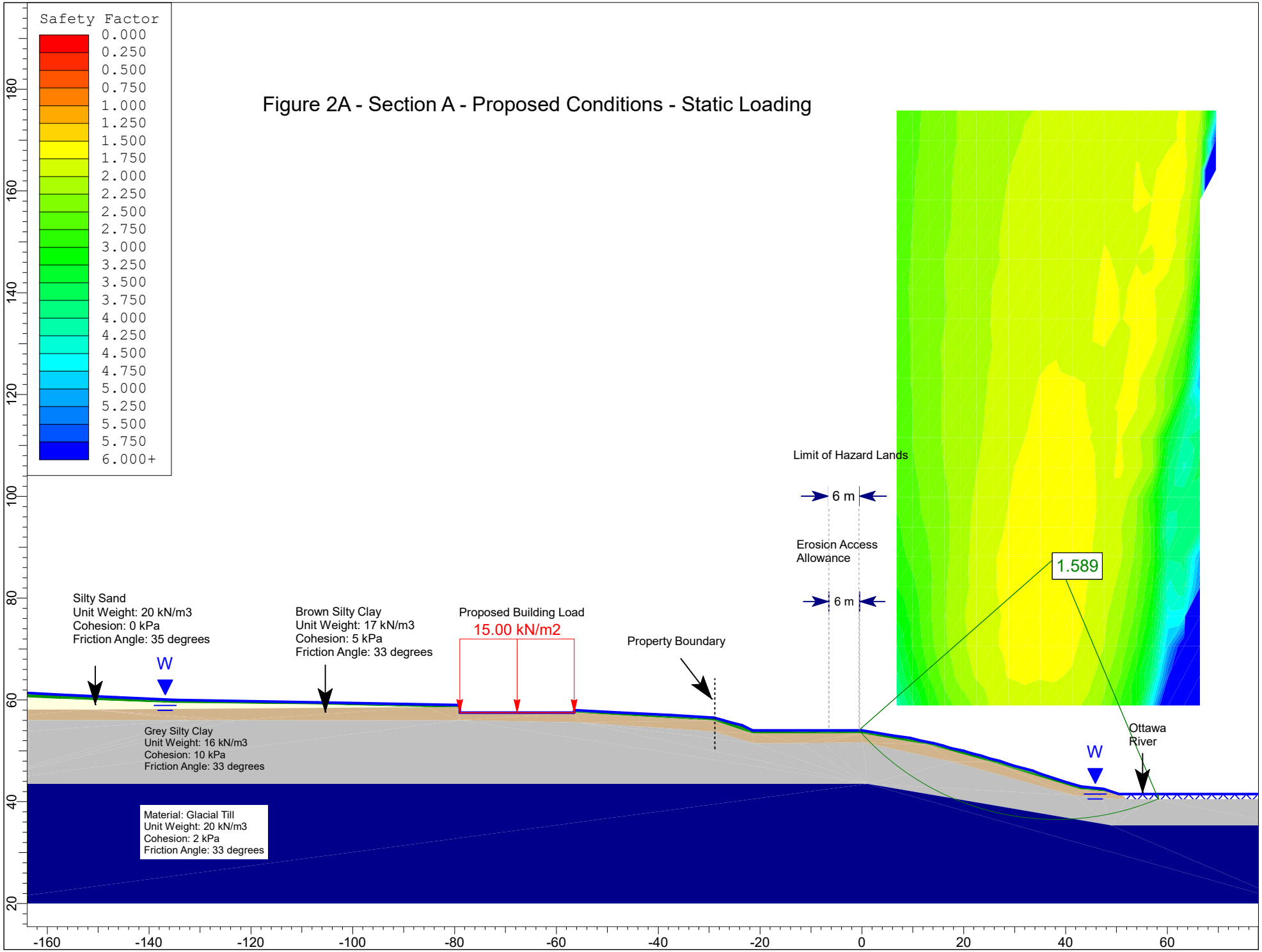
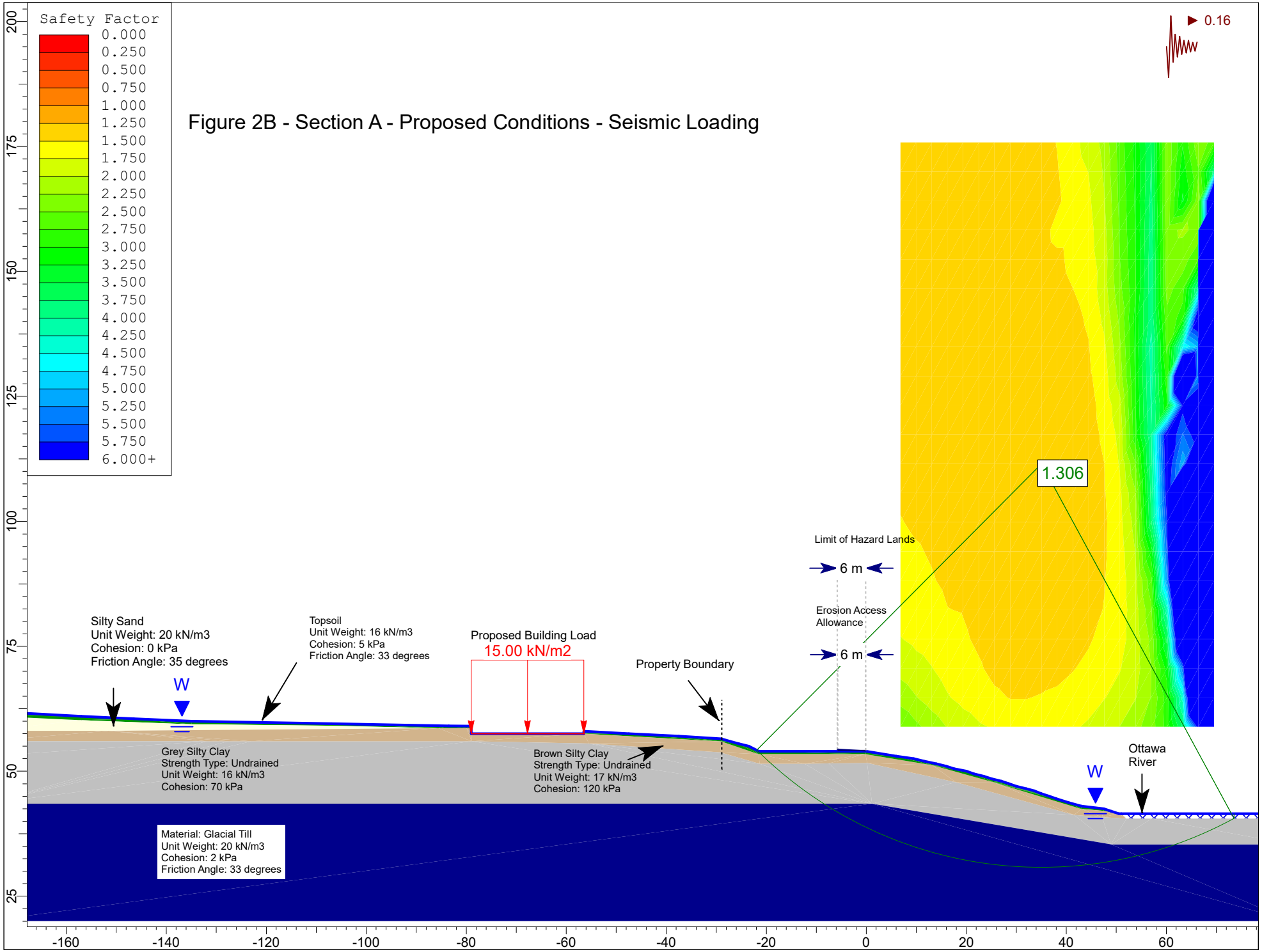
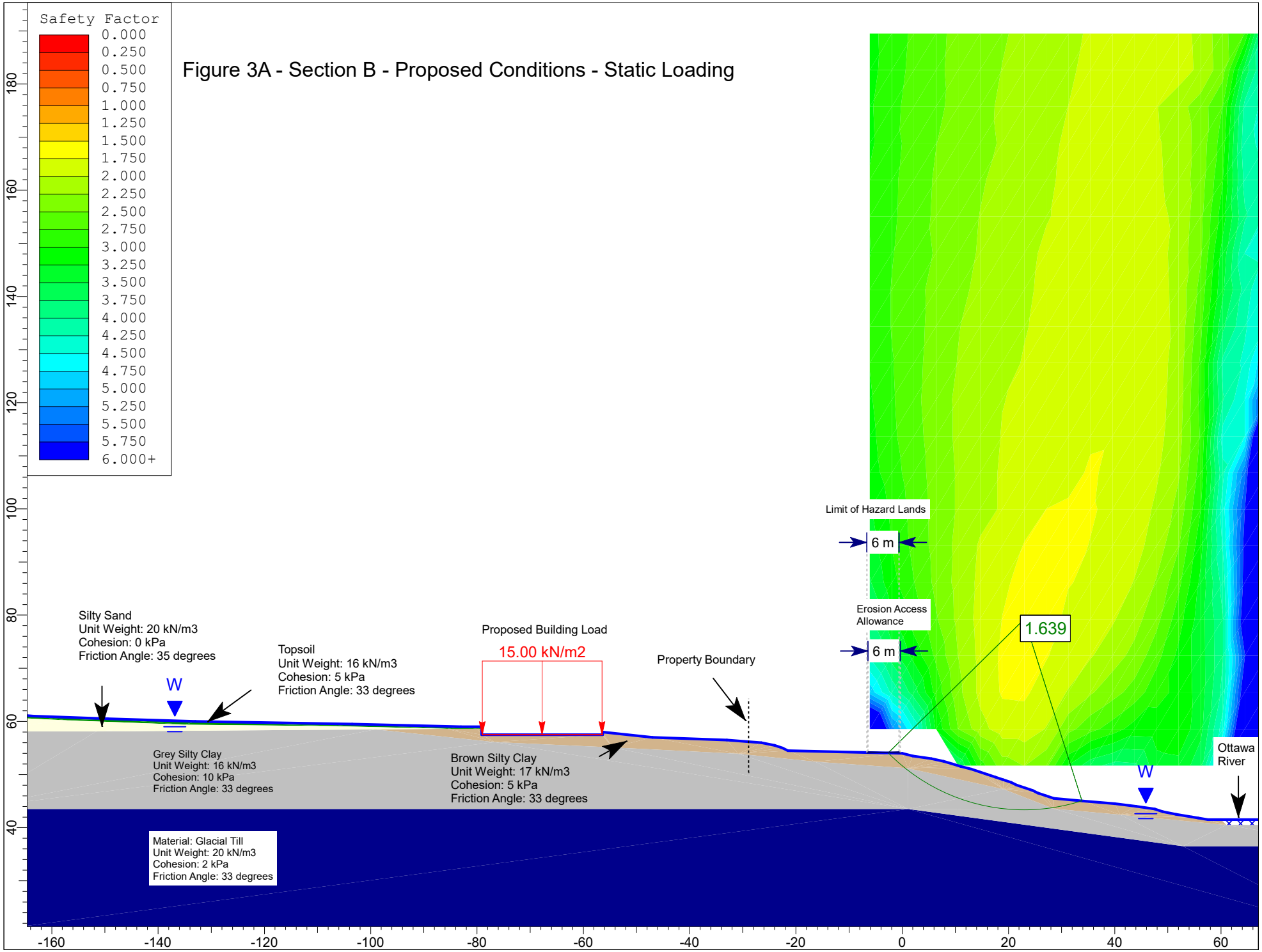


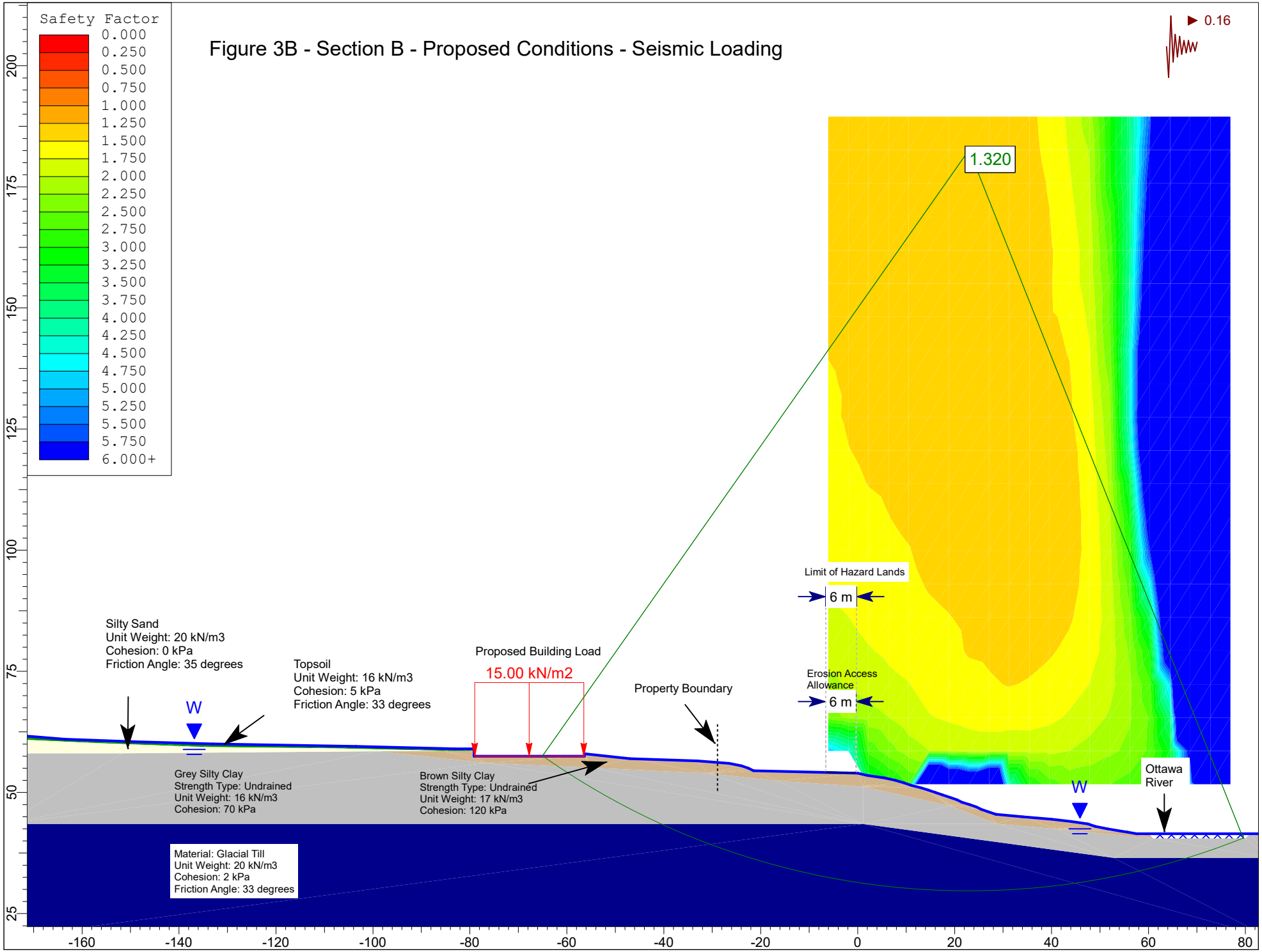
FIGURE 1

KEY PLAN


Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Figure 2A - Section A - Proposed Conditions - Static Loading
Limit of Hazard Lands
6 m
Erosion Access Allowance
6 m
1.589
Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 degrees
W
Brown Silty Clay
Unit Weight: 17 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Proposed Building Load
15.00 kN/m2
Property Boundary
Grey Silty Clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Material: Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 2 kPa
Friction Angle: 33 degrees
W
Ottawa River
180
160
140
120
100
80
60
40
20
-160
-140
-120
-100
-80
-60
-40
-20
0
20
40
60


Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
Figure 2B - Section A - Proposed Conditions - Seismic Loading
1.306
Limit of Hazard Lands
6 m
Erosion Access Allowance
6 m
Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 degrees
W
Topsoil
Unit Weight: 16 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Proposed Building Load
15.00 kN/m2
Property Boundary
Grey Silty Clay
Strength Type: Undrained
Unit Weight: 16 kN/m3
Cohesion: 70 kPa
Brown Silty Clay
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion: 120 kPa
W
Ottawa River
Material: Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 2 kPa
Friction Angle: 33 degrees
200
175
150
125
100
75
50
25
-160
-140
-120
-100
-80
-60
-40
-20
0
20
40
60


Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Figure 3A - Section B - Proposed Conditions - Static Loading
Limit of Hazard Lands
6 m
Erosion Access Allowance
6 m
1.639
Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 degrees
W
Topsoil
Unit Weight: 16 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Proposed Building Load
15.00 kN/m2
Property Boundary
Grey Silty Clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Brown Silty Clay
Unit Weight: 17 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
W
Ottawa River
Material: Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 2 kPa
Friction Angle: 33 degrees
180
160
140
120
100
80
60
40
-160
-140
-120
-100
-80
-60
-40
-20
0
20
40
60


Figure 3B - Section B - Proposed Conditions - Seismic Loading
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
1.320
Limit of Hazard Lands
6 m
Erosion Access Allowance
6 m
Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 degrees
Topsoil
Unit Weight: 16 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Proposed Building Load
15.00 kN/m2
Property Boundary
W
W
Grey Silty Clay
Strength Type: Undrained
Unit Weight: 16 kN/m3
Cohesion: 70 kPa
Brown Silty Clay
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion: 120 kPa
Ottawa River
Material: Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 2 kPa
Friction Angle: 33 degrees
200
175
150
125
100
75
50
25
-160
-140
-120
-100
-80
-60
-40
-20
0
20
40
60
80



LEGEND:

- BOREHOLE LOCATION BY OTHERS (MCINTOSH PERRY)
- TEST PIT LOCATION BY OTHERS (MCINTOSH PERRY)
- 102.70 GROUND SURFACE ELEVATION (m)
- (86.60) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
- SLOPE STABILITY CROSS SECTION LOCATION

TOPOGRAPHIC ELEVATIONS ARE PROVIDED BY CITY OF OTTAWA (CAD 2000 OPEN DATA)

TBM - HYDRO POLE LOCATED SOUTH OF REGIONAL ROAD 174. ASSUMED ELEVATION = 100.00 m

SCALE: 1:1500

0 25 50 75 100m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
2	TOP OF SLOPE & LIMIT OF HAZARD LANDS ADDED TO PLAN	01/26/2022	FC
1	CONCEPTUAL PLAN UPDATED TO DRAWING	01/18/2022	FC

MR. NED LATHROP
GEOTECHNICAL INVESTIGATION
2835 OLD MONTREAL ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:1500	Date: 11/2021
Drawn by: RCG	Report No.: PG4197-1
Checked by: FC	Dwg. No.: PG4197-1
Approved by: DJG	Revision No.: 2

p:\autocad drawings\geotechnical\pg41xx\pg4197\pg4197-1-thlp rev.02.dwg