Geotechnical Investigation

Proposed Multi-Storey Building

71 Russell Avenue
Ottawa, Ontario

Prepared for Jersey Developments Inc.

Report PG7696-1 Revision 1 dated Jan. 27, 2026

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Jersey Developments Inc. to prepare a Geotechnical Investigation Report for the proposed multi-storey building to be located at 71 Russell Avenue in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 of this report for the general site location).

This report summarizes subsurface and groundwater information from prior investigations completed by others, incorporates Paterson's review of this information, and presents geotechnical recommendations for the design and construction of the proposed development, including relevant construction considerations.

This report has been prepared specifically and solely for the aforementioned project which is described herein.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a multi-storey residential building with a lowest level slab at an approximate geodetic elevation of 60 m. The lower 2 floors will extend below-grade along Russell Avenue, but will daylight to the south due to the sloping grades across the site.

Access lanes, at-grade parking, and landscaped areas are also anticipated surrounding the proposed building. It is further understood that the proposed building will be municipally serviced.

3.0 Available Geotechnical Information from Others

3.1 Field Investigation

A previous geotechnical investigation was carried out at the subject site by others on May 4 and June 20, 2018, and consisted of a total of 3 boreholes advanced to a maximum depth of 8.9 m below the existing surface. The borehole locations are shown on Drawing PG7696-1 – Test Hole Location Plan, included in Appendix 2.

Reference should be made to the Log of Borehole sheets, prepared by others, for specific details of the subsurface profiles encountered at the borehole locations, which are presented in Appendix 1.

Groundwater

A piezometer was installed in borehole BH 18-1 by others, in order to monitor the groundwater level after the completion of the investigation. The measured groundwater level is presented in Appendix 1 and further discussed in Section 4.3.

3.2 Field Survey

The ground surface elevations at the borehole locations were surveyed by others and referenced to a geodetic datum. The locations of the boreholes and ground surface elevation at each borehole location are presented on the Borehole Location Plan and Log of Borehole sheets, respectively.

3.3 Laboratory Testing

Two (2) soil samples were tested by others for grain size distribution analysis, and 2 additional soil samples were tested for their Atterberg limits. The results of this testing are provided in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted by others to assess the corrosivity of the soil at the subject site. The detailed results are provided in Appendix 1.

4.0 Site Conditions

4.1 Surface Conditions

Based on aerial photographs, the subject site is currently vacant and covered with trees, overgrown vegetation, and grass. It is understood that the subject site previously had a 3-storey detached residence located in the northwest corner of the site. The subject site slopes downwards from west to east, dropping about 6 to 7 m across the site.

The subject site is bordered to the north by a residential dwelling, to the east and south by three-storey residential buildings, and to the west by Russell Avenue.

4.2 Subsurface Profile

Overburden

From the prior geotechnical investigation program completed by others, the soil profile at the western portion of the site consists of topsoil underlain by fill extending to approximate depths of 1.2 to 1.5 m below the existing ground surface. The fill was generally described as brown silty sand with gravel, trace to some clay, and trace organics. A stiff to very stiff, brown silty clay was encountered underlying the fill, extending to the glacial till deposit at depths of about 5.2 to 6.7 m.

On the eastern portion of the site, which is lower than the western portion, the glacial till deposit was encountered immediately underlying the topsoil. The glacial till across the site was generally observed to consist of loose to compact, clayey silt to silty sand with gravel.

Bedrock

Practical refusal to the auger or split spoon sampler was encountered at depths of 7.0 m, 8.9 m, and 2.4 m in boreholes BH18-1, BH18-2, and BH 18-3, respectively, which correspond to approximate geodetic elevations of 57.7 m, 58.2 m, and 57.3 m. The bedrock was cored at the location of borehole BH18-3 from depths of 2.4 m to 4.7 m below the existing ground surface, and was observed to consist of limestone with shale partings. The calculated RQD value ranged from 0 to 95%, which is indicative of a very poor to excellent quality bedrock.

Reference should be made to the Log of Borehole sheets in Appendix 1 for the details of the subsurface profile encountered at each borehole location.

Based on available geological mapping, the bedrock in this area consists of interbedded limestone and shale of the Verulam formation.

Grain Size Distribution Testing by Others

As part of the geotechnical investigation by others, grain size distribution testing was completed on 2 selected soil samples. The results of the grain size analysis are summarized in Table 1 below, and are presented on the Grain Size Distribution Testing Results sheets in Appendix 1.

Table 1 - Grain Size Distribution Testing

Test Hole	Sample	Gravel (%)	Sand (%)	Fine (%)
BH18-1	SS-1	3	77	20
BH18-1	SS-9	6	16	78

Atterberg Limit Tests by Others

As part of the geotechnical investigation by others, 2 selected silt/clay samples were tested for Atterberg Limit testing. The results are summarized in Table 2 below and presented in Appendix 1.

Table 2 - Summary of Atterberg Limits Test Results

Test Hole	Sample No.	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
BH18-1	SS7	27	18	9
BH18-2	SS5	46	20	26

4.3 Groundwater

As part of the geotechnical investigation by others, a standpipe piezometer was installed in borehole BH18-1. The water level was measured 18 days after drilling (May 2018) at a depth of 5.4 m below the existing grade (approx. geodetic elevation 59.3 m).

However, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, groundwater levels could differ 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 is recommended to be founded on conventional spread footings bearing on the undisturbed, compact glacial till bearing surface.

The contractor should be prepared for building debris, including foundation remnants, which may be encountered due to the building which was previously demolished at the site.

A temporary shoring system will be required along Russell Avenue and portions of the north and south sides of the site to allow for the completion of the building excavation.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, asphalt, and deleterious fill, such as those containing organic materials, should be stripped from under the proposed building and other settlement sensitive structures. Further, any loose glacial till encountered at the underside of footing (USF) elevation should be sub-excavated to the undisturbed, compact glacial till, and then reinstated up to USF with engineered fill.

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

Engineered Fill Placement

Engineered fill used for grading beneath the proposed building, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II, with a maximum aggregate size of 50 mm.

The fill should be tested and approved prior to delivery to the site. It 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 areas 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 thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD.

5.3 Foundation Design

Conventional Spread Footings

Footings founded on an undisturbed, compact glacial till 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 **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

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 placed on an undisturbed, compact glacial till bearing surface and designed using the bearing resistance values at SLS given above 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 the in-situ bearing medium soils 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.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class X_D**. If a higher seismic site class is required (such as Class X_C) for the proposed building, then a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024.

As noted above, any loose glacial till will be sub-excavated as part of the site preparation. Therefore, the soils underlying the proposed building foundation will not be susceptible to liquefaction. Reference should be made to the latest version of the OBC 2024 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 surface, approved by the Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab construction.

For basement areas, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. However, for any below-grade parking areas, the sub-slab soils should be as per Table 1 provided in Section 5.7. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill or a mud mat. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

An underfloor drainage system, consisting of a series of perforated drainage pipes connected to a positive outlet, should be provided in the clear crushed stone under the lower basement floor (discussed 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, 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 drained unit weight of 20 kN/m^3.

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

Static 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 soil, 0.5
γ = unit weight of the 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 $0.372 \cdot a_c \cdot \gamma \cdot H^2/g$, where:

a_c = $(1.45-a_{max}/g) \cdot a_{max}$
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 subject site is 0.372 g according to OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

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

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

The earth pressures calculated are unfactored. For the ULS case, the earth pressure loads should be factored as live loads, as per OBC 2024.

5.7 Pavement Design

For design purposes, the pavement structure presented in the following tables could be used for the design of the pavement structures at this site.

Table 1 – Recommended Pavement Structure – Parking Garage

Thickness (mm)	Material Description
125	Reinforced Concrete Slab
300	**BASE –** OPSS Granular A Crushed Stone
SUBGRADE – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, or engineered fill	

Table 2 – Recommended Pavement Structure –Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil, approved existing granular fill, or OPSS Granular B Type I or II material placed over in situ soil.	

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material’s SPMDD using suitable vibratory equipment.

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 building. The system should consist of a 100 mm diameter geotextile wrapped, perforated and corrugated plastic pipe surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit.

Underfloor Drainage

Underfloor drainage is recommended to control water infiltration for the lower basement area. For preliminary design purposes, we recommend that 100 mm diameter perforated PVC pipes be placed at approximate 6.0 m spacing under the lowest level floor slab. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

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 or an approved equivalent, 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. A waterproofing system should be provided to the elevator pits (pit bottom and walls).

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the building should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

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 of 1.5 m thick soil cover (or an equivalent thickness of soil cover and insulation) should be provided in this regard.

A minimum of 2.1 m thick soil cover (or an equivalent thickness of soil cover and insulation) should be provided for exterior unheated footings.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

Unsupported excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be Type 2 and 3 soils 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.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

It is understood that the subject site may require temporary shoring where there is insufficient room for an unsupported open excavation with appropriate side slopes, particularly on the north, south and west sides of the site.

The design and approval of the temporary shoring system will be the responsibility of the shoring contractor and the shoring designer, who is a licensed professional engineer and is hired by the shoring contractor.

It is the responsibility of the shoring contractor to ensure that the temporary shoring system is in compliance with safety requirements, designed to avoid any damage to adjacent structures, and includes dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition, which can occur during significant precipitation events.

The temporary shoring system may generally consist of a soldier pile and lagging system. However, where adjacent structures abut the excavation, a secant pile wall is recommended. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included in the earth pressures described below.

The earth pressures acting on the temporary shoring system may be calculated with the following parameters.

Table 3 – Soils Parameter for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_O)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

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

The hydrostatic groundwater pressure should be included in the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated as full weight, with no hydrostatic groundwater pressure component. For design purposes, a minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The bedding material should extend at least to the spring line of the pipe. The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

It should generally be possible to re-use the glacial till above the cover material if the excavation and filling operations are carried out in dry weather conditions, provided that all stones 300 mm or greater in their longest dimension are removed.

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

It is anticipated that groundwater infiltration into the excavations should be moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. Where excavation is to extend below the long-term groundwater level, higher infiltration rates are anticipated that may require dewatering wells.

Permit to Take Water

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three 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. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

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.

Impacts on Neighbouring Structures

It is understood that one underground basement level and a sub-basement level are planned for the proposed building. The proposed excavation is not expected to extend significantly below the groundwater level. Therefore, based on the proximity of neighbouring buildings and the existing groundwater level within the glacial till deposit, the proposed development is not anticipated to impact the neighbouring structures negatively.

6.6 Winter Construction

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

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, 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.

The trench excavations should be constructed in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost-susceptible soils, which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow, or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. 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%. These results are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the chloride content, pH, and resistivity indicate the presence of a non-aggressive to moderately aggressive environment for exposed ferrous metals at this site.

6.8 Slope Stability Assessment

As noted above, the subject site slopes downwards from west to east, dropping about 6 to 7 m across the site. Therefore, the slope stability was analyzed under proposed conditions.

The slope stability analysis of the proposed site conditions was carried out using SLIDE2, a computer program, which permits a two-dimensional slope stability analysis using several widely-accepted limit equilibrium methods, such as the Bishop's 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 1.0 is usually required to ascertain that the risks of failure are acceptable. Minimum factors of safety of 1.5 and 1.1 are generally recommended under static and seismic conditions, respectively, where the failure of the slope would endanger permanent structures.

Subsoil conditions at the cross-section were inferred based on the completed boreholes by others at the subject site, and our knowledge of the area. The soil parameters used for the analysis are shown on Figures 2A and 2B in Appendix 2. For conservative modelling of the groundwater conditions, the native subsoil was noted to be fully saturated for our analysis along the slope.

Static Loading Analysis

The result for the proposed static conditions is shown in Figure 2A in Appendix 2. The result indicates that the factor of safety for Section A-A' was found to be greater than 1.5 under static loading. Therefore, the slope is considered to be stable under static loading conditions.

Seismic Loading Analysis

The result for the proposed seismic conditions is shown in Figure 2B in Appendix 2. A horizontal acceleration of 0.186g was considered for the slope under seismic conditions.

The result indicates that the factor of safety for Section A-A' was found to be greater than 1.1 under seismic loading. Therefore, the slope is considered to be stable under seismic loading.

Conclusion

In summary, as the factors of safety for the slope stability analyses under static and seismic conditions exceed 1.5 and 1.1, respectively, the slope at the subject site is considered stable for the proposed development conditions.

7.0 Recommendations

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

☐ Review of the temporary shoring system design.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Sampling and testing of the concrete and granular 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 ensure that the specified level of compaction has been achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

☐ Review detailed temporary excavation and shoring drawings from a geotechnical perspective, once available.

☐ Review of the frost protection measures proposed for the loading areas, from a geotechnical perspective.

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.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per ***Ontario Regulation 406/19: On-Site and Excess Soil Management***.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test hole locations, we request 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 Jersey Developments Inc. 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.

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APPENDIX 1

TEST HOLE LOGS SHEETS BY OTHERS

GRAIN SIZE DISTRIBUTION TESTING RESULTS BY OTHERS

ATTERBERG LIMITS TESTING RESULTS BY OTHERS

ANALYTICAL TESTING RESULTS BY OTHERS

LOG OF BOREHOLE BH18-1

Project: 71 Russell Avenue
Client: Royal LePage Team Realty
Project Location: 71 Russell Ave, Ottawa,ON
Datum: Geodetic
BH Location: See borehole location plan N 5030403 E 447004

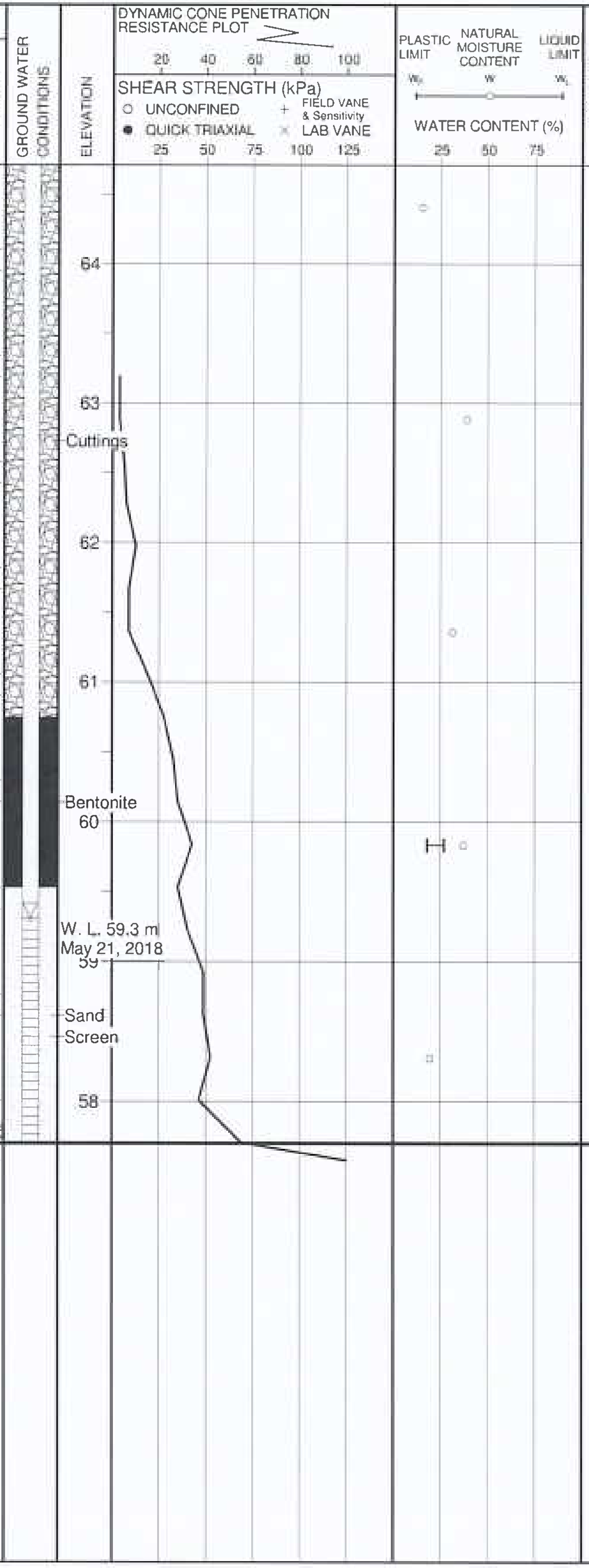
DRILLING DATA
Rig Type:CME 75
Method: Hollow Stem Augering
Borehole Diameter: 203
Core Diameter: n/a

Project No.: 181-05178-00
Date Started: 5/4/2018
Supervisor: DR/AA
Reviewer: DW

SOIL PROFILE		SAMPLES			REMARKS AND GRAIN SIZE DISTRIBUTION (%)			
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m	GR	SA	SI	CL
64.7								
64.6 0.1	**TOPSOIL** - 110 mm **SILTY SAND** some clay, trace gravel, trace organics, brown, loose (FILL)	1	SS	2	3	77	(20)	
		2	SS	3				
63.2 1.5	**SILTY CLAY** trace sand, trace gravel, brown grey, moist, stiff to very stiff	3	SS	3				
		4	SS	2				
		5	SS	5				
		6	SS	8				
		7	SS	3				
59.5 5.2	**CLAYEY SILT** some sand, trace gravel, grey, mosit, loose (GLACIAL TILL)	8	SS	4				
		9	SS	6	6	16	(78)	
		10	SS	>50/25 mm				
57.7 7.0	**END OF BOREHOLE**							

1) Auger refusal at 7.0 m below the existing surface elevation.
2) DCPT refusal at 7.1 m below the existing surface elevation.
3) Borehole was dry at the completion of augering.
4) 19 mm piezometer installed at 7.0 m below the existing ground surface.
5) Date Groundwater Depth

Date	Groundwater Depth
5/21/2018	5.4 m



GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Sheet No. 1 of 1

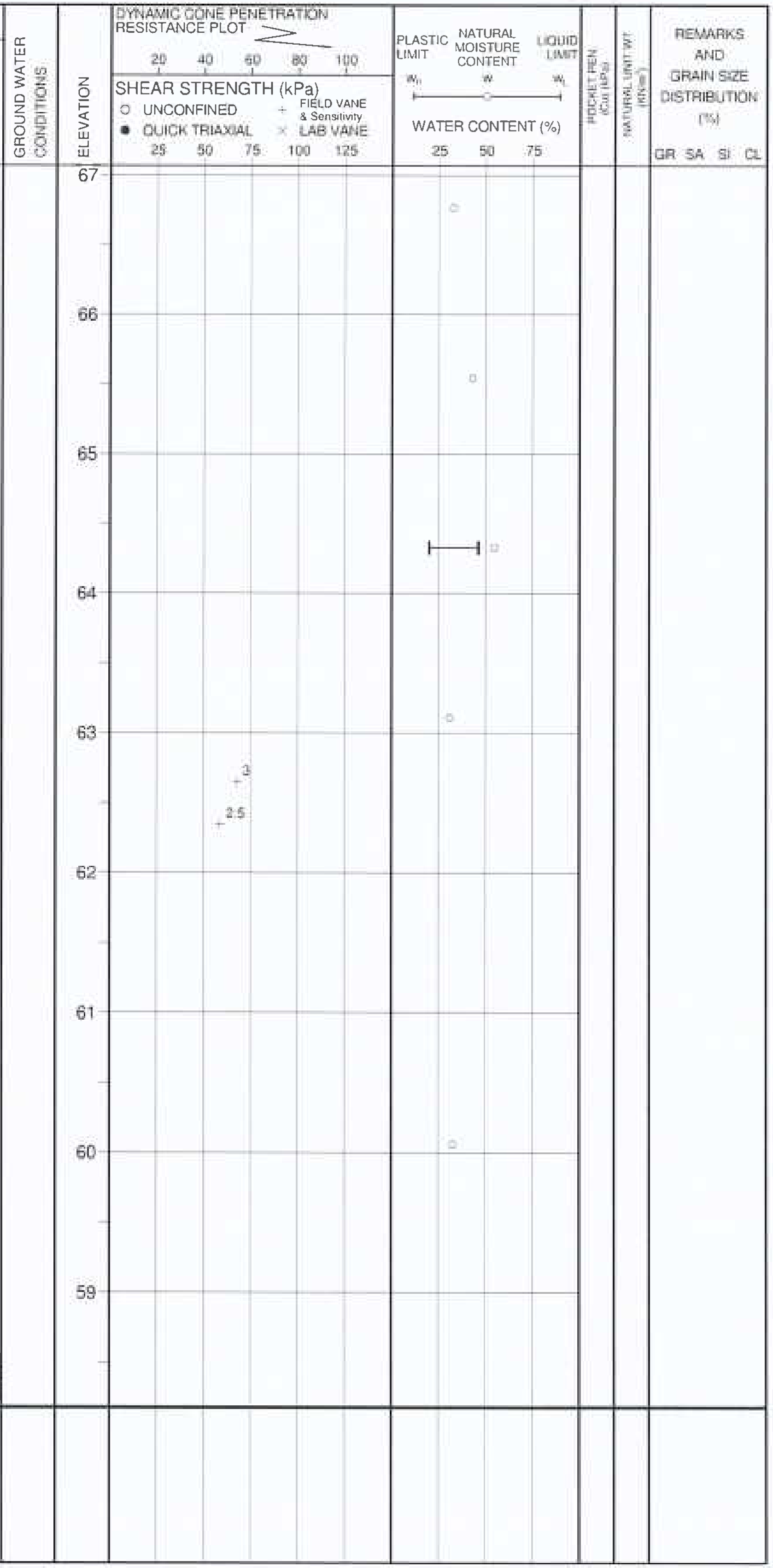
LOG OF BOREHOLE BH18-2

Project: 71 Russell Avenue
Client: Royal LePage Team Realty
Project Location: 71 Russell Ave, Ottawa,ON
Datum: Geodetic
BH Location: See borehole location plan N 5030398 E 447021

DRILLING DATA
Rig Type:n/a
Method: Tripod
Borehole Diameter: 89
Core Diameter: n/a

Project No.: 181-05178-00
Date Started: 5/4/2018
Supervisor: DR/AA
Reviewer: DW

SOIL PROFILE		SAMPLES		
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m
67.1				
67.0 0.1	TOPSOIL - 110 mm **SILTY SAND** some clay, trace gravel, trace organics, brown, loose to compact (FILL)	1	SS	2
		2	SS	11
65.9 1.2	**SILTY CLAY** trace sand, trace gravel, brown grey, moist, stiff to very stiff	3	SS	11
		4	SS	6
		5	SS	5
		6	SS	3
		7	SS	3
			VANE	
			VANE	
		8	TW	n/a
		9	SS	4
		10	SS	8
60.4 6.7	**CLAYEY SILT** some sand, trace gravel, grey, mosit, loose (GLACIAL TILL)	11	SS	4
59.8 7.3	**SILTY SAND AND GRAVEL** some clay, grey, mosit, loose to compact (GLACIAL TILL)	12	SS	4
		13	SS	12
		14	SS	>50/50 mm
58.2 8.9	**END OF BOREHOLE** 1) Sampler refusal at 8.9 m below the existing surface elevation. 2) Borehole was dry at the completion of sampling.			



GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES
$+^3$, $\times^3$: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

Sheet No. 1 of 1

LOG OF BOREHOLE BH18-3

Project: 71 Russell Avenue
Client: Royal LePage Team Realty
Project Location: 71 Russell Ave, Ottawa,ON
Datum: Geodetic
BH Location: See borehole location plan N 5030405 E 447034

DRILLING DATA
Rig Type:n/a
Method: Hand Portable
Borehole Diameter: 50
Core Diameter: HQ

Project No.: 181-05178-00
Date Started: 6/20/2018
Supervisor: DR/AA
Reviewer: DW

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION					
59.7												
59.6	**TOPSOIL - 150 mm**											
0.2	**SILTY SAND,** trace to some clay, brown, moist, loose (GLACIAL TILL)		1	SS	5							
			2	SS	7		59					
58.3 1.4	**SAND,** some silt, some clay, trace gravel, brown, wet, compact (GLACIAL TILL)		3	SS	12		58					
			4	SS	29							
57.3 2.4	**LIMESTONE** with shale partings, grey, fresh to slightly weathered Run 1: 2.4 m - 3.6 m TCR - 100% SCR - 17% RQD - 0%		5	CORE			57					
	Run 2: 3.6 m - 4.7 m TCR - 100% SCR - 95% RQD - 95%		6	CORE			56					
55.0 4.7	**END OF BOREHOLE** 1) Sampler refusal at 2.4 m below the existing surface elevation. Switch to NQ coring. 2) Coring terminated at 4.7 below the existing surface elevation. 3) Blow count corrected from third weight hammer						55					

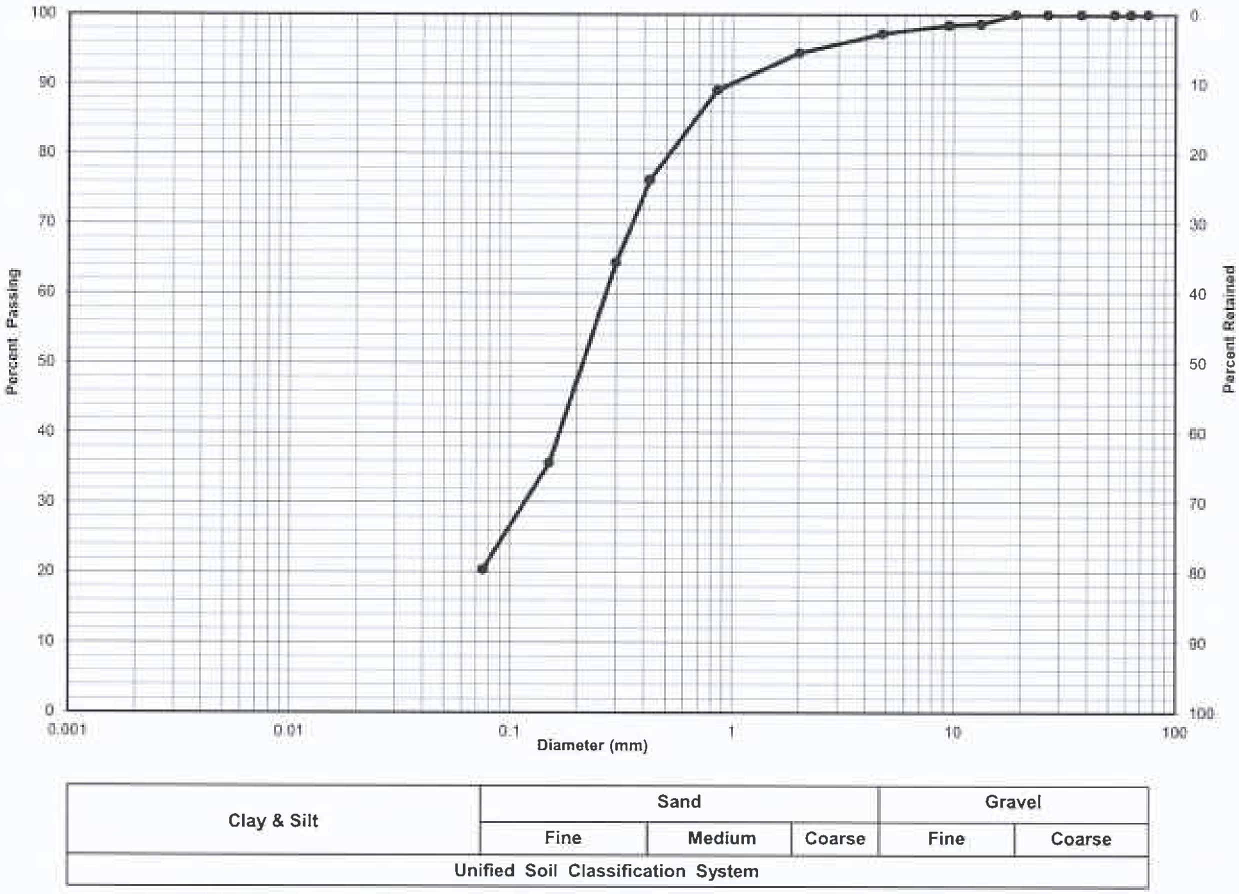
GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES +³, ×³: Numbers refer to Sensitivity ○ ε =3% Strain at Failure

Particle-Size Analysis of Soils
(ASTM D422)

Client:	RLTR	**Lab no.:**	OL 322-1
Project/Site:	71 Russell Ave, Ottawa	**Project no.:**	181-05178-00

Borehole no.:	18-1	Sample no.:	SS1
Depth:	0.00-0.60 m		



	Gravel	Sand	Clay & Silt	Silt	Clay
Percent %	2.7	76.9	20.4	-	-

Remarks:

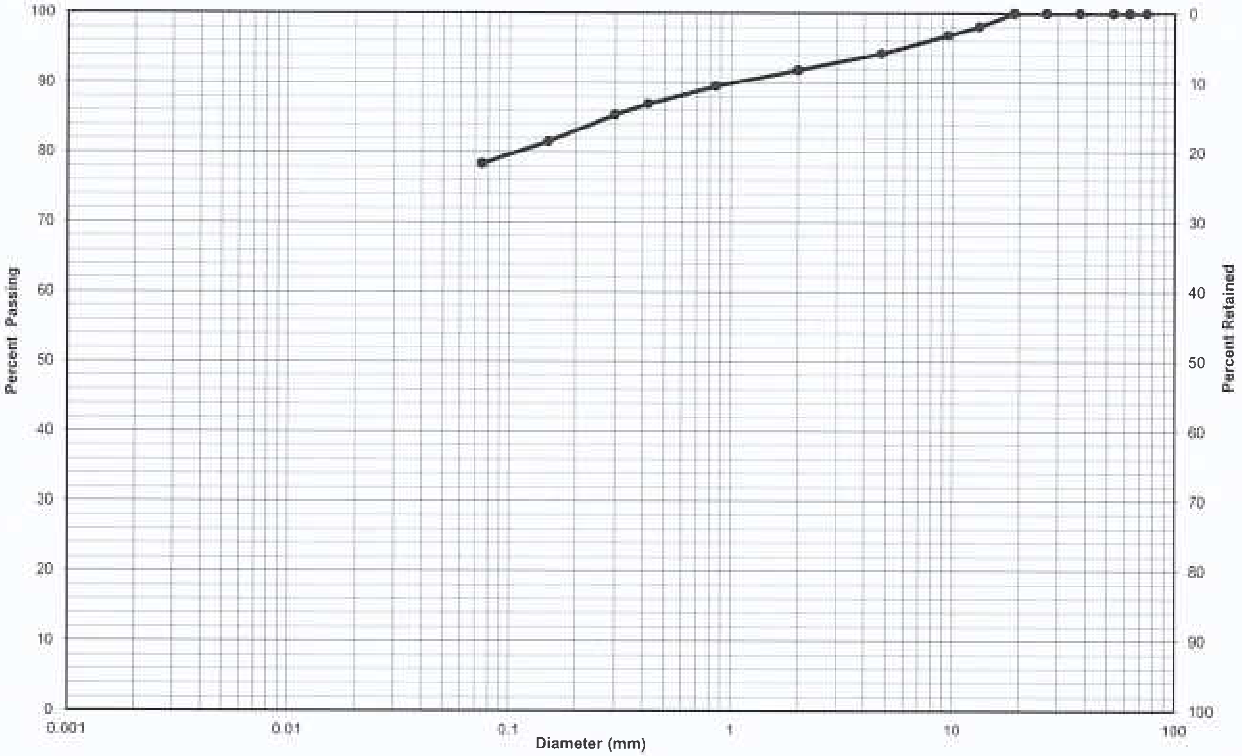
*one small piece of glass found on the 9.5 mm sieve

Performed by:	M. Tippett	**Date:**	May 29, 2018
Verified by:	N. Krebs	**Date:**	May 29, 2018

Particle-Size Analysis of Soils
(ASTM D422)

Client:	RLTR	**Lab no.:**	OL 322-3
Project/Site:	71 Russell Ave, Ottawa	**Project no.:**	181-05178-00

Borehole no.:	18-1	Sample no.:	SS9
Depth:	6.00-6.70 m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent %	Gravel	Sand	Clay & Silt	Silt	Clay
	5.8	15.9	78.3	-	-

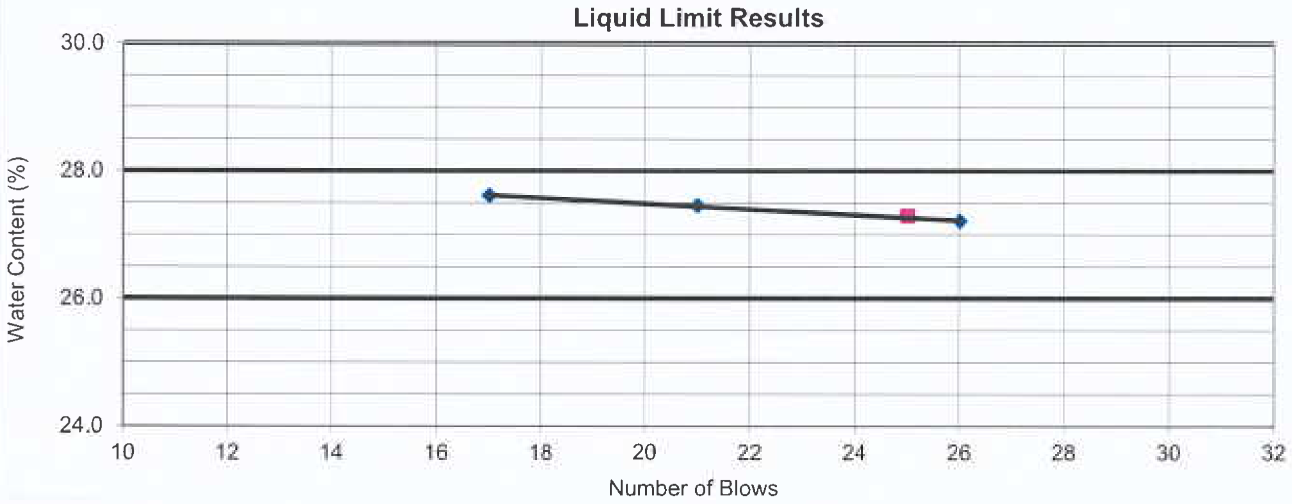
Remarks:

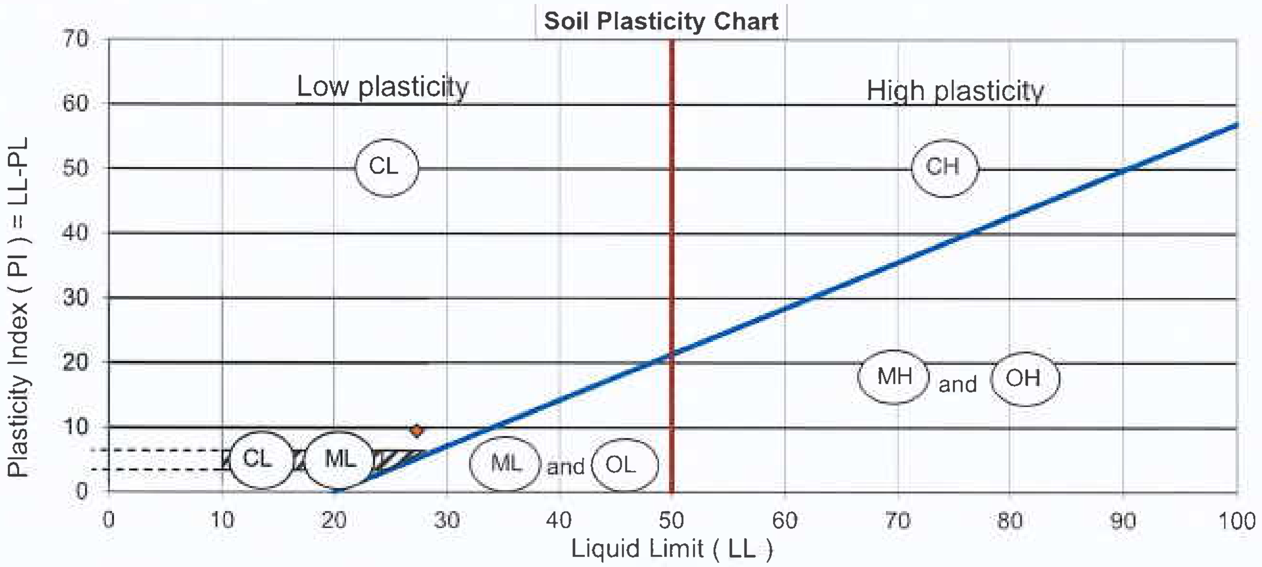
Performed by:	M. Tippett	**Date:**	May 29, 2018
Verified by:	N. Krebs	**Date:**	May 29, 2018

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client: RLTR
Lab No.: OL322-2
Project/Site: 71 Russell Ave.
Project No.: 181-05178-00

Borehole No.: BH18-1
Sample No.: SS7
Sample Depth: 4.6-5.2m





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
27	18	9	37.3

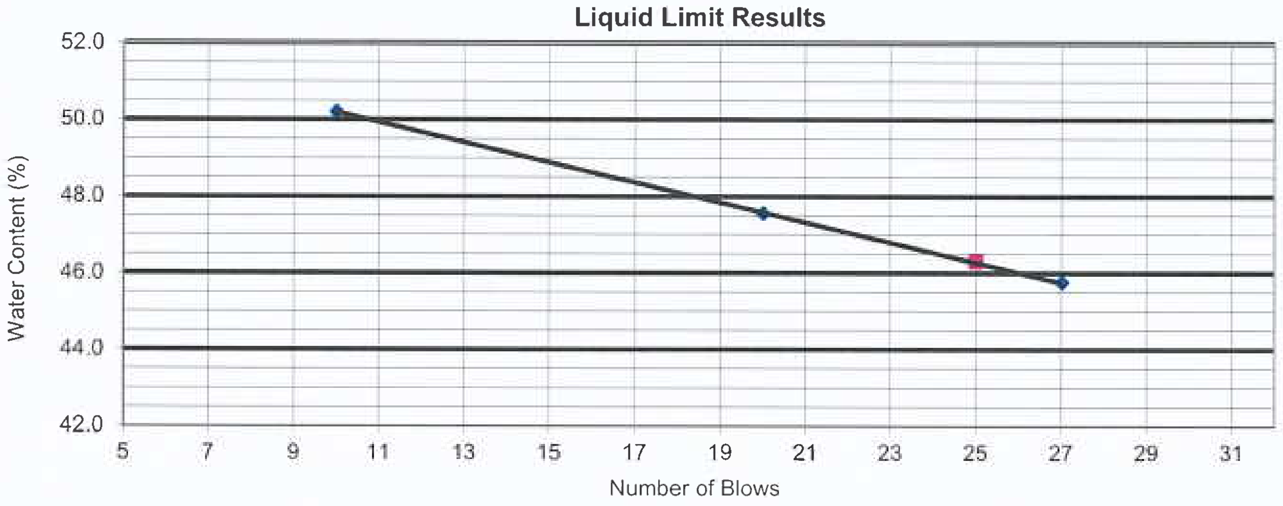
Sample Description: **CL - Low plasticity, inorganic clay**

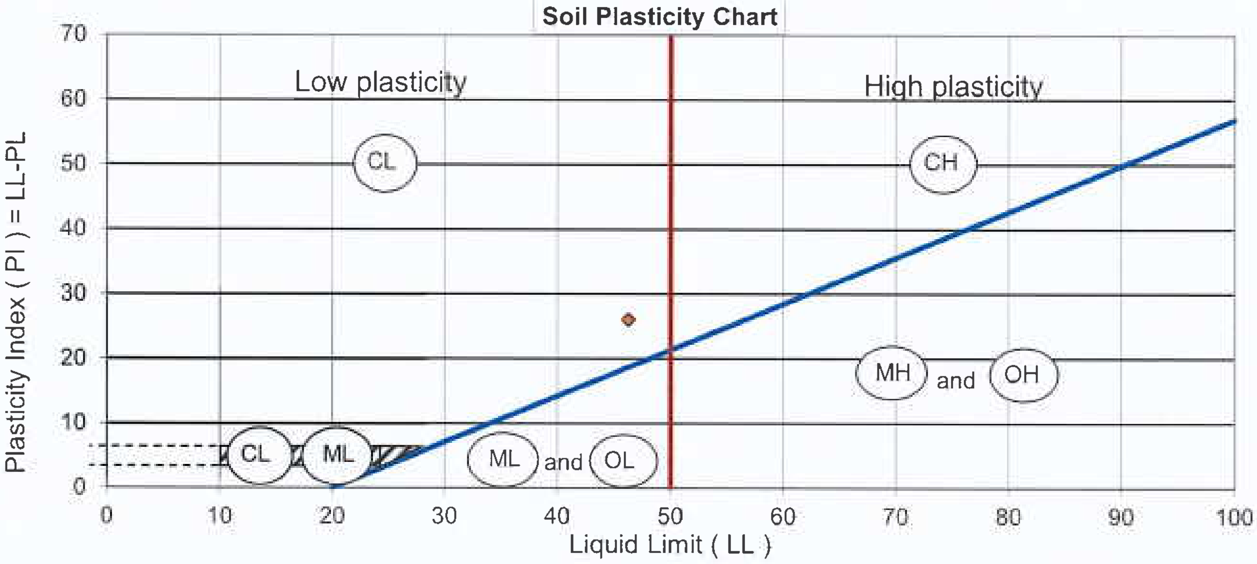
Performed By: M. Tippett
Date: May 28, 2018

Verified By: N.Krebs
Date: May 29, 2018

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	RLTR	**Lab No.:**	OL322-4
Project/Site:	71 Russell Ave.	**Project No.:**	181-05178-00
Borehole No.:	BH18-2	Sample No.:	SS5
Sample Depth:	2.4-3m		





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
46	20	26	54.4

Sample Description: CL - Low plasticity, inorganic clay

Performed By:	M. Tippett	**Date:**	May 28, 2018
Verified By:	N.Krebs	**Date:**	May 29, 2018

Certificate of Analysis

Client: WSP Canada Inc. (SPL)
146 Colonnade Rd., Unit 17
Ottawa, ON
K2E 7Y1
Attention: Mr. Daniel Wall
PO#:
Invoice to: WSP Canada Inc.

Report Number: 1808090
Date Submitted: 2018-05-23
Date Reported: 2018-05-30
Project: 181-05178-00
COC #: 193408

				Lab I.D.	1360443	1360444
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2018-05-04	2018-05-04
				Sample I.D.	BH18-2 SS2 2-4'	BH18-2 SS4 6-8'
Group	**Analyte**	**MRL**	**Units**	**Guideline**		
Agri. - Soil	pH	2.00			8.08	7.49
	SO4	0.01	%		<0.01	<0.01
General Chemistry	Cl	0.002	%		0.003	0.010
	Electrical Conductivity	0.05	mS/cm		0.21	0.24
	Resistivity		mS/cm		4760	4170

Guideline =

*** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range eurofins | Environment Testing

Certificate of Analysis

Client: WSP Canada Inc. (SPL)
146 Colonnade Rd., Unit 17
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K2E 7Y1
Attention: Mr. Daniel Wall
PO#:
Invoice to: WSP Canada Inc.

Report Number: 1808090
Date Submitted: 2018-05-23
Date Reported: 2018-05-30
Project: 181-05178-00
COC #: 193408

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 345966 **Analysis/Extraction Date** 2018-05-24 **Analyst** C F **Method** C CSA A23.2-4B			
Chloride		100	90-110
Run No 346358 **Analysis/Extraction Date** 2018-05-30 **Analyst** C F **Method** AG SOIL			
SO4	<0.01 %	87	70-130
Run No 346428 **Analysis/Extraction Date** 2018-05-30 **Analyst** AC **Method** Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	98	85-115
pH	5.50	100	90-110
Resistivity			

Guideline =

*** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX 2

FIGURE 1 – KEY PLAN

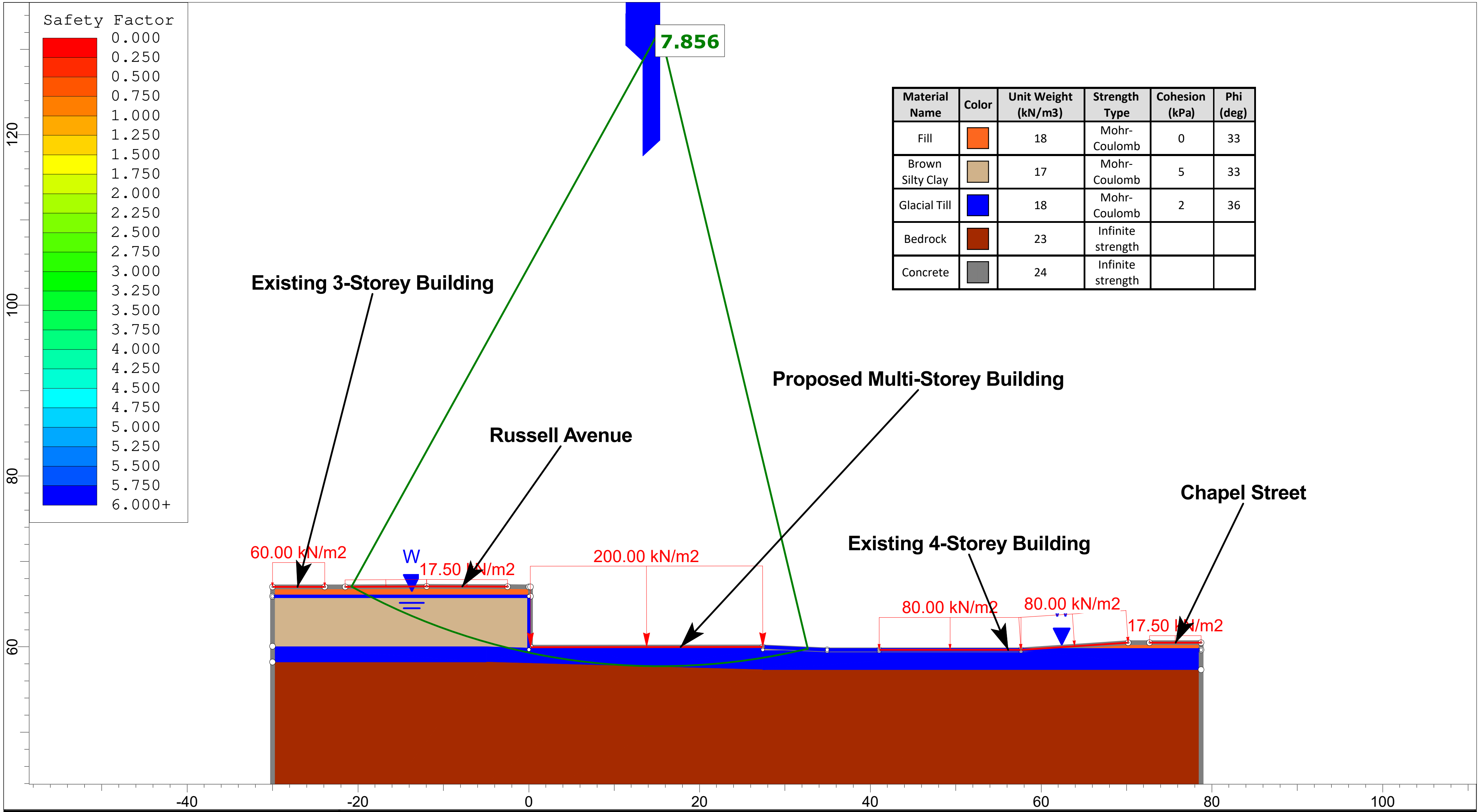
FIGURES 2A AND 2B – SECTIONS FOR SLOPE STABILITY ANALYSIS

DRAWING PG7696-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Fill	🟧	18	Mohr-Coulomb	0	33
Brown Silty Clay	🟫	17	Mohr-Coulomb	5	33
Glacial Till	🟦	18	Mohr-Coulomb	2	36
Bedrock	🟥	23	Infinite strength		
Concrete	⬛	24	Infinite strength		

PATERSON GROUP

SLIDEINTERPRET 9.025

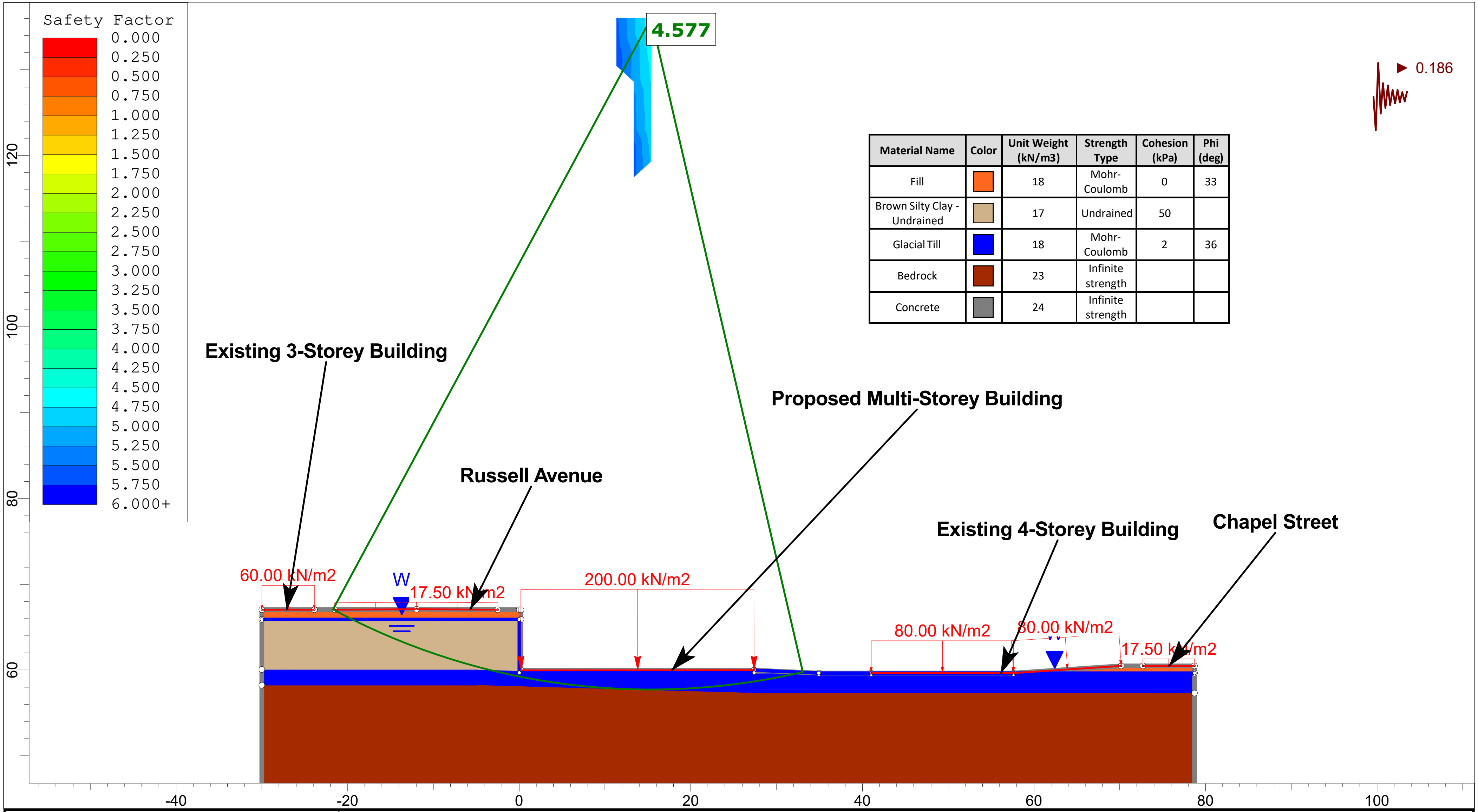
Project No.: **PG7696 - Jersey Developments Inc. - 71 Russell Avenue**

Figure No.: **Figure 2A - Proposed Condition - Static Loading - Section A-A'**

Prepared By: YZ

Approved By: SD

Date: 2026-01-27



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Fill	🟧	18	Mohr-Coulomb	0	33
Brown Silty Clay - Undrained	🟫	17	Undrained	50	
Glacial Till	🟦	18	Mohr-Coulomb	2	36
Bedrock	🟥	23	Infinite strength		
Concrete	⬛	24	Infinite strength		

PATERSON GROUP

SLIDEINTERPRET 9.025

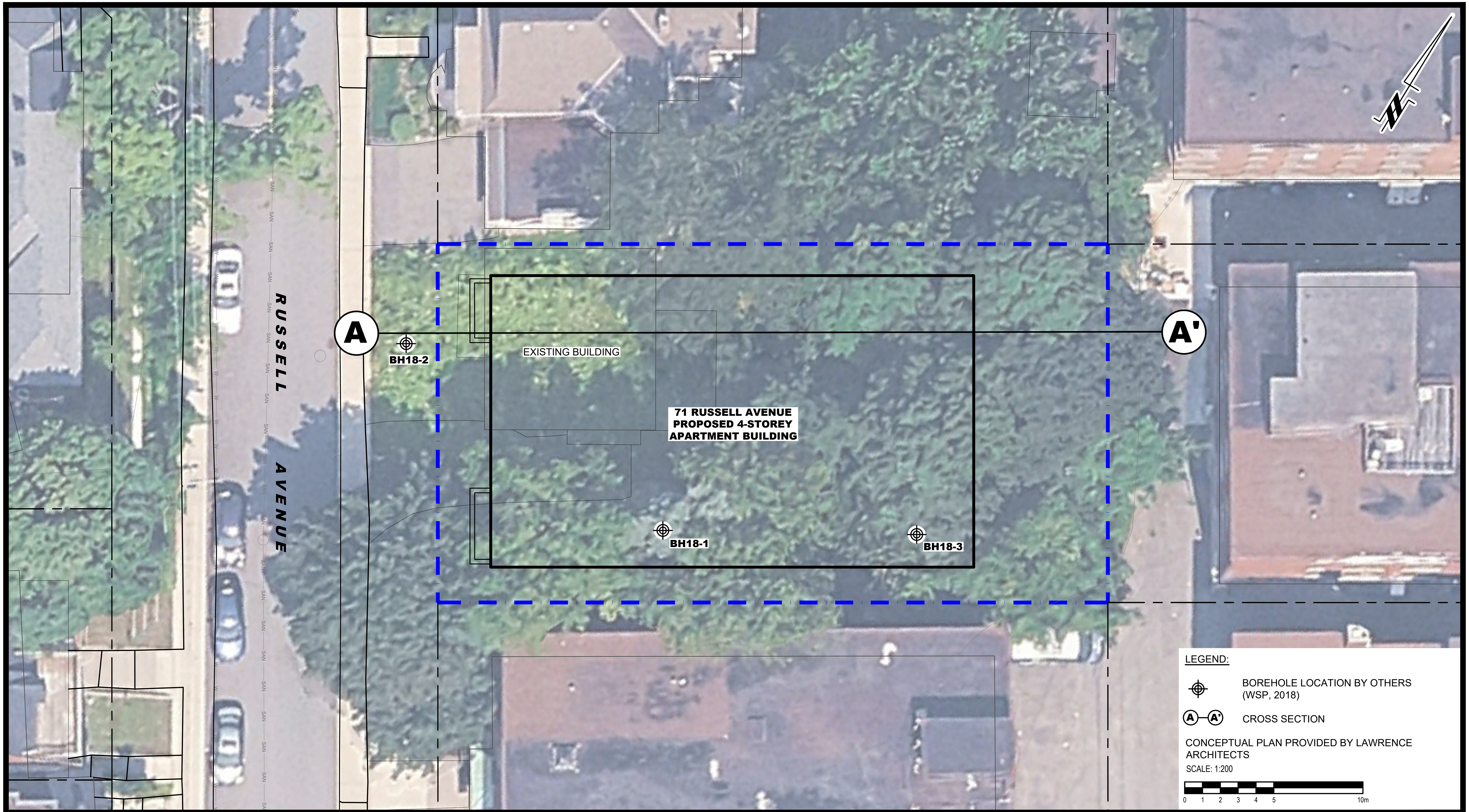
Project No.: **PG7696 - Jersey Developments Inc. - 71 Russell Avenue**

Figure No.: **Figure 2B - Proposed Condition - Seismic Loading - Section A-A'**

Prepared By: YZ

Approved By: SD

Date: 2026-01-27



PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

JERSEY DEVELOPMENTS INC.
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
71 RUSSELL AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:200	Date: 01/2026
Drawn by: GK	Report No.: PG7696-1
Checked by: YZ	Dwg. No.: PG7696-1
Approved by: ZA	Revision No.:

p:\autocad drawings\geotechnical\pg76xx\pg7696\pg7696-1 test hole location plan.dwg

11x17