PATERSON GROUP

Geotechnical Investigation

Proposed Multi-Storey Building

St. Joseph Boulevard and Duford Drive
Ottawa, Ontario

Prepared for Vuze Construction

Report PG6609-1 dated May 10, 2023

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Vuze Construction to conduct a supplemental geotechnical investigation for the proposed multi-storey building to be located at southwest corner of St. Joseph Boulevard and Duford Drive in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2). The objectives of the geotechnical investigation were to:

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

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

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

2.0 Proposed Development

It is expected that the proposed development will consist of a multi-storey building with 3 to 4 underground parking levels. It is further expected that the building footprint will occupy the majority of the subject site and the remainder of the site will be landscaped.

The proposed development is expected to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on April 26, 2017, which consisted of extending a total of three (3) boreholes (BH 1 to BH 3) to a maximum depth of 15.4 m below existing ground surface. Two supplemental investigations were carried out on April 19, 2018 and April 10, 2023; consisted of advancing 2 boreholes (BH1-18 and BH2-18) to a maximum depth of 9.75 m below existing grade and 2 boreholes (BH1-23 and BH 2-23) to a maximum depth of 15 m below existing grade, respectively. The borehole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG6609-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

Overburden thickness was evaluated by dynamic cone penetration testing (DCPT) at BH 1. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Rock samples were recovered from boreholes BH 1-23 and BH 3-23 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes, and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data Sheets in Appendix 1 of this report.

Groundwater

Flexible polyethylene standpipes were installed in the boreholes advanced during the geotechnical investigations carried out on April 26, 2017 and April 10, 2023, to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Two groundwater monitoring wells were installed in the supplemental investigation carried out on April 19, 2018, to further monitor the groundwater levels below the subject site. Typical monitoring well construction details are described below:

- ❏ 1.5 m of slotted 51 mm diameter PVC screen at the base of the aforementioned boreholes.
- ❏ 51 mm diameter PVC riser pipe from the top of the screen to ground surface.
- ❏ No.3 silica sand backfill within annular space around screen.
- ❏ Bentonite hole plug placed directly above PVC slotted screen extending to the existing ground surface.
- ❏ The 51 mm diameter PVC riser extended above the ground surface was covered with a protective steel monitoring well casing.

Specific details of the installation of each monitoring well are further included in the Soil Profile and Test Data Sheets, in Appendix-1.

3.2 Field Survey

The borehole locations and ground surface elevations at the borehole locations were surveyed by Paterson field personnel. The ground surface elevations at the borehole locations were referenced to two temporary benchmarks (TBM), TBM 1 consists of the top of the catch basin located along Duford Drive (Geodetic elevation = 76.32 m) and TBM2 consists of the top spindle of the fire hydrant located in front of 3018 St. Joseph Boulevard (Geodetic elevation = 69.77 m). The borehole locations, TBMs and the ground surface elevation of the borehole locations are presented on Drawing PG4083-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Moisture content testing was performed on a total of 9 to 10 samples recovered from each borehole during the supplemental geotechnical investigation on April 10, 2023. The results of the moisture content testing are shown on the Soil Profile and Test Data Sheets in Appendix 1 of this report.

All samples will be stored in the laboratory for 1 month after this report is completed. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential for sulphate attacks against subsurface concrete structures. The sample was tested to determine the concentration of sulphate and chloride, and the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is located at the southwest corner of St. Joseph Boulevard and Duford Drive and designated as 3030 St. Joseph Boulevard. The ground surface across the site is mostly grass covered with mature trees in the south portion of the site. The ground surface is sloping significantly downward towards the north.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of topsoil with brown silty clay over the upper 75 to 125 mm. Some boreholes shown the presence of a brown to reddish brown silty clay fill material with traces of sand, gravel and organics to depth of 0.8 to 2.3 m, underlain by firm to hard consistency brown silty clay. The brown silty clay was observed to be soft in consistency under one borehole (BH1 - 18). The brown silty clay was observed to turn grey silty clay of soft to very stiff in consistency, approximately between 5.6 to 7.3 m, underlain by very stiff grey silty clay with sand, gravel, occasional cobbles and boulders between approximate depths 8 to 11.30 m.

Bedrock

The bedrock was generally encountered at depths varying from about 9.8 to 12.0 m (elevation of 61.8 to 63.0 m below the existing ground surface. Based on the recovered rock core samples, the bedrock was observed to be comprised of excellent quality grey limestone bedrock.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of limestone of the Bobcaygeon Formation with an overburden drift thickness of 5 to 10 m depth.

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

4.3 Groundwater

Groundwater levels were measured in the monitoring wells installed at BH 1-18 and BH 2-18 on June 13, 2018. The measured groundwater level (GWL) readings are presented in Table 1 below and in the Soil Profile and Test Data sheets in Appendix 1.

Based on these monitoring well readings, we have confirmed that the previous groundwater level readings taken from the piezometers installed at BH 1 and BH 2 on May 4, 2017, were influenced by surface water trapped within the backfilled borehole column. The trapped surface water led to elevated groundwater level readings at the previous boreholes (BH 1 and BH 2), which did not agree with other long-term groundwater indicators, such as observed moisture levels, colouring and undrained shear strengths of the recovered soil samples from the boreholes. Also, moisture content testing completed on the recovered soil samples from BH 1-18 and BH 2-18 are consistent with the recorded groundwater level readings from June 13, 2018.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1-23*	72.85	11.00	61.85	April 17, 2023
BH 2-23*	74.65	dry	---	April 17, 2023
BH 1-18	76.38	6.72	69.66	June 13, 2018
BH 2-18	71.66	3.99	67.67	June 13, 2018
BH 1	77.16	3.29	73.87	May 4, 2017
BH 2	72.75	1.68	71.07	May 4, 2017
BH 3	70.24	Blocked	---	May 4, 2017

Note: -Ground surface elevations at borehole locations were surveyed by Paterson and are referenced to a geodetic datum.
-* indicates borehole was instrumented with a groundwater monitoring well.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is expected that the proposed multi-storey building could be founded by conventional style shallow foundations placed on a clean, limestone bedrock bearing surface.

Bedrock removal will most likely be required to complete a portion of the underground parking levels. Where large quantities of bedrock need to be removed, controlled blasting may be required. If blasting is considered, the blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations. A vibration monitoring program should be implemented and monitored by the geotechnical consultant.

Due to the presence of the silty clay layer, the finished grading adjacent to the proposed building footing will be subjected to a permissible grade restriction in areas where settlement sensitive structures are present.

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 organic or other deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. Due to the anticipated number of underground parking levels and depth of the bedrock at the subject site, it is anticipated that all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal will be required for the construction of the parking garage levels.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the effects on the existing services, buildings and other structures should be addressed.

A pre-blast or construction survey located in proximity of the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Vibration Considerations

Construction operations could cause vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

The following construction equipments could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles or sheet piling will require these pieces of equipments. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the proposed building footprint, 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 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 area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

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

5.3 Foundation Design

Bearing Resistance Values

Footings placed over a clean, surface sounded limestone bedrock surface can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

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 sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Permissible Grade Raise Recommendation

Due to the presence of the silty clay deposit, a permissible grade raise restriction of 1.5 m is recommended for the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. A higher seismic site class, such as Class A or B, may be achievable for this site. However, a site-specific shear wave velocity test would be required to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

The soils underlying the proposed foundations are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

The upper 200 mm below the basement floor slab should consist of a 19 mm clear crushed stone. Alternatively, excavated limestone bedrock could be used as select subgrade material around the proposed building footings, provided the excavated bedrock is suitably crushed to 50 mm in its longest dimension and approved by the geotechnical consultant at the time of placement.

In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lowest basement floor.

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

It is expected that the basement walls are to be poured against a waterproofing and/or drainage system, which will be placed against the shoring face and exposed bedrock face, where encountered. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Where soil is to be retained, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e., below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

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

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.5

γ = unit weight of fill of the applicable retained soil (kN/m^3)

H = height of the wall (m)

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

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

Seismic Earth Pressures

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

a_c = $(1.45 - a_{max}/g)a_{max}$

γ = unit weight of fill of the applicable retained soil (kN/m^3)

H = height of the wall (m)

g = gravity, 9.81 m/s^2

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

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

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

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

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

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

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as DYWIDAG Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four (4) times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the "passive" or the "post-tensioned" type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length.
As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

The unconfined compressive strength of limestone bedrock ranges between 80 and 100 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of 1.0 MPa, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on available bedrock information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock. Therefore, Hoek and Brown parameters (m and s) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required. For our calculations, the following parameters were used.

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unit weight - Submerged Bedrock	15kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75- and 125-mm diameter hole are provided in Table 3.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor				
Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.2	0.55	1.75	250
	2	0.8	2.8	500
	3.2	1.4	4.6	1000
	5.3	2.2	7.5	2000
125	1	0.5	1.5	250
	1.7	0.7	2.4	500
	2.6	1.1	3.7	1000
	4.1	1.8	5.9	2000

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.7 Pavement Design

Car only parking areas, access lanes and heavy truck parking areas are proposed as part of the site development site. The proposed pavement structures for these areas are provided in Tables 4 and 5 below.

Table 4 - Recommended Pavement Structure - Driveways / 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.	

Table 5 - Recommended Pavement Structure – Access Lanes / Heavy Truck Parking	
Thickness (mm)	**Material Description**
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II	

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. 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 structure. It is expected that insufficient room is available for exterior backfill. It is recommended that the composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) and waterproofing membrane extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. An interior perimeter drainage consisting of a minimum 150 mm diameter perforated, corrugated PVC pipe be placed along the interior side of the exterior footing. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Subfloor Water Infiltration

Due to the variability in the limestone, it is expected that water might infiltrate to the bottom of the excavation, through seams and cracks in the bedrock surface. Paterson should review the water infiltration upon completion of the excavation. It is recommended to carry a minimum 75 mm mubslab and horizontal membrane to act has hydraulic barrier on top of the bedrock.

Foundation Backfill

For areas where sufficient space is available for backfill against the exterior sides of the foundation walls, the backfill material 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 Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system.

Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.
Exterior unheated footings, such as isolated 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 an equivalent combination of soil cover and foundation insulation.

The underground parking area should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

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 expected that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. Excavations below the groundwater level should be cut back at a maximum slope of 1.5H:1V, although, it is expected that all the excavations will be above the long-term groundwater table.

The subsurface soil at this site is considered to be mainly 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.

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

Due to the depth of excavation, a temporary shoring system will be required to complete the excavation. The proposed shoring system will have to take into consideration support of the existing structure due to the close proximity of the neighboring buildings and road. Paterson requests permission to review the building cross section to assess the shoring requirements for the site.

The shoring requirements designed by a geostructural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 6 - Soil Parameters	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

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

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

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

6.4 Pipe Bedding and Backfill

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm. The bedding should extend to the spring line of the pipe. Cover material, should be placed from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

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

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

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (i.e.- less than 25,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts to Neighbouring Properties

Based on our observations, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The neighbouring structures are expected to be founded within the native silty clay and/or over a bedrock bearing surface. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the low permeability of the native soils.

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 using 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 into 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.

Where excavations are completed in proximity to existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.8 Landscaping Considerations

Tree Planting Restrictions

The proposed residential dwellings are located in a moderate sensitivity area with respect to tree plantings over a silty clay deposit. It is recommended that trees placed within 4.5 m of the foundation wall consist of low water demanding trees with shallow roots systems that extend less than 1.5 m below ground surface. Trees placed greater than 4.5 m from the foundation wall may consist of typical street trees, which are typically moderate water demand species with roots extending to a maximum 2 m depth.

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

6.9 Slope Stability Analysis

Slope Conditions

Three slope sections (Sections A, B and C) were identified as worst-case scenarios based on available topographic mapping of the area. The cross-section locations and topographic mapping information are presented on Drawing PG4083-1 - Test Hole Location Plan in Appendix 2.

Section A was profiled across the site from Duford Drive to St. Joseph Boulevard. A difference in elevation of approximately 7 m is present across the slope section. The slope across the subject site is shaped to an approximately 5H:1V slope. The top of slope at Section B and Section C is located behind the rear yards of the Kennedy Lane West dwellings with a difference in elevation of approximately 18 m between the top and toe of slope. The slope surface across the subject slopes was noted to be grass covered with no signs of slope instability noted.

Section C was located within a former slope failure area. It is understood that a slope failure occurred along the east side of Duford Drive in the 1960s. Photographs of the slope failure were provided to Paterson for this response. Photographs 1 and 2 presented in Appendix 2 show a shallow slope failure across a limited section of overall slope face. Based on slope features noted in the photographs, such as lack of vegetation across the slope face and the soil surface in the area of Duford Drive, it appears that the slope failure occurred across a section of the slope, which had been recently re-shaped as part of the construction of Duford Drive.

The natural grade of the slope face was drastically changed during the construction of Duford Drive. It is expected that the slope failure can be directly contributed to the steepness of excavated slope face along with exposure to precipitation events before a vegetative layer could establish. It should be further noted that a vegetative layer across a slope face promotes surficial run-off during precipitation events and limits infiltration of rainwater into the slope soil. Infiltration of water from precipitation events into a slope reduces overall slope stability.

The current slope face was noted to include a terraced area along the base of the slope face in the area of the former slope failure (see Photos 3, 4 and 5 in Appendix 2). It is suspected that the terraced area was introduced after the initial slope failure to stabilize the reinstated slope. Currently, the slope face was noted to be grass covered with mature trees. No signs of slope instability were noted.

Slope Stability Analysis

The analysis of the stability of the slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's 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 the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

The cross-sections were analyzed taking into account a groundwater level at ground surface. Subsoil conditions at the cross-sections were inferred based on the findings at nearby borehole locations, field observations during our site visit and general knowledge of the area's geology.

Static Analysis

The results for the existing slope conditions at Section A and Section B are shown in Figure 2 and Figure 4 in Appendix 2. The factor of safety was found to be greater than 1.5 for Section A and B when analyzed under static conditions. It should be noted that a slope stability analysis was completed for Section A due to the steepness of the slope observed, which was considered to be a worst-case scenario for the subject site. Section B was analyzed to include the adjacent slope opposite of Duford Drive. Section C was analyzed considering the upper 2 m of the slope face to be fully saturated and the remainder of the slope is saturated below the long-term groundwater table at the former slope failure location. A global slope stability factor of safety of greater than 1.5 was determined for Section C based on our analysis. This result indicates a stable slope. It should be noted that the above noted saturated condition for the subject slope is considered to be a worst-case scenario due to the low permeability of the stiff silty clay deposit based on our knowledge of the subsoil conditions and the spring groundwater level readings at the monitoring well locations within the subject site. Based on the monitoring program measurements, the groundwater level was found to be at an elevation of 69.7 m at the top of slope (6.7 m depth) within the subject site and an elevation of 67.7 m at the bottom of slope (4 m depth).

Seismic Loading Analysis

An analysis considering seismic loading was also completed. A horizontal seismic acceleration, K_h, of 0.16G was considered for the analyzed sections.
A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the analyses including seismic loading are shown in Figure 3, Figure 5, and Figure 7B for the slope sections. The results indicate that the factor of safety at Section A, B and C is greater than 1.1. Based on these results, the slopes are considered to be stable under seismic loading.

Construction Consideration

Based on the slope stability of the slope across the site and the stiffness of the underlying silty clay deposit, it is not expected that the vibrations associated with the temporary shoring installation will not have negative impacts on the overall slope stability.

The proposed structures should be able to support the differential lateral forces across the south portion of the site. It is expected that the south grade will be located 4 to 4.5 m higher than the front of the building.

6.7 Retaining Wall Design

It is expected that retaining walls will be required to grade the property. Retaining walls higher than 1.0 m should be designed by a professional engineer, as per City of Ottawa retaining wall design standards. The bearing resistance provided in Section 5.3 are applicable to the proposed retaining walls.

The soil parameters presented in Tables 6 and 7 should be used for the design of the retaining walls. The design should also include a global stability analysis of the system.

Global stability analysis should include static and seismic analysis of the system and present the minimum factor of safety. The system should be design for a factor of safety of 1.5 under static conditions and 1.1 for seismic conditions.

Backfill Material

The retaining wall should be backfilled with free-draining granular backfill materials and incorporate longitudinal drains and weep holes to provide positive drainage of the backfill. For the purpose of this report, it is recommended that the wall be backfilled with either OPSS Granular B Type II or Granular A materials. The backfill should be placed within a wedge-shaped zone defined by a line drawn up and back from the back edge of the base block of the wall at an inclination of 1H:1V or a minimum of 1 m behind the back of the blocks. All material should be compacted to a minimum of 98% of the material's SPMDD.

Lateral Earth Pressures

It is recommended that a minimum of 1 m of the backfill material to consist of clean imported engineered crushed stone such as OPSS Granular A or Granular B Type II. The soil parameters presented in Table 6 should be used for the design of the retaining wall.

The Geotechnical Parameters for backfill material can be used as shown in Table- 7.

Table-7 Geotechnical Parameters for Backfill Material

Material Description	Unit Weight (KN/m³)		Friction Angle Φ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_a	At-Rest K_o	Passive K_p
OPSS Granular A (Crushed stone)	22	13.5	0.6	36	0.26	0.41	3.85
OPSS Granular B Type II (Crushed stone)	22	13.5	0.6	36	0.26	0.41	3.85

Notes:

I. Properties for fill materials are for condition of 98.0% of Standard Proctor Maximum Dry Density
II. The earth pressure coefficients provided are for horizontal backfill profile.
III. For soil above the groundwater level the "drained" unit weight should be used and below groundwater level the "effective" unit weight should be used.

Retaining Wall Types

Where the retaining wall is to be higher than 1 m and or support a roadway or slope consideration can be given to using large precast concrete retaining wall system such as Redi-Rock and Stone Strong. Quality precast products are designed to resist large load under gravity and may not require as much excavation or reinforcement. Typical products vary in size from 0.6 to over 2.4 m in depth depending on the total height of the wall. The size of these supporting structures should be considered when drafting site plan and grading plans.

6.8 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

- Prepare a temporary shoring plan and associate construction monitoring plan.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

All excess soils, 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*.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by Paterson.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Vuze Construction, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 10, 2023
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Pratheep Thirumoolan, M.Eng.

Joey R. Villeneuve, M.A.Sc., P.Eng, ing.

Report Distribution:

- ❏ The Vuze Construction (e-mail copy)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
3020 St-Joseph Boulevard
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 10, 2023

FILE NO. PG6609

HOLE NO. BH 1-23

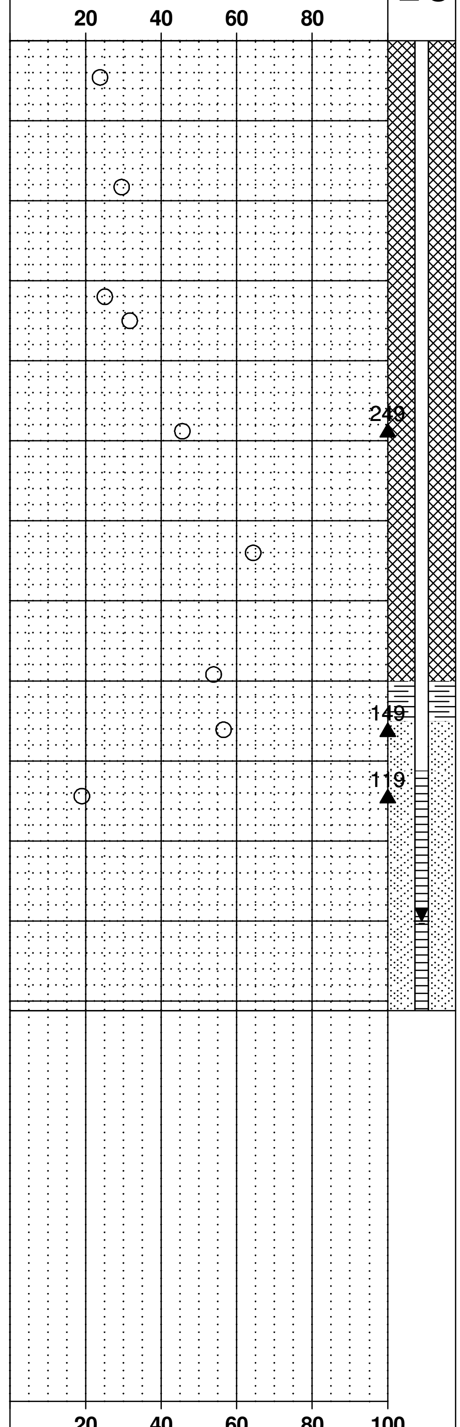
SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
TOPSOIL	0.08	AU	1		
FILL: Reddish brown silty clay, trace gravel and organics	3.25	SS	2	63	15
Hard to very stiff, brown **SILTY CLAY** - grey by 5.6m depth	8.08	SS	3	88	16
		SS	4	100	P
		SS	5	100	6
		SS	6	100	6
GLACIAL TILL: Very stiff, grey silty clay with sand, some gravel, occasional cobbles	9.86	G	7		
		SS	8	21	P
BEDROCK: Excellent quality, grey limestone	12.12	RC	1	100	93
		RC	2	100	100
End of Borehole					
(GWL @ 11.00m - April 17, 2023)					

DEPTH (m)	ELEV. (m)
0	72.85
1	71.85
2	70.85
3	69.85
4	68.85
5	67.85
6	66.85
7	65.85
8	64.85
9	63.85
10	62.85
11	61.85
12	60.85

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
3020 St-Joseph Boulevard
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

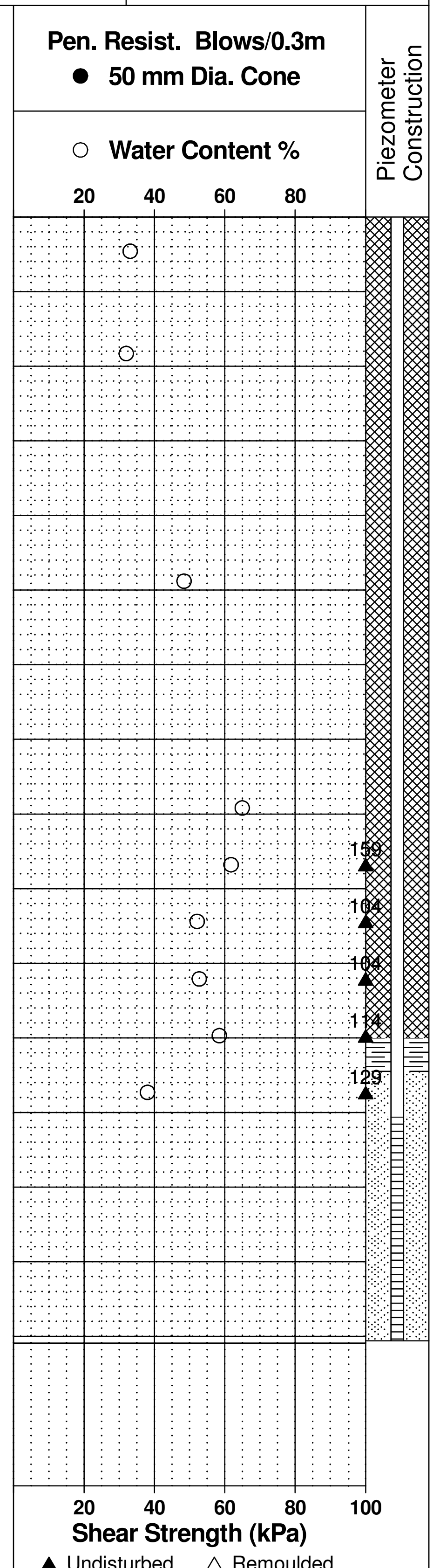
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** April 10, 2023

FILE NO. PG6609

HOLE NO. BH 2-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	74.65
TOPSOIL 0.13		AU	1				
FILL: Reddish brown silty clay, trace gravel						1	73.65
		SS	2	63	7	2	72.65
3.20						3	71.65
Hard to very stiff, brown **SILTY CLAY**						4	70.65
		SS	3	100	8	5	69.65
						6	68.65
- grey by 7.3m depth						7	67.65
		SS	4	100	2	8	66.65
		SS	5	100	P		
		SS	6	100	P	9	65.65
		SS	7	100	P	10	64.65
11.35		SS	8	100	P	11	63.65
GLACIAL TILL: Very stiff, grey silty clay with sand, some gravel, occasional cobbles 12.06		SS	9	100	P	12	62.65
BEDROCK: Excellent quality, grey limestone		RC	1	100	92	13	61.65
		RC		100	100	14	60.65
15.08						15	59.65
End of Borehole							
(BH dry - April 17, 2023)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - St. Joseph Blvd.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 3018 St. Joseph Boulevard. Geodetic elevation = 69.77m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 19, 2018

FILE NO. PG4083

HOLE NO. BH 1-18

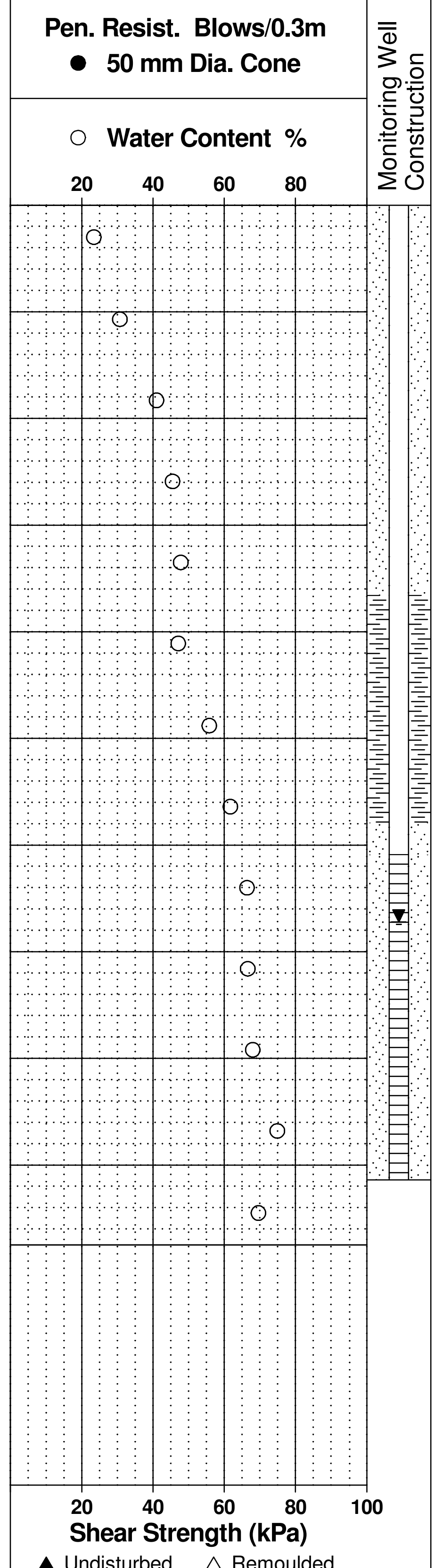
SOIL DESCRIPTION

GROUND SURFACE

- **TOPSOIL** — 0.15
- Brown **SILTY CLAY,** trace sand — 0.76
- Stiff to firm, brown **SILTY CLAY**
- - firm to soft and grey by 5.5m depth — 9.75
- End of Borehole
- (GWL @ 6.72m - June 13, 2018)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	71	8
SS	3	100	9
SS	4	100	7
SS	5	100	7
SS	6	100	6
SS	7	100	5
SS	8	100	1
SS	9	100	2
SS	10	100	1
SS	11	100	W
SS	12	100	W
SS	13	100	W

DEPTH (m)	ELEV. (m)
0	76.38
1	75.38
2	74.38
3	73.38
4	72.38
5	71.38
6	70.38
7	69.38
8	68.38
9	67.38



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - St. Joseph Blvd.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 3018 St. Joseph Boulevard. Geodetic elevation = 69.77m.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** April 19, 2018

FILE NO. PG4083

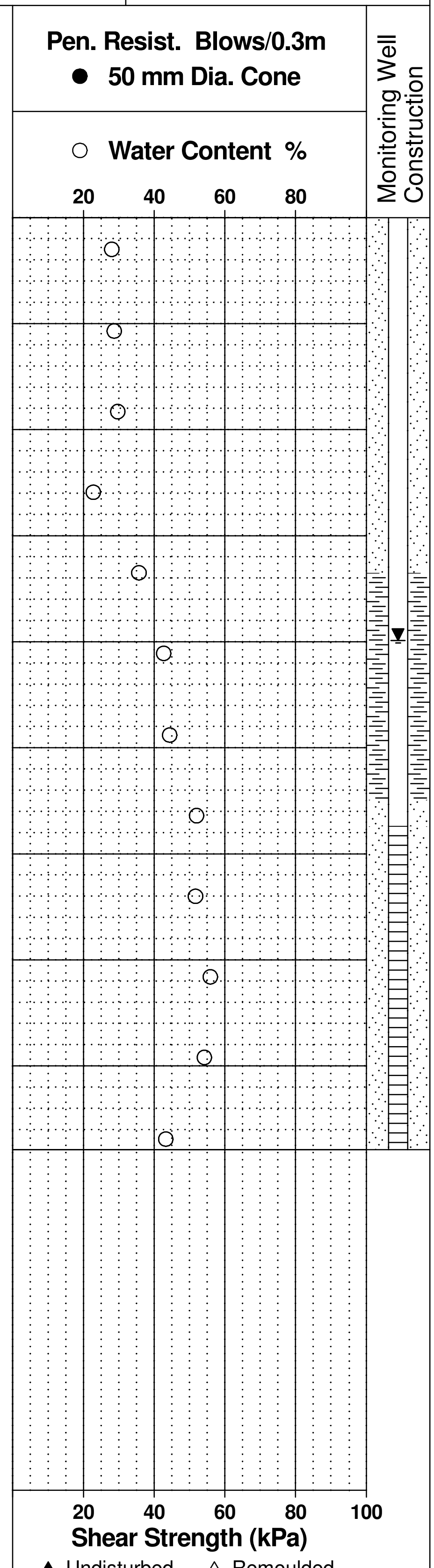
HOLE NO. BH 2-18

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.66
TOPSOIL	0.13	AU	1				
Very stiff, brown **SILTY CLAY**	1.52	SS	2	58	12	1	70.66
Very stiff, brown **SILTY CLAY,** some gravel, trace sand	2.30	SS	3	21	9	2	69.66
Very stiff to stiff, brown **SILTY CLAY**		SS	4	62	12		
		SS	5	100	17	3	68.66
		SS	6	100	13	4	67.66
		SS	7	100	9	5	66.66
		SS	8	100	5	6	65.66
- firm to soft and grey by 6.4m depth		SS	9	100	W		
		SS	10	100	W	7	64.66
		SS	11	100	2	8	63.66
	8.79	SS	12	92	50+		

End of Borehole

Practical refusal to augering at 8.79m depth

(GWL @ 3.99m - June 13, 2018)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - St. Joseph Blvd.
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin (as shown on Dwg. PG4083-1). Geodetic elevation = 76.32m.

REMARKS

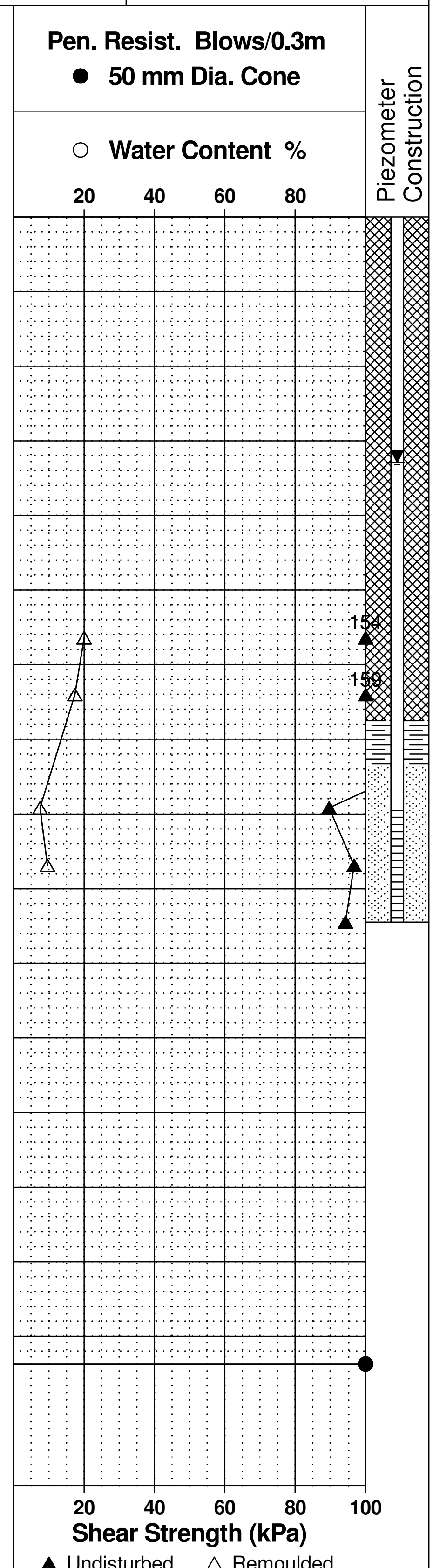
BORINGS BY CME 55 Power Auger

DATE April 26, 2017

FILE NO. PG4083

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	77.16
TOPSOIL 0.18	AU	1				
FILL: Brown silty clay, some sand, trace gravel, cobbles, boulders and wood 2.13	SS	2	46	8	1	76.16
	SS	3	38	10	2	75.16
Very stiff to stiff, brown **SILTY CLAY**	SS	4	54	16	3	74.16
	SS	5	100	17	4	73.16
	SS	6	100	11	5	72.16
	SS	7	100	9	6	71.16
- grey by 7.6m depth					7	70.16
9.45					8	69.16
Dynamic Cone Penetration Test (DCPT) commenced at 9.45m depth. Cone pushed to 15.37m depth.					9	68.16
					10	67.16
					11	66.16
					12	65.16
					13	64.16
(GWL @ 3.29m - May 4, 2017)					14	63.16
15.37					15	62.16
End of Borehole Practical DCPT refusal at 15.37m depth						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - St. Joseph Blvd.
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin (as shown on Dwg. PG4083-1). Geodetic elevation = 76.32m.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** 26 April 2017

FILE NO. PG4083

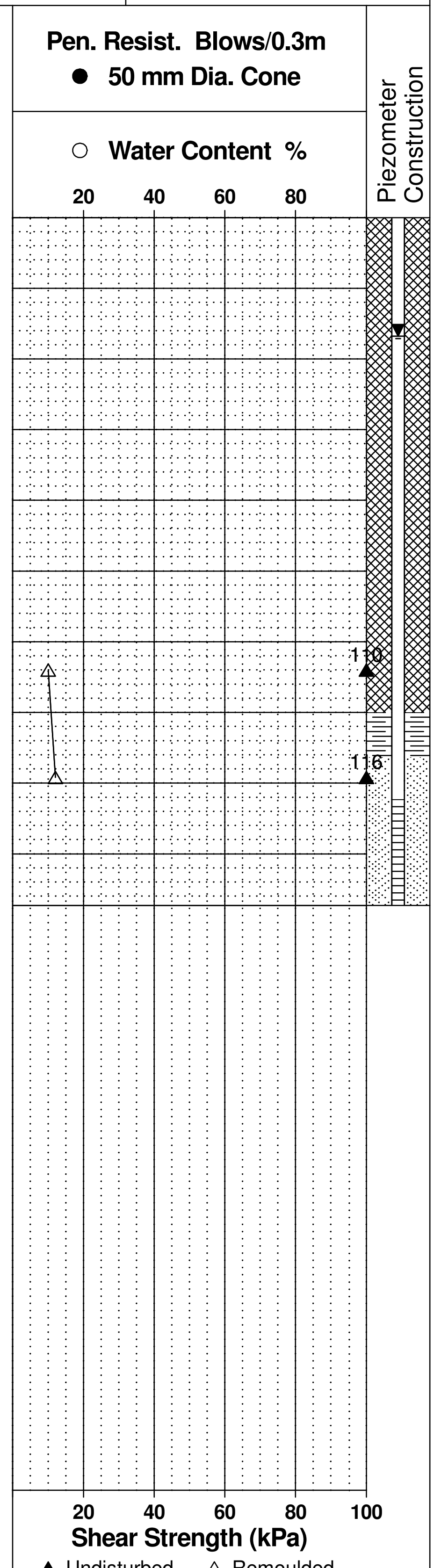
HOLE NO. BH 2

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	72.75
TOPSOIL	0.15	AU	1				
FILL: Loose, brown silty clay, some sand, trace gravel and organics	1.52	SS	2	67	8	1	71.75
Very stiff to firm, brown **SILTY CLAY** - sand seams to 2.5 m depth - grey by 7.6m depth	9.14	SS	3	75	13	2	70.75
		SS	4	83	16	3	69.75
		SS	5	100	12		
		SS	6	100	10	4	68.75
		SS	7	100	10	5	67.75
		SS	8	100	6	6	66.75
						7	65.75
						8	64.75
						9	63.75
GLACIAL TILL: Very dense, grey silt with clay, sand, gravel, cobbles and shale fragments	9.73	SS	9	100	50+		

End of Borehole

Practical refusal to augering at 9.73m depth

(GWL @ 1.68m - May 4, 2017)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Building - St. Joseph Blvd.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of 3018 St. Joseph Boulevard. Geodetic elevation = 69.77m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 26, 2017

FILE NO. PG4083

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	70.24
FILL: Brown silty clay with topsoil, trace construction debris — 0.20		AU	1				
		SS	2	62	15	1	69.24
		SS	3	58	21	2	68.24
		SS	4	100	17	3	67.24
Very stiff to firm, brown **SILTY CLAY**		SS	5	100	14		
		SS	6	100	7	4	66.24
						5	65.24
- grey by 5.3m depth						6	64.24
						7	63.24
8.13		SS	7		4	8	62.24

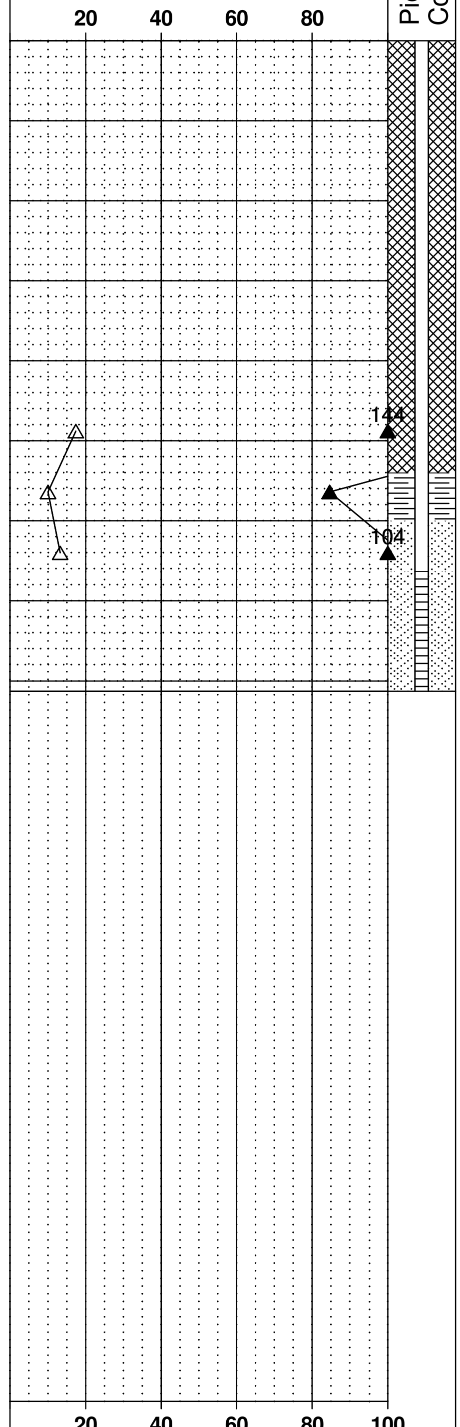
End of Borehole

Practical refusal to augering at 8.13m depth

(BH dry and blocked at 3.63m depth - May 4, 2017)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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 = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

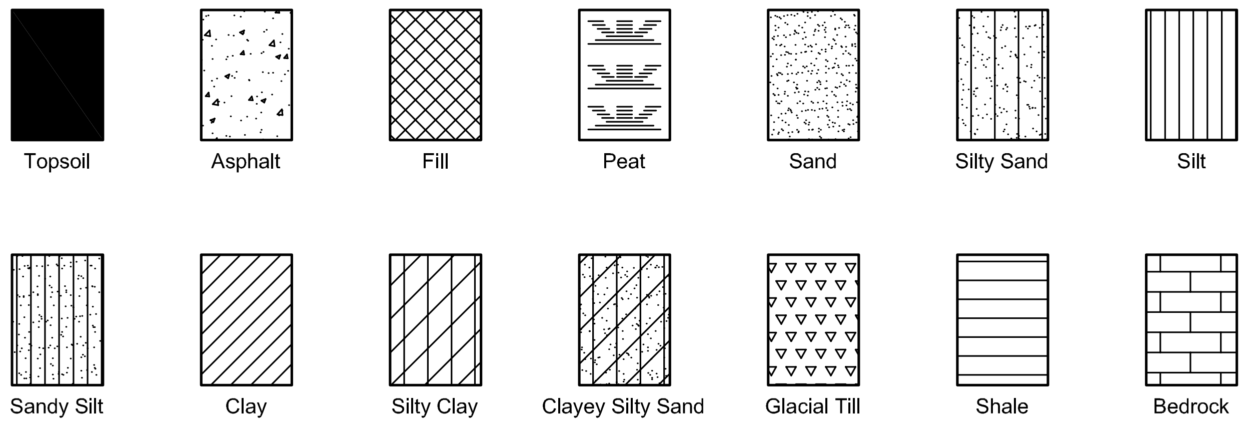
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

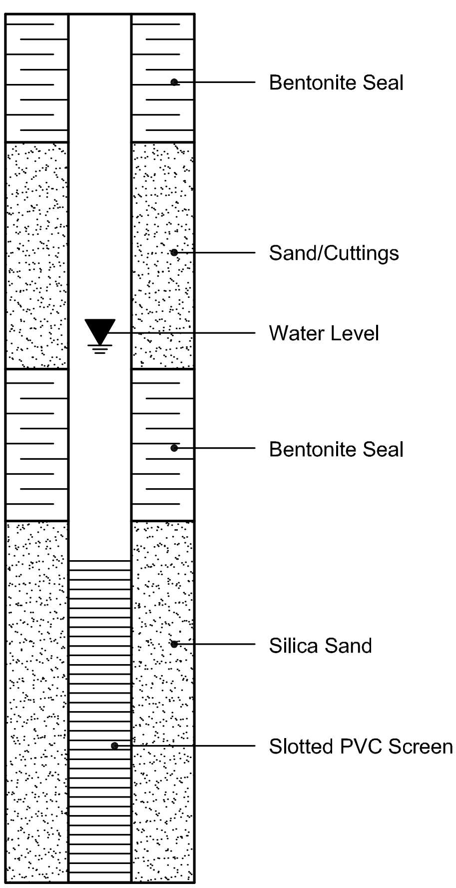
SYMBOLS AND TERMS (continued)

STRATA PLOT

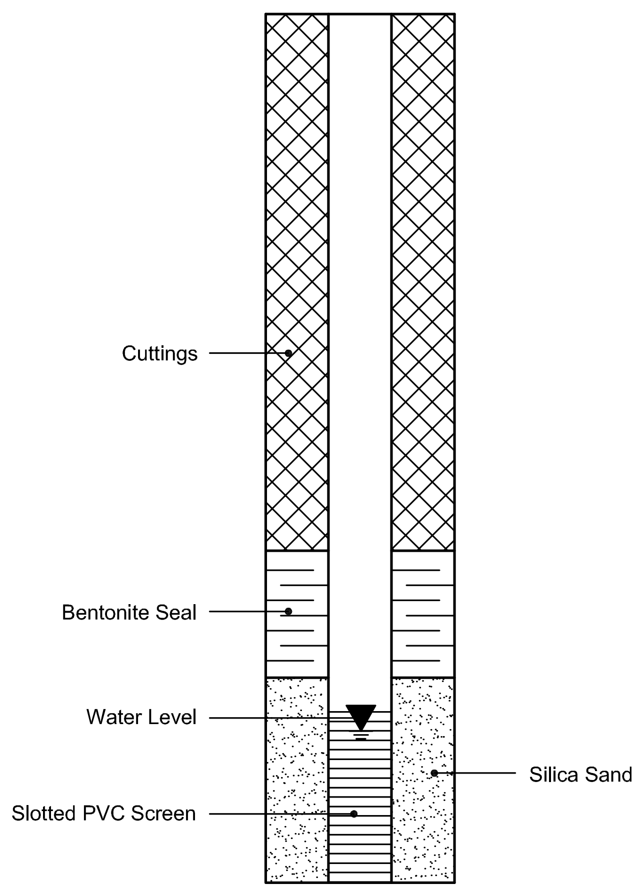


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL
LABORATORIES LTD.

Order #: 2315331

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 57221

Report Date: 18-Apr-2023

Order Date: 12-Apr-2023

Project Description: PG6609

	Client ID:	BH2-23 SS4 (25'-27')	-	-	-
	Sample Date:	10-Apr-23 00:00	-	-	-
	Sample ID:	2315331-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	61.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.73	-	-	-
Resistivity	0.1 Ohm.m	8.7	-	-	-
Anions					
Chloride	10 ug/g dry	468	-	-	-
Sulphate	10 ug/g dry	54	-	-	-

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 20659

Report Date: 04-May-2017

Order Date: 28-Apr-2017

Project Description: PG4083

	Client ID:	BH3-SS6	-	-	-
	Sample Date:	26-Apr-17	-	-	-
	Sample ID:	1717530-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	65.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.19	-	-	-
Resistivity	0.10 Ohm.m	10.3	-	-	-
Anions					
Chloride	5 ug/g dry	604	-	-	-
Sulphate	5 ug/g dry	103	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 TO 7 - SLOPE STABILITY SECTIONS

HISTORIC AND CURRENT SLOPE PHOTOGRAPHS

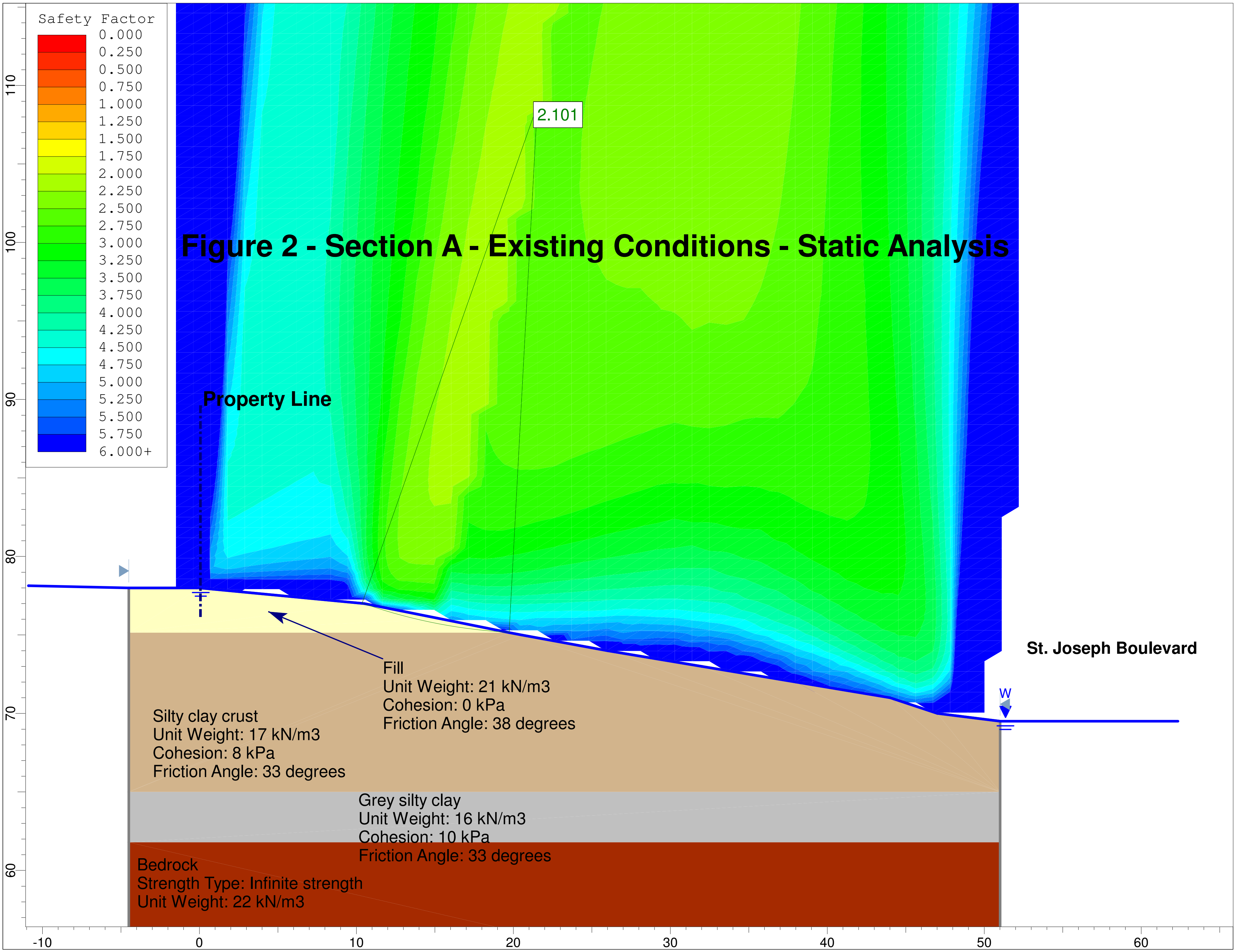
AT FORMER SLOPE FAILURE

DRAWING PG6609-1 - TEST HOLE LOCATION PLAN

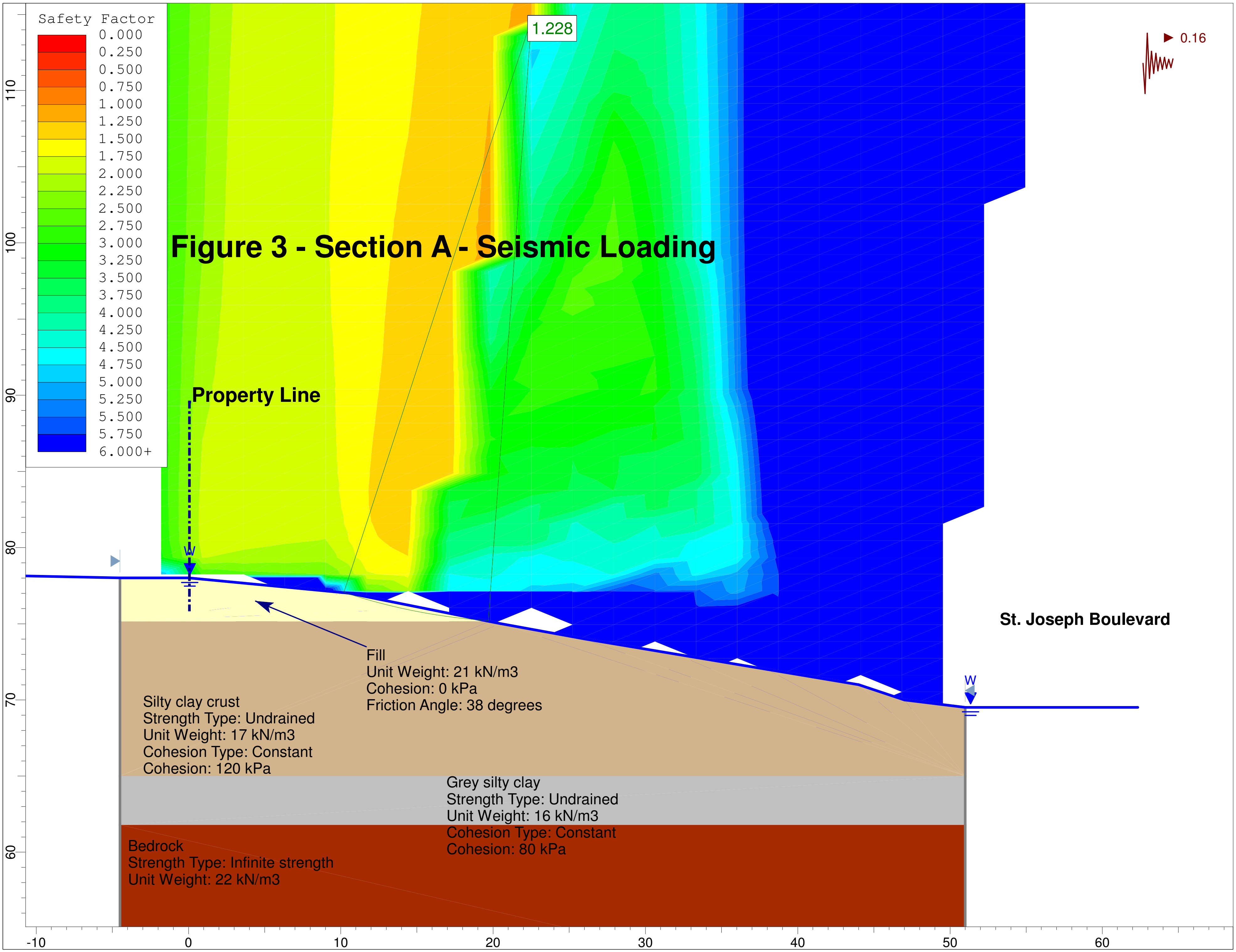
FIGURE 1

KEY PLAN

PATERSON GROUP



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+
2.101
Figure 2 - Section A - Existing Conditions - Static Analysis
Property Line
St. Joseph Boulevard
W
Fill
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Silty clay crust
Unit Weight: 17 kN/m3
Cohesion: 8 kPa
Friction Angle: 33 degrees
Grey silty clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
110
100
90
80
70
60
-10
0
10
20
30
40
50
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+
1.228
0.16
Figure 3 - Section A - Seismic Loading
Property Line
W
St. Joseph Boulevard
Fill
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Silty clay crust
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion Type: Constant
Cohesion: 120 kPa
Grey silty clay
Strength Type: Undrained
Unit Weight: 16 kN/m3
Cohesion Type: Constant
Cohesion: 80 kPa
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
110
100
90
80
70
60
-10
0
10
20
30
40
50
60

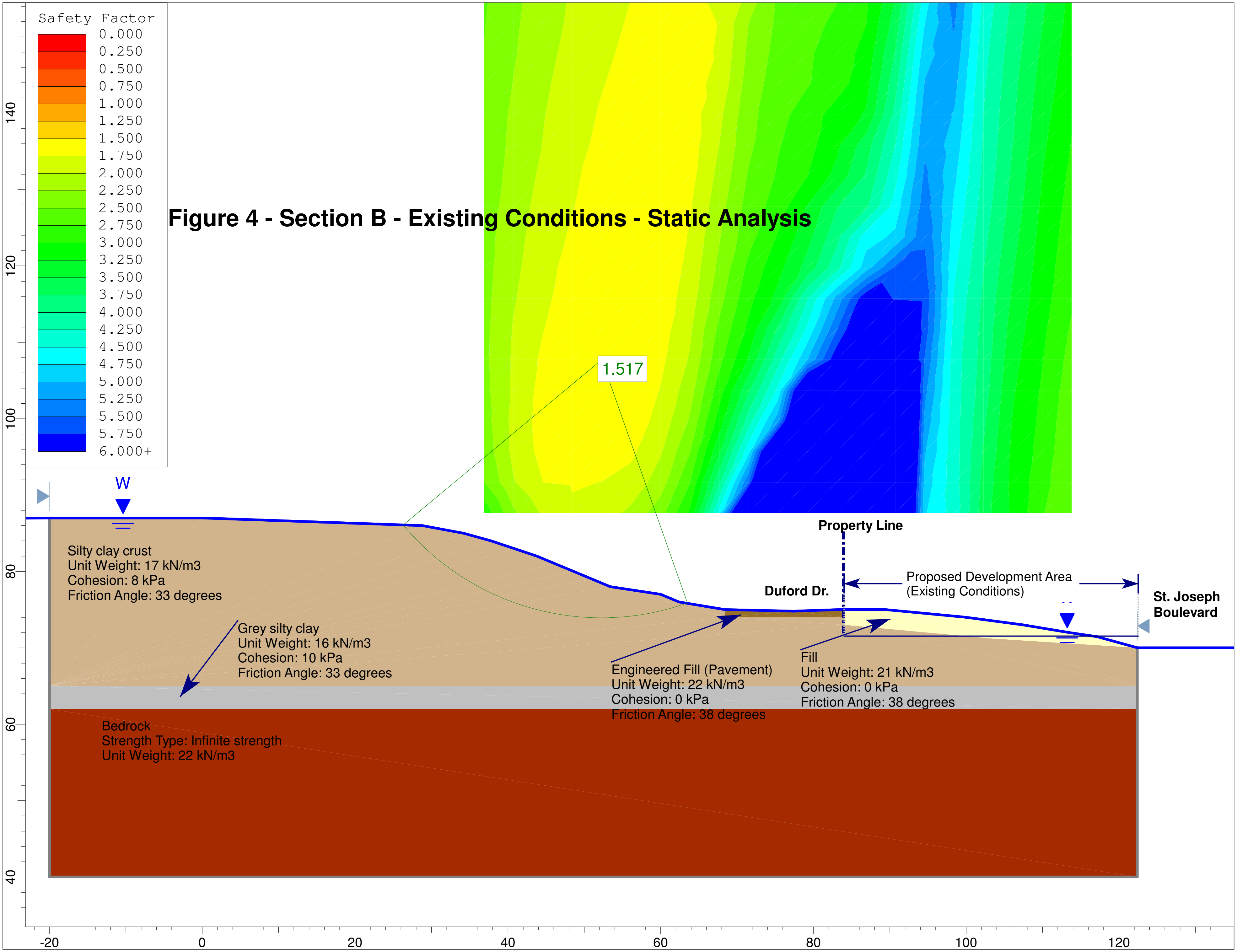


Figure 4 - Section B - Existing Conditions - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.517
W
Silty clay crust
Unit Weight: 17 kN/m3
Cohesion: 8 kPa
Friction Angle: 33 degrees
Grey silty clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Property Line
Duford Dr.
Proposed Development Area
(Existing Conditions)
St. Joseph
Boulevard
Engineered Fill (Pavement)
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Fill
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
140
120
100
80
60
40
-20
0
20
40
60
80
100
120

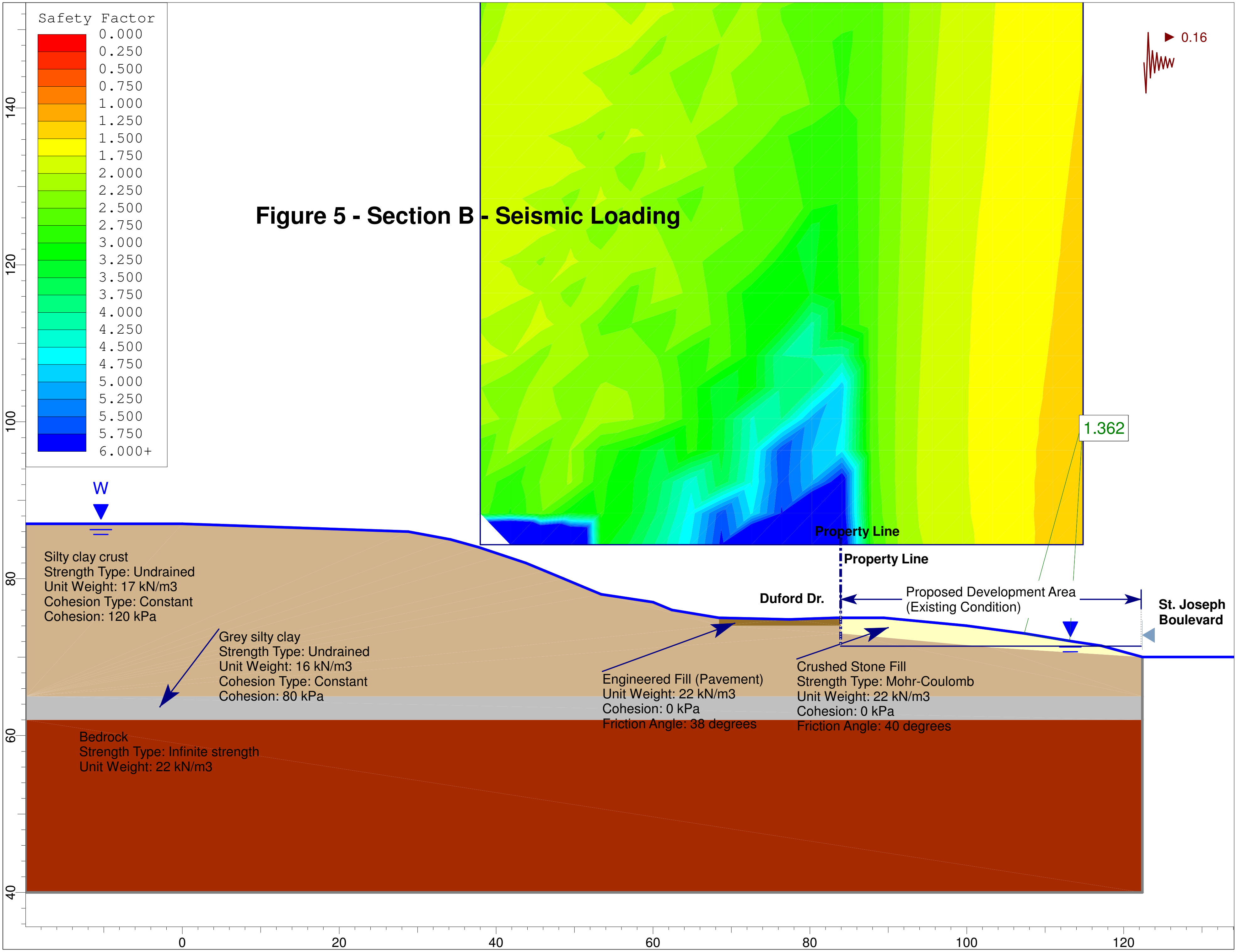


Figure 5 - Section B - Seismic Loading
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
0.16
1.362
W
Silty clay crust
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion Type: Constant
Cohesion: 120 kPa
Grey silty clay
Strength Type: Undrained
Unit Weight: 16 kN/m3
Cohesion Type: Constant
Cohesion: 80 kPa
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
Engineered Fill (Pavement)
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Crushed Stone Fill
Strength Type: Mohr-Coulomb
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Friction Angle: 40 degrees
Property Line
Property Line
Duford Dr.
Proposed Development Area
(Existing Condition)
St. Joseph
Boulevard
140
120
100
80
60
40
0
20
40
60
80
100
120

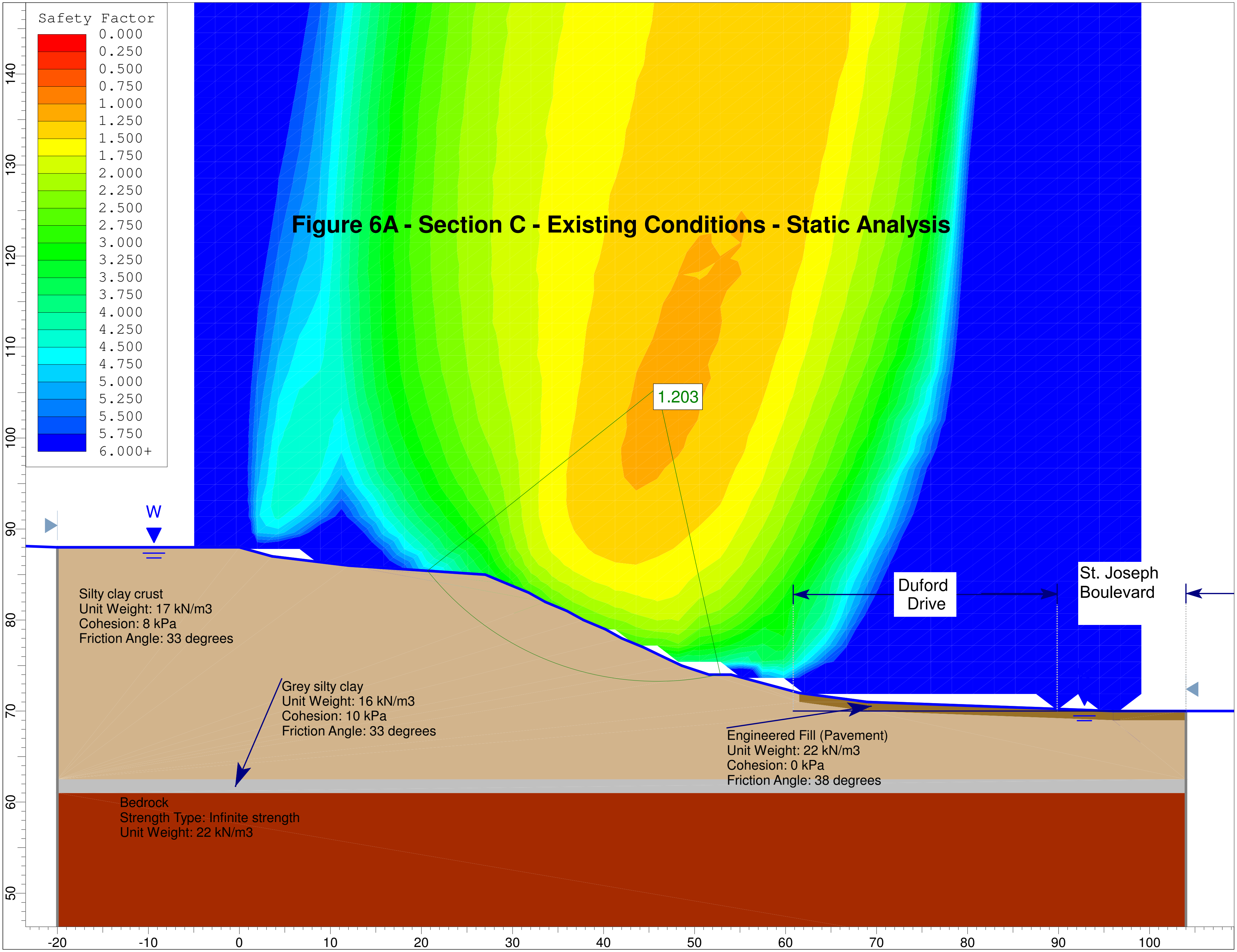


Figure 6A - Section C - Existing Conditions - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.203
W
Duford Drive
St. Joseph Boulevard
Silty clay crust
Unit Weight: 17 kN/m3
Cohesion: 8 kPa
Friction Angle: 33 degrees
Grey silty clay
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Engineered Fill (Pavement)
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
140
130
120
110
100
90
80
70
60
50
-20
-10
0
10
20
30
40
50
60
70
80
90
100

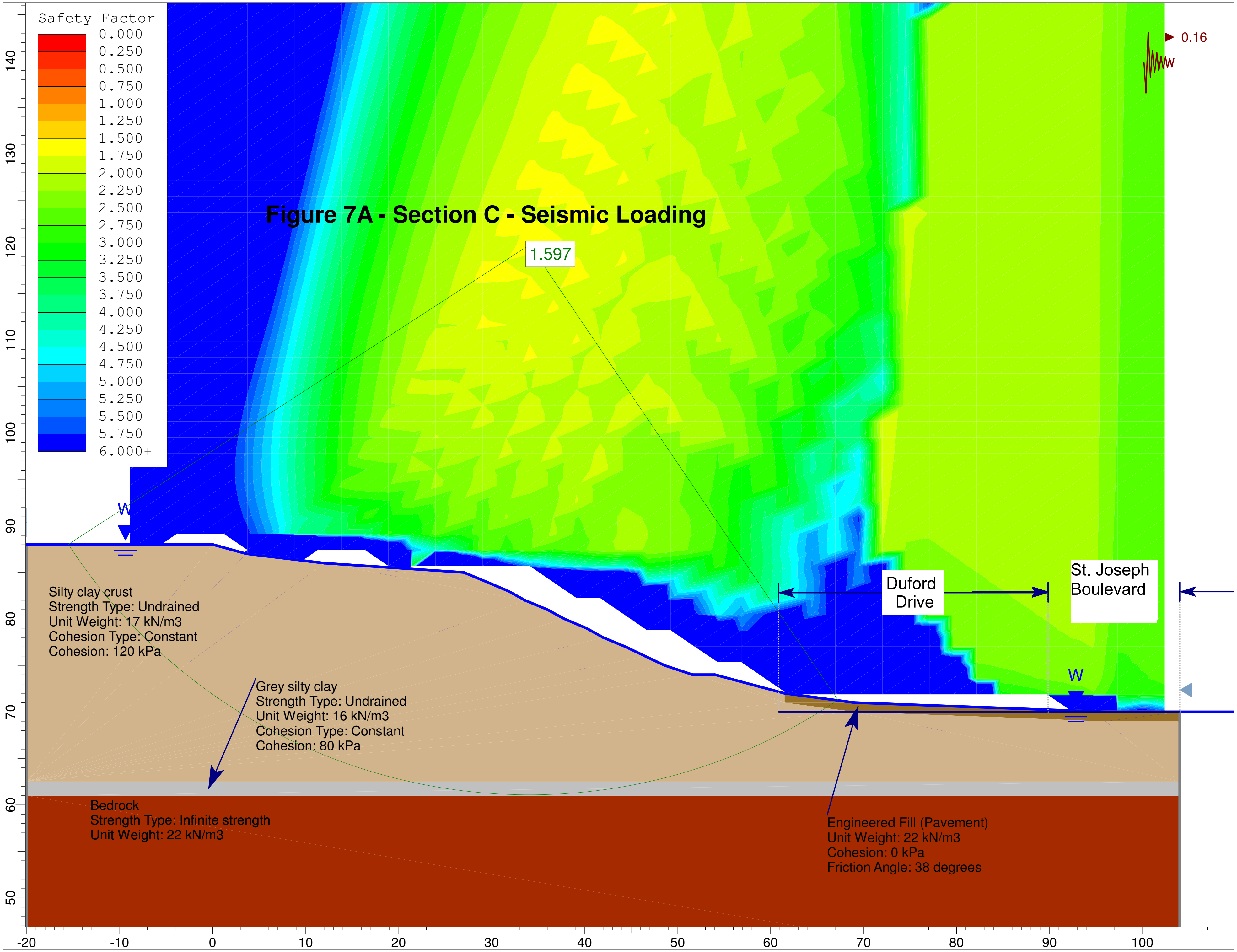


Figure 7A - Section C - Seismic Loading
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
1.597
0.16
W
Silty clay crust
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion Type: Constant
Cohesion: 120 kPa
Grey silty clay
Strength Type: Undrained
Unit Weight: 16 kN/m3
Cohesion Type: Constant
Cohesion: 80 kPa
Bedrock
Strength Type: Infinite strength
Unit Weight: 22 kN/m3
Engineered Fill (Pavement)
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees
Duford Drive
St. Joseph Boulevard
W
140
130
120
110
100
90
80
70
60
50
-20
-10
0
10
20
30
40
50
60
70
80
90
100

Historical Photographs, Aerial and Street View Images

Photo 1: Localized slope failure occurring in the mid 1960s adjacent to Duford Drive. Subject site is located within the foreground of the photograph. St. Joseph Boulevard is in the background. Ground surface adjacent to Duford Drive is noted to be free of vegetation, which is indicative that construction of the subject roadway section and cutting of the subject slope was recently completed. It is suspected that the exposed slope was re-shaped to an unstable slope angle as part of the construction work at that time.

Photo 2: Same localized slope failure, which occurred in mid 1960s.

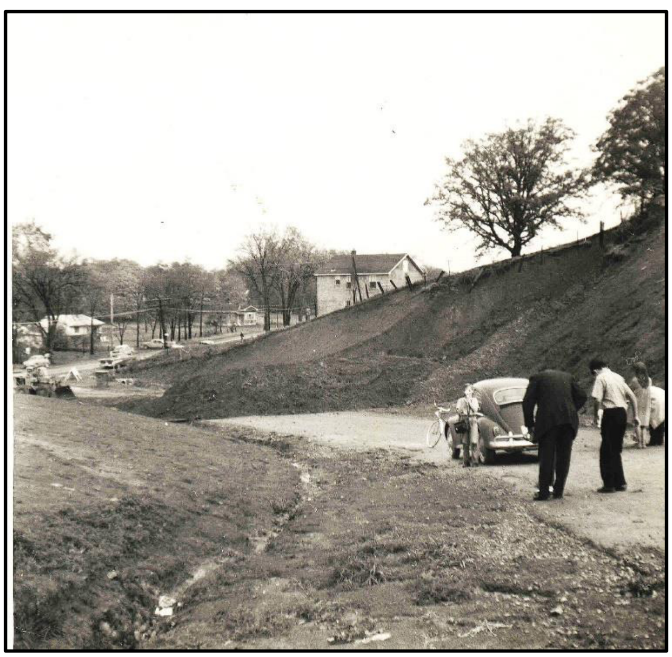


Photo 3: Street view image from Google Earth of former slope failure area (presented in Photos 1 and 2) adjacent to Duford Drive. Ground noted to be re-shaped with a terraced slope in front of reinstated slope.

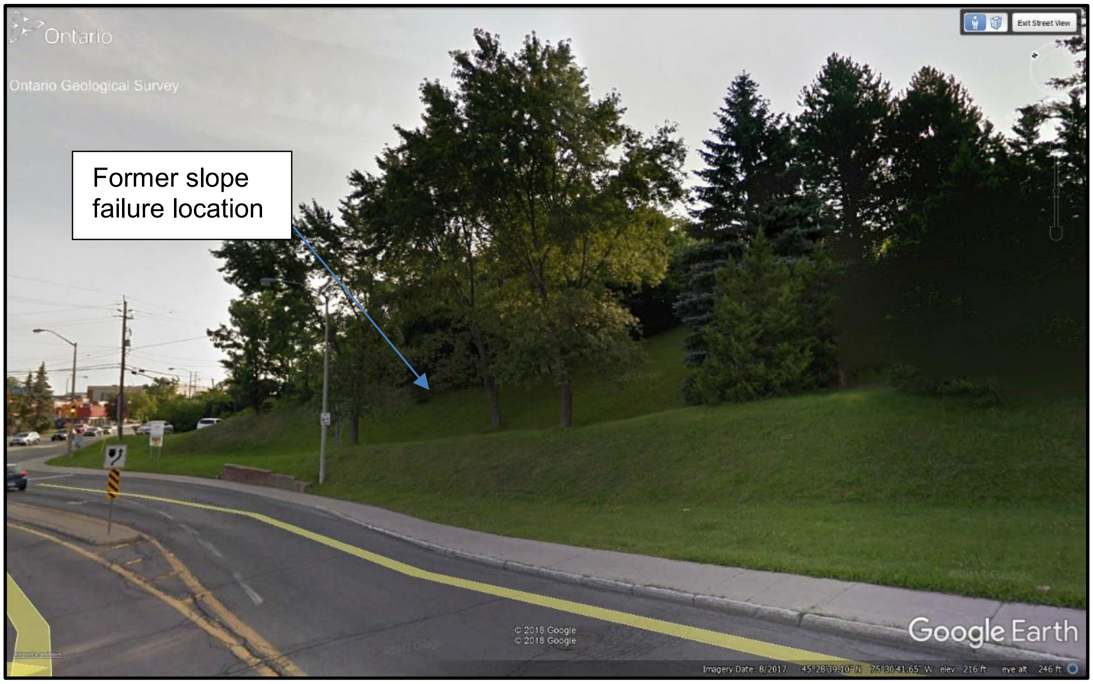
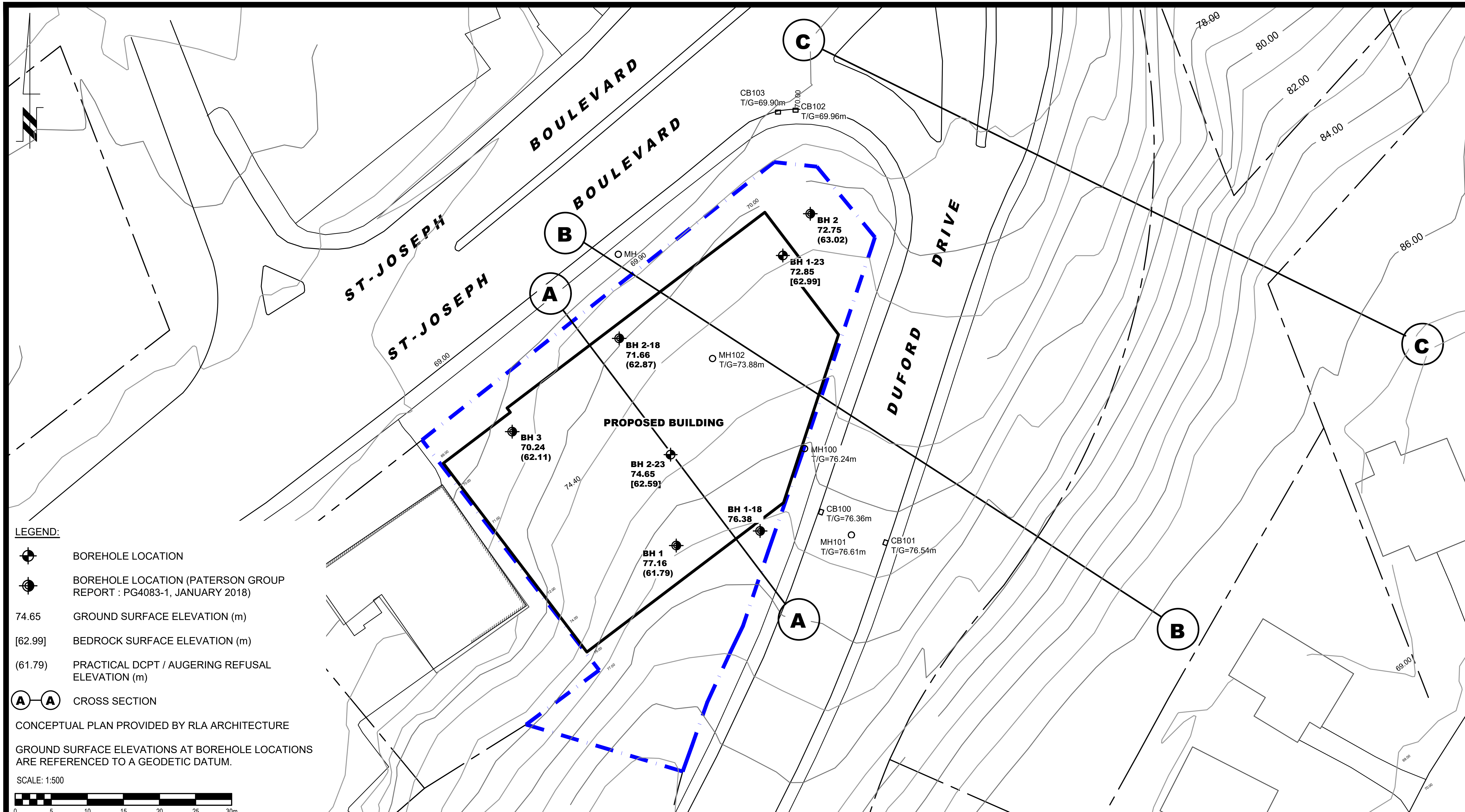


Photo 4: Street view image from Google Earth of the same former slope failure area presented in Photos 1 and 2. The ground surface is noted to be stable with no signs of slope instability.

Photo 5: Area of former slope failure noted in Photos 1 and 2. Subject site is property along the right side of the photograph.



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE LOCATION (PATERSON GROUP REPORT : PG4083-1, JANUARY 2018)
74.65	GROUND SURFACE ELEVATION (m)
[62.99]	BEDROCK SURFACE ELEVATION (m)
(61.79)	PRACTICAL DCPT / AUGERING REFUSAL ELEVATION (m)
A—A	CROSS SECTION

CONCEPTUAL PLAN PROVIDED BY RLA ARCHITECTURE

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:500

0 5 10 15 20 25 30m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

VUZE CONSTRUCTION

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
222 DUFORD DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 05/2023
Drawn by: YA	Report No.: PG6609-1
Checked by: AB	Dwg. No.: PG6609-1
Approved by: JV	Revision No.:

p:\autocad drawings\geotechnical\pg66xx\pg6609\pg6609-1-test hole location plan.dwg

11x17