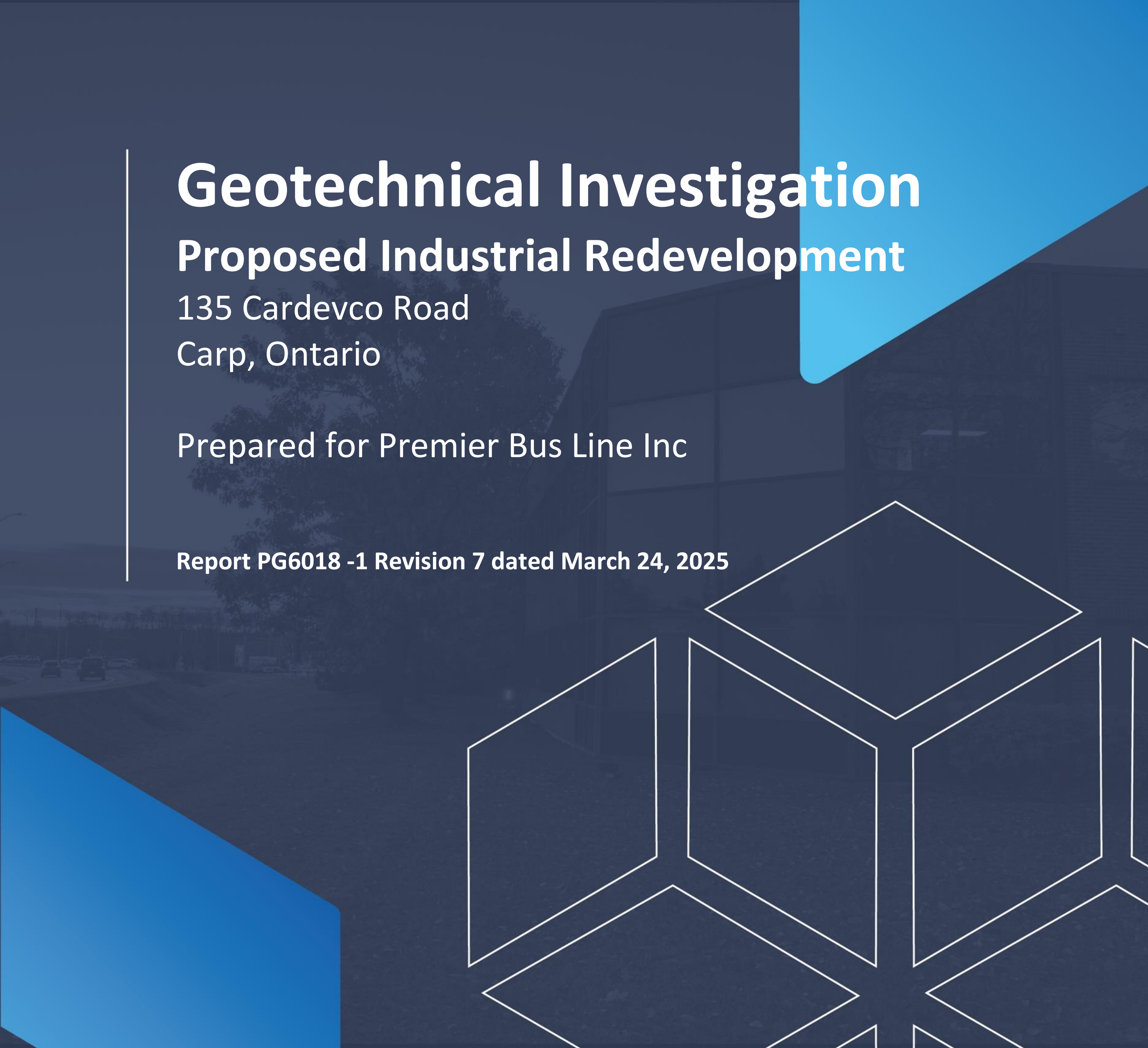


Geotechnical Investigation

Proposed Industrial Redevelopment

135 Cardevco Road
Carp, Ontario

Prepared for Premier Bus Line Inc

Report PG6018 -1 Revision 7 dated March 24, 2025

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Premier Bus Lines Inc. to conduct a geotechnical investigation for the proposed industrial redevelopment, located at 135 Cardevco Rd. Carp, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of test pits.

- ☐ 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

Based on the available drawings, it is understood that the proposed redevelopment will consist of demolishing the southern portion of the existing slab-on-grade warehouse structure present at the subject site, and constructing a new smaller warehouse addition. It is further understood that the northern portion of the existing warehouse will remain. The rest of the site will remain as an asphalt covered parking.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was conducted on November 12, 2021. At that time, a total of five (5) test pits were advanced to a maximum depth of 3.5 m below the existing ground surface. Two test pits were excavated within the vicinity of the proposed southern warehouse addition. The remaining three test holes were excavated adjacent to the exterior footings of the northern portion of the existing warehouse, in order to confirm founding conditions and to allow for scanning foundation rebars. Further, an additional test pitting program was completed on July 9, 2024, to complete in-situ infiltration testing at the subject site for hydrogeological purposes. At that time, two (2) test pits were excavated to a maximum depth of 1.7 m below the existing ground surface.

It should be noted that based on the available drawings and our field observations at the time of our geotechnical investigation, the subject site was occupied by an existing warehouse building. Furthermore, it is understood that the proposed warehouse addition will be constructed within the southern portion of the existing building, to replace and existing portion of the warehouse which is to be demolished. Therefore, no test holes were completed within the footprint of the proposed building addition due to limited access. Therefore, the test hole locations were selected by Paterson and distributed in a manner to provide general acceptable coverage of the subject site taking into consideration existing site features (i.e., the existing building at the location of the proposed building addition) and underground utilities and the nature of the proposed redevelopment (building addition within a predeveloped area) and our extensive knowledge of the soils within the subject area based on our geotechnical experience.

Based on that, the proposed program consisting of test holes within the subject site is considered adhering to the City of Ottawa guideline for building addition, and the number and depth of the excavated test holes completed on site are sufficient from a geotechnical perspective to provide information regarding the subgrade conditions at founding level for the proposed building addition.

The locations of the boreholes are shown on Drawing PG6018-1 - Test Hole Location Plan included in Appendix 2.

All test pits were excavated using a backhoe. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets presented in Appendix 1 of this report.

Sampling and In Situ Testing

Grab samples were recovered from the excavated tests pits at the time of the investigation. The samples were classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the grab samples were recovered from the test pits are shown as G, on the Soil Profile and Test Data sheets in Appendix 1.

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

At the time of the investigations, groundwater observations were recorded in the open-hole test pits during the fieldwork. The groundwater infiltration depth observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed warehouse addition, taking into consideration the existing site features and underground utilities and the nature of the proposed building addition in a predeveloped area as well as our experience from adjacent sites. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson personnel using a GPS and referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG6018-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined on site and in our laboratory to review the results of the field logging. Moisture contents completed on grab samples collected from the test pits were also conducted. The results of the moisture content testing have been included in the Soil Profile and Test Data Sheets enclosed in Appendix 1.

Consolidation and Undrained Shear Strength

Paterson reviewed the proposed grading for the subject site from a geotechnical perspective. Based on our review, it is understood that no significant grade raise is intended for the subject site. Furthermore, based on the encountered subsurface conditions at the subject site and discussed comprehensively under subsection 4.2-Subsurface Profile, and our knowledge in the area from previous investigations on 142 and 158 Cardevco Road (Borehole logs included in Appendix I), silty clay is not encountered at the subject site. Therefore, consolidation and undrained shear strength testing are not required from a geotechnical perspective and the site will not be subject to a permissible grade raise restriction.

Physical Soil Properties

Paterson did not conduct grain size analysis or unit weight measurement of the encountered material. However, identification of the relevant geotechnical engineering properties for the encountered soils were completed based on visual observations of the existing soils in the current test pits in general accordance with the procedures described in “ASTM D2488 – Standard Practice for description and identification of Soils (Visual-Manual Procedure)” which provides standard acceptable procedures for description of soils for engineering purposes based on visual examination and manual tests. Based on our visual soil descriptions, representative estimated typical values for the relevant geotechnical parameters (relative density, unit weight, permeability, friction angle) were selected. The results of our visual classification of the soil in general accordance with the above noted procedure as well as the estimated relevant geotechnical parameters and applicable references are discussed further under section 4.2 below.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing warehouse building with an asphalt and granular driveway and parking lot, and landscaped areas occupying the remainder of the site.

The subject site is bordered by commercial and/or industrial properties on all sides, and by Cardevco Road to the Northeast. The existing ground surface across the subject site is relatively flat with an approximate geodetic elevation of 118.5 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations was observed to consist of asphalt or topsoil overlying a fill layer consisting of crushed stone and brown silty sand with gravel and occasional cobbles and trace asphalt. A 560 mm thickened edge slab, underlain by 100 mm thick rigid insulation layer was observed to extend from ground surface at the locations of TP 1-21, TP 2-21, and TP 3-21. 300 mm of crushed stone were encountered below the rigid insulation layer, within Test Pit TP 2-21. Compact to dense brown silty sand with some gravel and cobbles and occasional boulders was encountered underlying the rigid insulation or fill materials at all borehole locations except TP 5-21 where the crushed stone layer was found to be underlain by very dense glacial till. Refusal to excavation on very dense glacial till was encountered in TP 5-21 at a depth of 2.2 m below ground surface.

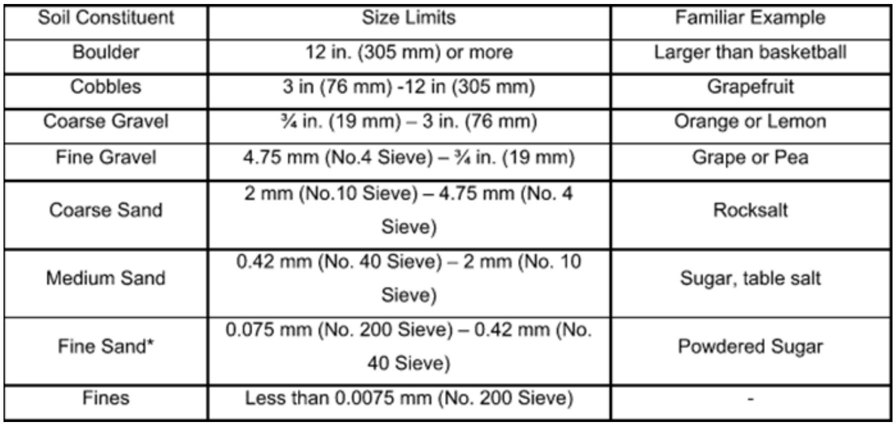
The test pits were terminated at depths between 1.6 m and 3.5 m below the existing ground surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Based on available geological mapping, the subject site is underlain by Paleozoic limestone of the Bobcaygeon Formation with an overburden thickness of 5 to 10 m.

Shearing Resistance

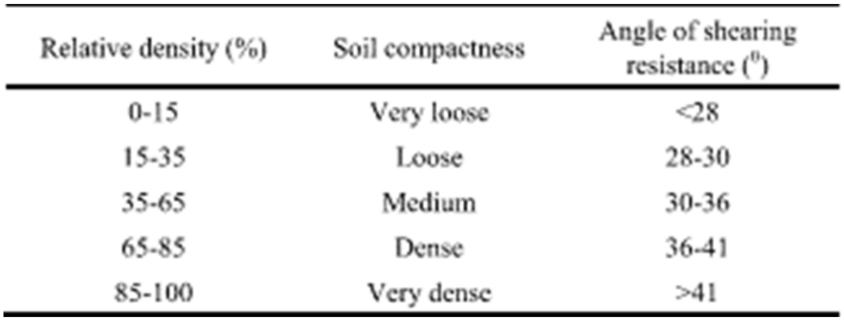
Based on our visual observation of the recovered soil samples and the sidewalls of the excavated test pits which appeared to be vertically stable with no signs of significant sloughing, the encountered formation at the majority of the test pit locations with the exception of TP 5-21, was observed to mainly consist of silty sands based on criteria for grain size distribution in section 3.1.5 to 3.1.9 in the above noted ASTM D2488, and utilizing the ranges specified below:



Soil Constituent	Size Limits	Familiar Example
Boulder	12 in. (305 mm) or more	Larger than basketball
Cobbles	3 in (76 mm) -12 in (305 mm)	Grapefruit
Coarse Gravel	¾ in. (19 mm) – 3 in. (76 mm)	Orange or Lemon
Fine Gravel	4.75 mm (No.4 Sieve) – ¾ in. (19 mm)	Grape or Pea
Coarse Sand	2 mm (No.10 Sieve) – 4.75 mm (No. 4 Sieve)	Rocksalt
Medium Sand	0.42 mm (No. 40 Sieve) – 2 mm (No. 10 Sieve)	Sugar, table salt
Fine Sand*	0.075 mm (No. 200 Sieve) – 0.42 mm (No. 40 Sieve)	Powdered Sugar
Fines	Less than 0.0075 mm (No. 200 Sieve)	-

Figure 1. Description of grain size distribution based on visual observations (Professor Krishna Reddy-UIC, NA)

Furthermore, most samples could not be broken with finger pressure while some samples in the upper soil layers appeared to slightly "break into pieces or crumble with moderate finger pressure", indicating a medium to high dry shear strength in accordance with Table 9 of ASTM D2488. Therefore, the silty sands encountered at the subject site are characterized as compact (medium) to dense. Based on the published relative density for such soils, the angle of shearing resistance for the compact to dense soil is taken as 35o, which is typical for this type of soil, as provided in Roy and Bhalla (2017).



Relative density (%)	Soil compactness	Angle of shearing resistance (^{0})
0-15	Very loose	<28
15-35	Loose	28-30
35-65	Medium	30-36
65-85	Dense	36-41
85-100	Very dense	>41

Figure 2. Characteristics of soils based on relative density (Roy and Bhalla (2017)

4.3 Groundwater

Groundwater infiltration depths were observed during the investigations on November 12, 2021, and July 9, 2024, within the walls of the excavated test pits. The observed depth of infiltration is presented in Table 2 below.

Table 2 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
TP 1-24	118.66	1.70	116.96	July 9, 2024
TP 2-24	118.62	1.60	117.02	
TP 1-21	118.94	Dry	N/A	November 12, 2021
TP 2-21	118.94	Dry	N/A	
TP 3-21	118.83	Dry	N/A	
TP 4-21	119.06	2.00	117.06	
TP 5-21	118.60	1.90	116.70	

Note: The ground surface elevation at each test pit location was surveyed using a handheld GPS using a geodetic datum.

The variability in ground surface elevations across the site influences groundwater depth measurements, as such the groundwater elevations have been reported as geodetic values above sea level (ASL). The groundwater elevations measured in July 2024 ranged between the maximum and minimum geodetic groundwater levels recorded in November 2021. The groundwater elevations encountered onsite on November 12, 2021, and July 9, 2024, ranged from the geodetic elevation of 116.70 to 117.06 m ASL.

Based on the observed groundwater elevations and infiltration depths, the long-term groundwater table is anticipated to range between the geodetic elevation of 117.0 m and 116.0 m ASL. The value provided for the long-term groundwater table does not account for seasonal high groundwater levels. The recorded groundwater infiltration depths are provided in the applicable Soil Profile and Test Data sheet in Appendix 1.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed warehouse addition. It is highly recommended that the finish floor level for the proposed warehouse addition match that for the existing northern warehouse portion to remain. As revealed from the excavated test pits TP 1-21, TP 2-21, and TP 3-21, it was determined that the northern portion of the warehouse to remain is founded on a concrete thickened edge slab and that the USF for the existing slab is at approximate geodetic elevation 118.4m.

The lateral support zone of 1.5H:1V for footings should be protected. Upon our review of the grading plans for the proposed building addition, it was noted that the proposed USF will be at geodetic elevation 117.4m. Therefore, underpinning of the eastern footings along the existing building will be required to complete the proposed excavation. Details regarding the underpinning program are discussed further under subsection 6.3 – Excavation Side Slopes.

The proposed warehouse addition can be founded on conventional shallow spread footings placed on undisturbed, compact to dense brown silty sand bearing surface.

Due to demolition of the existing warehouse, concrete removal is anticipated at the subject site.

Based on the encountered subsurface conditions at the subject site, the proposed building addition is not subject to a permissible grade raise restriction.

Retaining walls are proposed at the subject site. Therefore, Paterson completed a global and external stability analysis for the proposed retaining walls. Geotechnical soil parameters have also been provided to assist in the retaining wall design from a geotechnical perspective.

Where the footing subgrade consists of silty sand which is observed to be in a loose state of compactness, the material should be proof compacted using suitable vibratory equipment making several passes under dry conditions and above freezing temperatures and which is approved by Paterson at the time of construction.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant organic materials, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures.

It should be noted that the existing fill layer, where noted to be free of significant amounts of deleterious and organic materials, can be left in place below parking areas and access lanes provided that a proof-rolling program is witnessed and approved by Paterson personnel at the time of construction. Any poor performing areas should be removed and reinstated with an approved granular fill.

Fill Placement

Fill used for grading beneath the proposed development should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and paved areas should be compacted to at least 98% of the material’s standard Proctor maximum dry density (SPMDD), unless noted otherwise throughout this report.

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. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material’s SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Terraxx.

5.3 Foundation Design

Shallow Foundation

Footings placed directly on an undisturbed, compact to dense silty sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **125 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **200 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted 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 a soil 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. The settlement was estimated based on the theory of elasticity.

Permissible Grade Raise

As discussed earlier, site silty clay is not encountered at the subject site. Therefore, the site will not be subject to a permissible grade raise restriction.

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 compact to dense silty sand bearing surface above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher bearing capacity as the bearing medium soil. Based on our field observations, the existing USF for the thickened edge slab was encountered at an approximate geodetic elevation 118.4m. However, the proposed USF for the building addition will be at geodetic elevation 117.4m. Therefore, the lateral support zone of the existing footings along the eastern wall of the existing warehouse should be protected. Consideration can be given to underpinning the existing footings along the impacted foundation wall. The underpinning should be done by a specialist, and it should extend down to the proposed USF elevation. Further details are provided under subsection 6.3- Excavation Side Slopes.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. Soils underlying the subject site are not susceptible to liquefactions. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

Liquefaction Potential

It is anticipated that the design USF for the proposed building addition will be at geodetic elevation 117.4 m. Therefore, the footings will be placed on the unsaturated undisturbed compact to dense brown silty sand (which corresponds to SPT blows of 10 to 30). Furthermore, based on available coverage from nearby sites on 142 and 158 Cardevco Road (refusal to excavation on inferred bedrock was confirmed at shallow depths between 1 to 3 m indicating a relatively limited thickness of the silty sand formation in the area. Based on the observed compactness and the relatively limited thickness of the overburden soils, the Cyclic Resistance Ratio (CRR7.5) will be greater than the Cyclic Stress Ratio which is governed by the effective overburden pressure, across the subject site. Considering a conservative 7.5 magnitude earthquake, (MSF=1) and the potential for liquefaction equation below (Youd et al, 2001), the Factor of Safety is greater than 1 across the site.

$$FS = \left(\frac{CRR_{7.5}}{CSR}\right) MSF$$

where CRR7.5 = Cyclic Resistance Ratio at Magnitude 7.5 Earthquake
CSR = Cyclic Stress Ratio
MSF = Magnitude Scaling Factor

The CRR is determined by a standard penetration test blow count, $(N1)_{60}$, normalized to an overburden pressure of approximately 100 kPa and a hammer energy ratio of 60%. A conservative estimate of 10 blows was considered for the compact silty sand formation encountered at the subject site. The following equations presented below, developed by the U.S. National Center for Earthquake Engineering Research, were used to derive the $CRRM$=7.5 value. The calculated CRR value was **0.113**.

$$100 * CRR_{M=7.5} = \frac{95}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{1.3} - \frac{1}{2}$$

The CSR was also calculated using the formula presented below and the value ranged between **0.087 and 0.104** at the hypothetical depths of the liquefaction occurrence, 1.5 to 3 m, respectively.

$$CSR = 0.65 * \frac{a_{max}}{a} * \frac{\sigma_{vo}}{\sigma'_{vo}} * \gamma_d$$

Where a_{max} = maximum peak horizontal acceleration generated by the earthquake (0.32g)
g = acceleration due to gravity (9.81 m/s^2)
σ_{vo} = total vertical overburden stress at the depth of interest (kN/m^2)
σ'_{vo} = effective vertical overburden stress at the depth of interest (kN/m^2)
γ_d = stress reduction factor which accounts for soil flexibility

The factor of safety against liquefaction (FS_{liq}) for the subject site was determined to be **1.1 to 1.3**. It is generally accepted that if FS_{liq} < 1.0, liquefaction is assumed to occur while FS_{liq} > 1.0 indicates no liquefaction. Therefore, buildings supported on the encountered subsurface formation at the subject site are not susceptible to liquefaction.

5.5 Slab on Grade Construction

With the removal of all topsoil and deleterious fill within the footprint of the proposed warehouse, the native soil subgrade and or existing crushed stone fill, proof-rolled with a suitably sized vibratory roller making several passes under dry and above freezing conditions, reviewed and approved by Paterson personnel at the time of construction, will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction.

It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of the materials SPMDD.

5.6 Pavement Design

Car only parking areas, access lanes and loading areas are anticipated at this site. The proposed pavement structures are shown in Tables 3 and 4.

Table 3 - Recommended Flexible Pavement Structure – Car Only Parking Areas

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

Table 4 - Recommended Flexible Pavement Structure – Access Lanes and Heavy Truck Parking Areas

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 fill, in situ soils or bedrock or OPSS Granular B Type I or II material placed over in situ soil, bedrock or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granulars (base and subbase) should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

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

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

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 warehouse addition. The system should consist of a 150 mm diameter perforated corrugated plastic pipe wrapped in a geosock, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The clear stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pump pit.

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 Terraxx, 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 Against Frost Action

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

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations). Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

Unsupported Excavations

The excavations for the proposed warehouse will be mostly through crushed stone, silty sand, or glacial till. The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level.

The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress. Excavation side slopes should also be protected from erosion by surface water and rainfall events by the use of tarpaulins or other means of erosion protection along their footprint.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Underpinning

Based on our review of the grading plans, and on information collected from our geotechnical investigation, it is understood that the USF for the proposed building addition and the eastern foundation wall are at approximate geodetic elevation 117.4 and 118.4 m, respectively. Furthermore, there will be no horizontal setback between the proposed building addition and the eastern foundation wall of the existing warehouse. Therefore, the underpinning of the eastern foundation wall of the existing warehouse should be underpinned.

- ☐ The underpinning program should be completed in sections (panels) by excavating each panel individually in a piano key fashion to maintain adequate lateral support for the existing footings.
- ☐ A maximum 1.0 m horizontal spacing is required between each excavated panel.
- ☐ The maximum height of excavation per stage is 1.0 m.
- ☐ Each panel should be excavated using suitable excavation equipment and infilled with a minimum 15 MPa (28-day compressive strength) concrete once the forms are secured in place. Concrete infilling will be done through the cored holes in the floor slab.
- ☐ For each excavated panel, place 0.75 to 1.0 m forms below the top of the existing footings down to the bottom of the excavation at each stage. The forms should be firmly secured in place prior to pouring concrete.
- ☐ Once the concrete in the first set of panels has set (12 to 24 hours), the second set of panels can be completed. The process is then repeated in consecutive order to maintain adequate lateral support during the duration of the underpinning program.
- ☐ In cold weather conditions the concrete should be sufficiently protected with insulated tarps, until the concrete attains its design strength.
- ☐ The subsequent courses of panels should be offset from the previous course.
- ☐ The underpinning program should extend down to the USF elevation of the proposed building addition.

Further details regarding the proposed underpinning program are provided in Figures 3 and 4 in appendix 2.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe.

Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's SPMDD.

It should generally be possible to re-use the site-excavated material above the cover material if the excavation and filling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement.

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

Based on our observations, 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. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

6.6 Winter Construction

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means.

In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations should be carried in a manner to avoid the introduction of frozen materials, snow, or ice into the trenches.

6.7 Corrosion Potential and Sulphate

The results of analytical testing taken on sample G3 from TP2-21 show that the sulphate content is 9 ug/g which 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 was found to be <5 ug/g which is less than 0.2 % and the pH was measured to be 7.27 which both indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site. However, the measured resistivity (14700 ohm.cm which is greater than 4000) is indicative of a non-aggressive to slightly aggressive corrosive environment according to the well known procedures by A.B. Chance Company Bulletin 01-9204, Miller and Al, and Fisher and Bue.

6.8 Retaining Walls

Bearing Capacity

Retaining wall Footings placed directly on an undisturbed compact to dense silty sand, very dense glacial till, or on approved engineering fill placed on undisturbed compact to dense silty sand or very dense glacial till bearing surface undisturbed, can be designed using a bearing resistance value at serviceability limit states (SLS) of **125 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **200 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Slope Stability

The proposed retaining wall was subjected to external global stability analysis. The acceptable criteria for a sufficient retaining wall system requires the wall to achieve a factor of safety of 1.5 and 1.1 under static and seismic loadings, respectively. The applicable seismic design incorporates a PGA of 0.32, as per NBCC 2015.

The global stability analysis was modeled using SLIDE, a computer program which permits a two-dimensional slope stability analysis calculating several limit equilibrium methods, which are widely accepted slope analysis methods.

The program calculates a factor of safety, which represents the ratio of the forces resisting failure to forces 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 subsurface soil and groundwater conditions, a factor of safety greater than 1.0 is generally required for the failure risk to be considered acceptable.

A minimum factor of safety of 1.5 is generally recommended for conditions where the slope failure would comprise permanent structures. An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16 g was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The retaining wall section was reviewed using the design loading according to CHBDC 2015.

One critical retaining wall cross-section at Wall 8 was studied as the worst-case scenario (refer to Figure 2-Markup Plan for cross-section location). The height of the remaining walls was found to be less than 1m. The following parameters were used for the slope stability analysis under static and seismic conditions.

Table 5 – Total and Effective Soil Parameters for Slope Stability

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Existing Fill	18	30	1
OPSS Granular B Type II	21	38	0
Compact Silty Sand	20	35	0
Glacial Till	21.5	36	1

It should be noted that the retaining wall is anticipated to be located in an area where the subsurface formation will consist of very dense glacial till. However, **conservatively**, the soil surrounding the retaining wall was considered to consist of compact to dense silty sand in the global slope stability analysis.

Analysis Results

The factor of safety for the retaining wall section, reflecting the geotechnical recommendations included below, was calculated to be 3.024, which is greater than 1.5 for static conditions as presented in the attached Figure 3A. Similarly, the results under seismic loading for the retaining wall section reflecting the geotechnical recommendations included below, yielded a factor of safety equal to 1.806, which is greater than 1.1 as presented in the attached Figure 3B.

Based on these results, the proposed retaining wall design is considered acceptable from a geotechnical perspective provided the recommendations mentioned below are implemented.

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. 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 placed in maximum 300 mm loose lifts and compacted to a minimum of 98% of the material's SPMDD.

Where there is a limited setback between the proposed retaining wall and the property line, 19 mm clear crushed stone may be used as alternative backfill material. To prevent soil migration and clogging, a non-woven geotextile fabric should be installed between the native soil and the clear stone. The clear crushed stone should be placed in maximum 300 mm loose lifts and compacted using a vibratory plate compactor to ensure proper compaction.

All bedding and backfill materials should be placed under dry conditions and above freezing temperatures and approved by the geotechnical consultant at the time of construction.

6.9 Tree Planting Setback

Based on the findings of the current geotechnical investigation and our knowledge in the area, the soils encountered below ground surface and down to a depth of 3.5m below finish floor elevation of the proposed building addition at the subject site consist of silty sands. Also, sensitive marine clays were not encountered within the depth of influence. Therefore, and in accordance with the City of Ottawa Guidelines for Tree Planting in Sensitive Clay Areas, tree planting setback restrictions are not required at the subject site from a geotechnical perspective.

7.0 Recommendations

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

☐ Review of the grading and site servicing plans from a geotechnical perspective.

☐ 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 and placement of mud slabs.

☐ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Premier Bus Line 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.

LICENSED PROFESSIONAL ENGINEER
March 24, 2025
Z.W. AL-MOSELLY
100194385
PROVINCE OF ONTARIO

Zubaida Al-Moselly, Ph.D., P.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ☐ Premier Bus Line Inc. (email copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA SHEETS FROM NEIGHBOURING SITES

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

135 Cardevco Road
Ottawa, Ontario

EASTING: 345715.269 **NORTHING:** 5017134.412 **ELEVATION:** 118.66

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** 2024 July 9

FILE NO. PG6018

HOLE NO. TP 1-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	PIEZOMETER CONSTRUCTION
Ground Surface									
ASPHALTIC CONCRETE 0.06						0	118.66		
FILL: Compact brown silty sand with gravel, some asphaltic concrete 0.24		G	1						
GLACIAL TILL: Dense to very dense brown silty sand with gravel, cobbles and boulders		G	2						
		G	3						
		G	4						
						1	117.66		
1.70		G	5						
End of Test Pit Groundwater Infiltration Observed at 1.7 mbgs									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

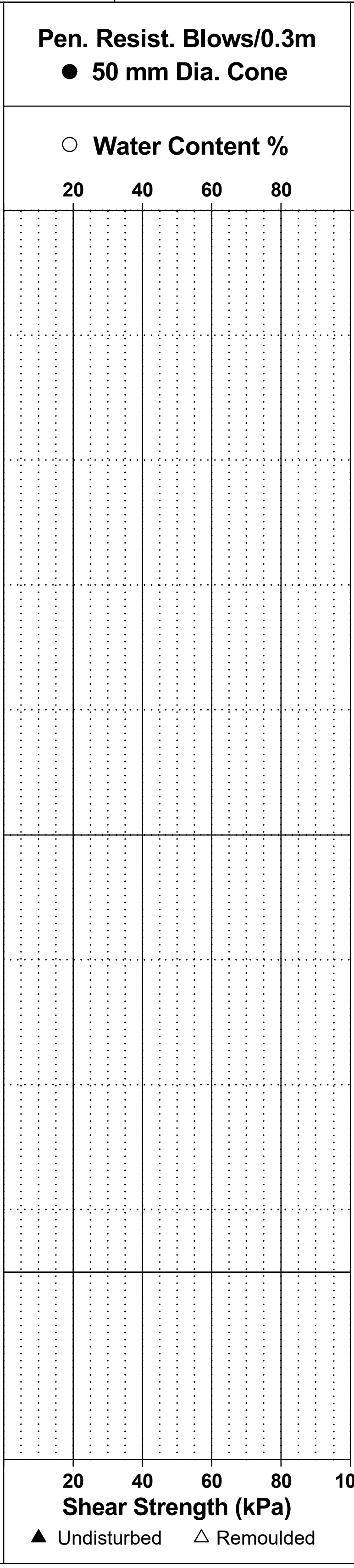
SOIL PROFILE AND TEST DATA

135 Cardevco Road
Ottawa, Ontario

EASTING: 345724.169 **NORTHING:** 5017117.07 **ELEVATION:** 118.62
DATUM: Geodetic
REMARKS:
BORINGS BY: Backhoe **DATE:** 2024 July 9

FILE NO. PG6018
HOLE NO. TP 2-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	PIEZOMETER CONSTRUCTION
Ground Surface						0	118.62	
ASPHALTIC CONCRETE 0.05								
FILL: Compact brown silty sand with gravel, some asphaltic concrete, trace clay 0.24		G	1					
GLACIAL TILL: Compact to very dense brown silty sand with gravel, cobbles and boulders - Boulder content increasing with depth		G	2					
		G	3			1	117.62	
1.70		G	4					
End of Test Pit Groundwater Infiltration Observed at 1.6 mbgs								



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Industrial Redevelopment - 135 Cardevco Road
Carp, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE November 12, 2021

FILE NO. PG6018

HOLE NO. TP 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	118.94
Asphaltic concrete 0.09							
FILL: Crushed stone 0.10		G	1				
FILL: Brown silty sand with crushed stone, gravel, occasional cobbles 0.60		G	2				
Rigid insulation 0.70							
Compact to dense, brown **SILTY SAND** 1.80		G	3			1	117.94
End of Test Pit							

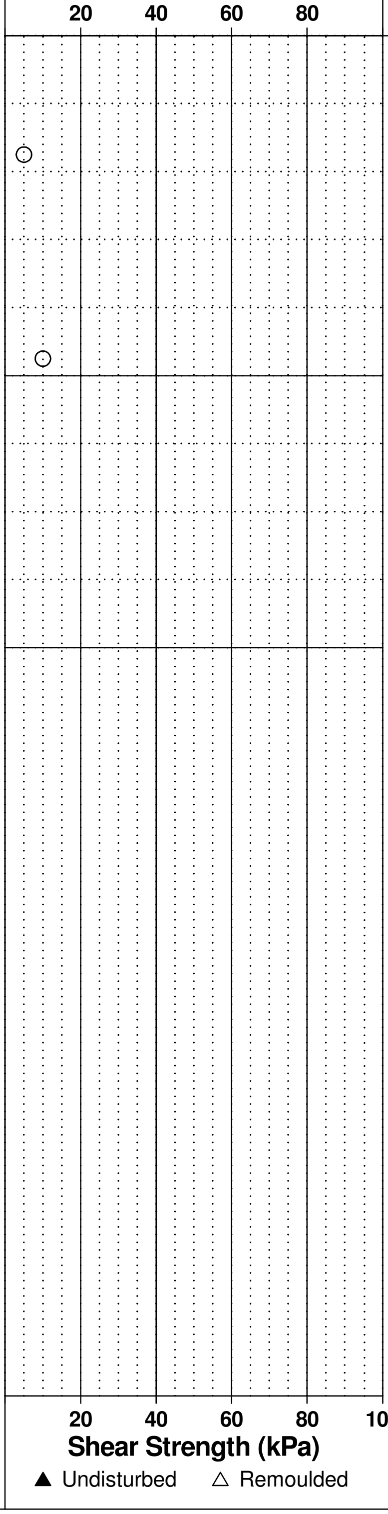
Bottom of thickened concrete slab encountered at 0.56m depth.

Underside of 100mm dia. PVC drainage pipe at 0.56m depth.

(TP dry upon completion)

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

○ **Water Content %**



Piezometer Construction

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Industrial Redevelopment - 135 Cardevco Road
Carp, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE November 12, 2021

FILE NO. PG6018

HOLE NO. TP 2-21

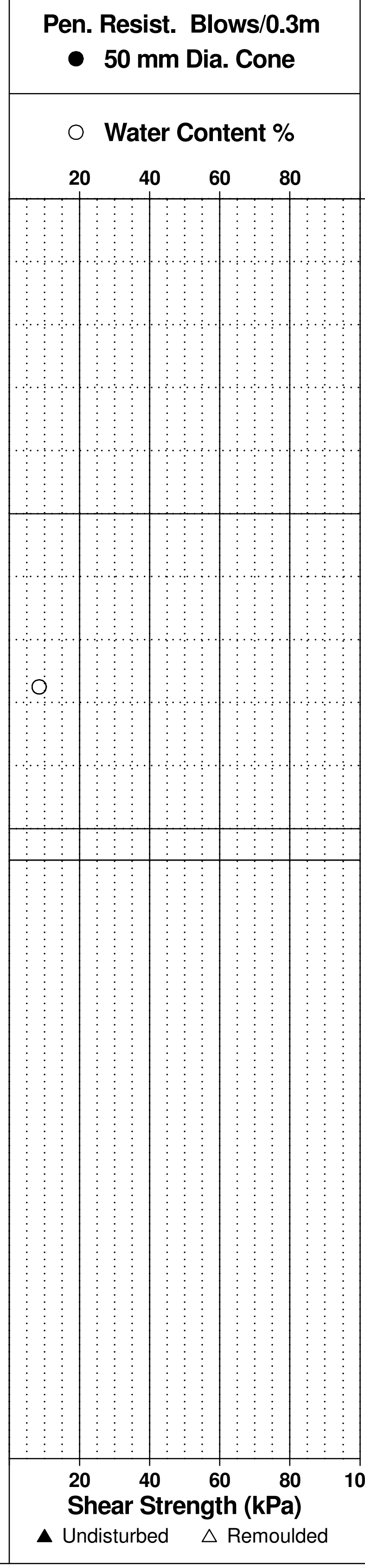
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
Ground Surface						0	118.94	
TOPSOIL 0.10								
FILL: Brown silty sand with crushed stone, gravel and cobbles, trace asphalt 0.60		G	1					
Rigid insulation 0.70								
FILL: Crushed stone 1.00		G	2			1	117.94	
Compact to dense, brown **SILTY SAND**		G	3			2	116.94	
2.10								

End of Test Pit

Bottom of thickened concrete slab encountered at 0.56m depth.

Underside of 100mm dia. PVC drainage pipe at 0.56m depth.

(TP dry upon completion)



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Industrial Redevelopment - 135 Cardevco Road
Carp, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE November 12, 2021

FILE NO. PG6018

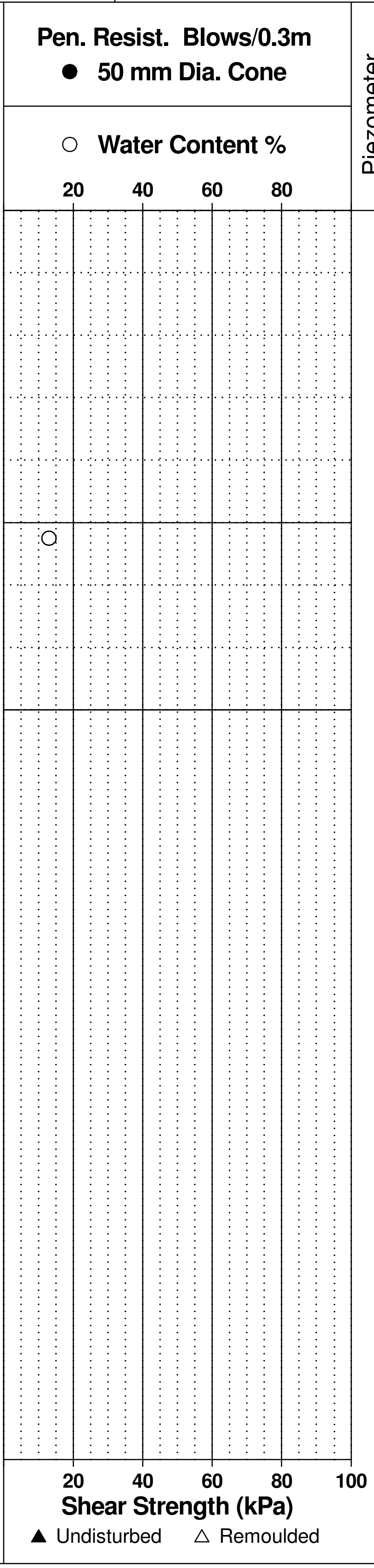
HOLE NO. TP 3-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	118.83
TOPSOIL 0.12							
FILL: Brown silty sand with crushed stone, gravel and cobbles, trace asphalt 0.60		G	1				
Rigid insulation 0.70							
Compact to dense, brown **SILTY SAND** 1.60		G	2			1	117.83
End of Test Pit							

Bottom of thickened concrete slab encountered at 0.56m depth.

Underside of 100mm dia. PVC drainage pipe at 0.56m depth.

(TP dry upon completion)



Piezometer Construction

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Industrial Redevelopment - 135 Cardevco Road
Carp, Ontario

DATUM Geodetic

REMARKS

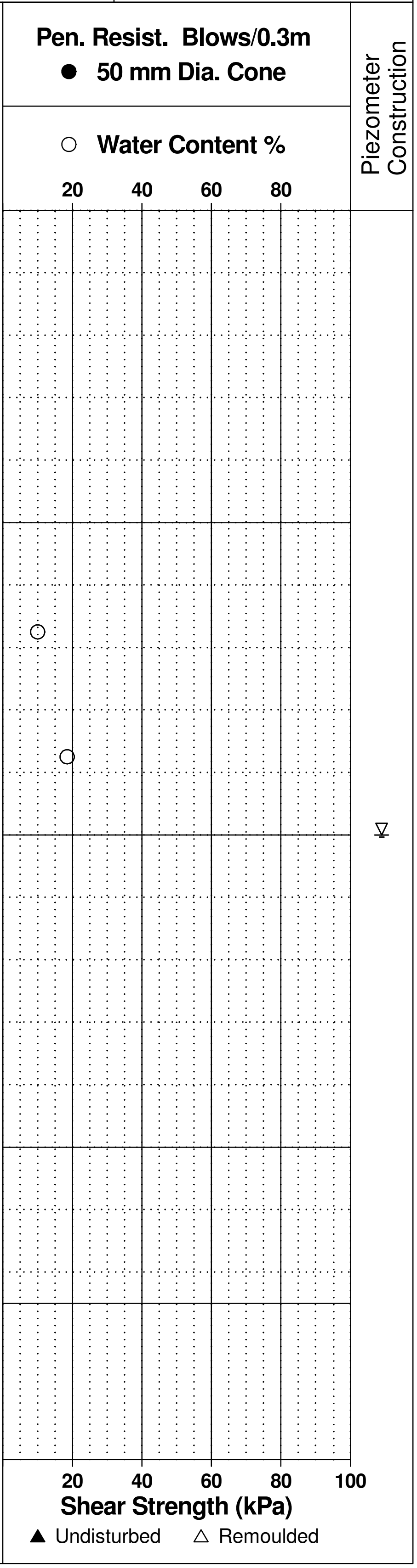
BORINGS BY Backhoe

DATE November 12, 2021

FILE NO. **PG6018**

HOLE NO. **TP 4-21**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	119.06
TOPSOIL 0.05							
FILL: Crushed stone and gravel 0.20		G	1				
FILL: Brown silty sand with gravel 0.40		G	2				
						1	118.06
		G	3				
Compact to dense, brown **SILTY SAND**		G	4				
						2	117.06
						3	116.06
3.50							
End of Test Pit (Groundwater infiltration at 2.0m depth)							



patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Industrial Redevelopment - 135 Cardevco Road
Carp, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE November 12, 2021

FILE NO. PG6018

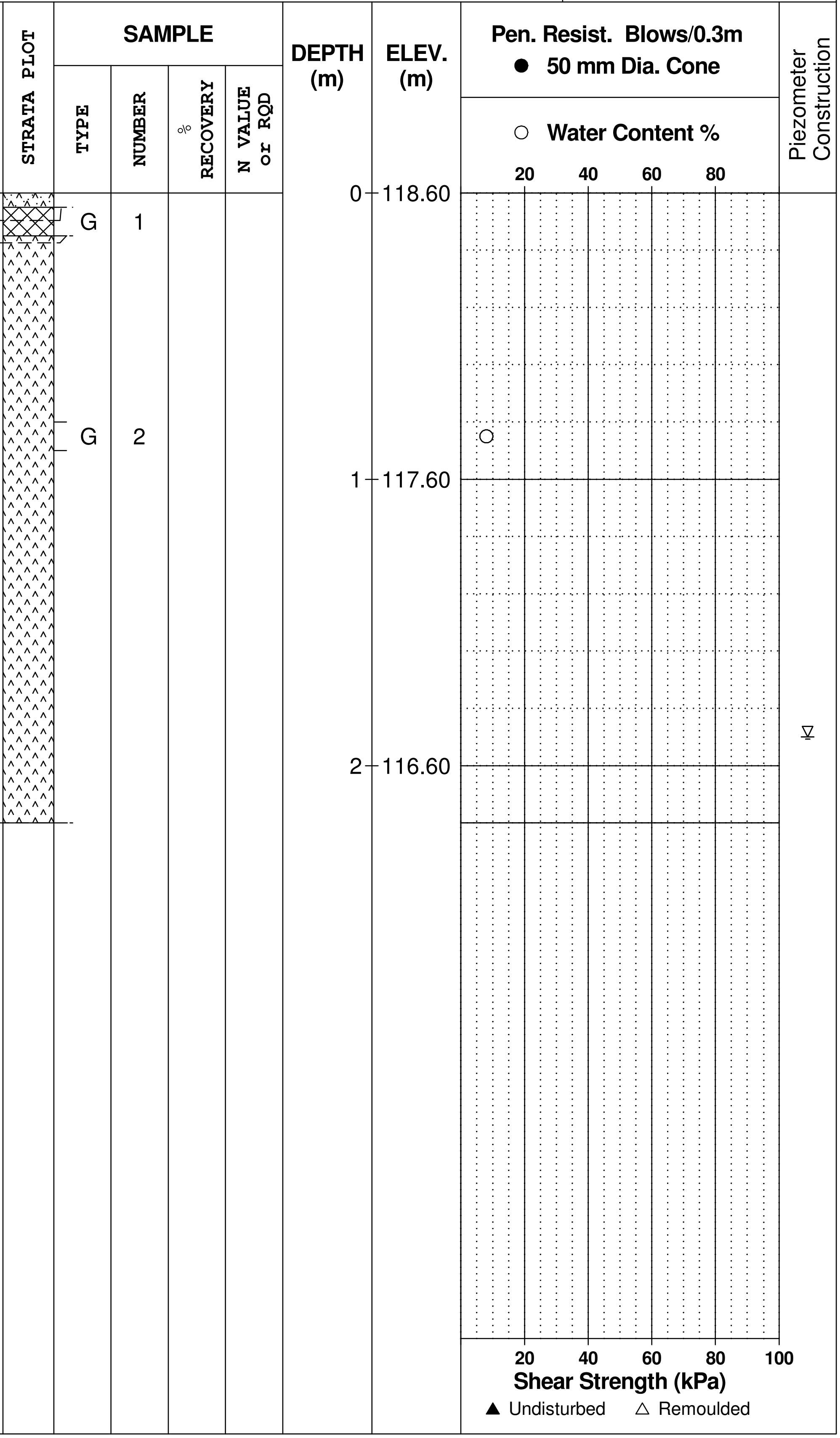
HOLE NO. TP 5-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	118.60
Asphaltic concrete 0.05						
FILL: Crushed stone 0.15	G	1				
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders	G	2			1	117.60
2.20					2	116.60

End of Test Pit

Practical refusal to excavation at 2.20m depth

(Groundwater infiltration at 1.9m depth)



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

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)

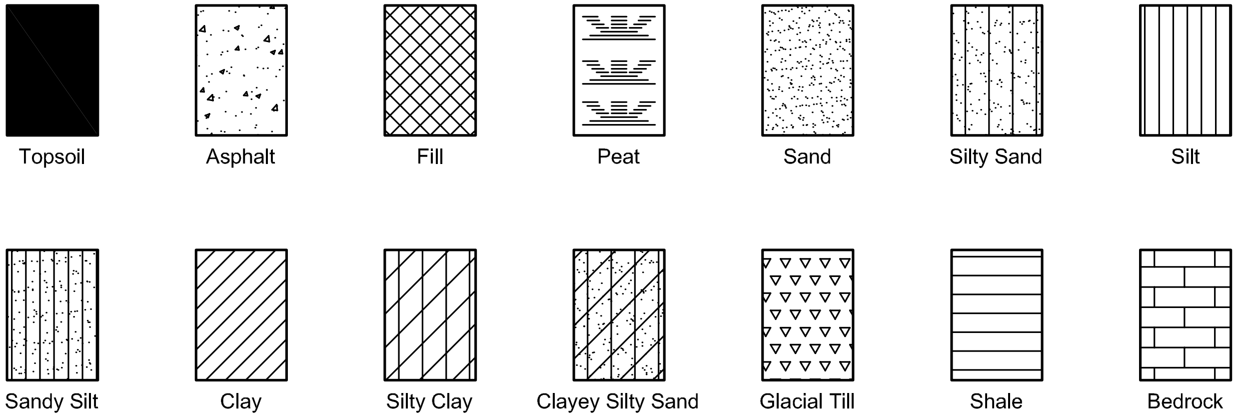
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

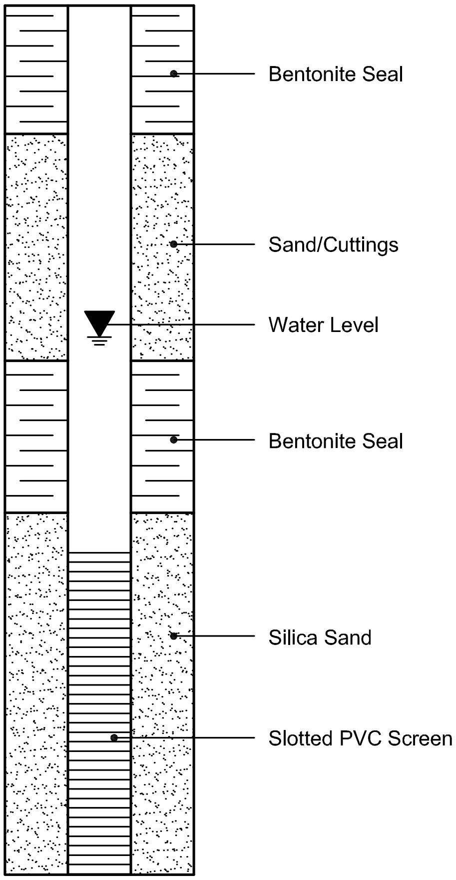
SYMBOLS AND TERMS (continued)

STRATA PLOT

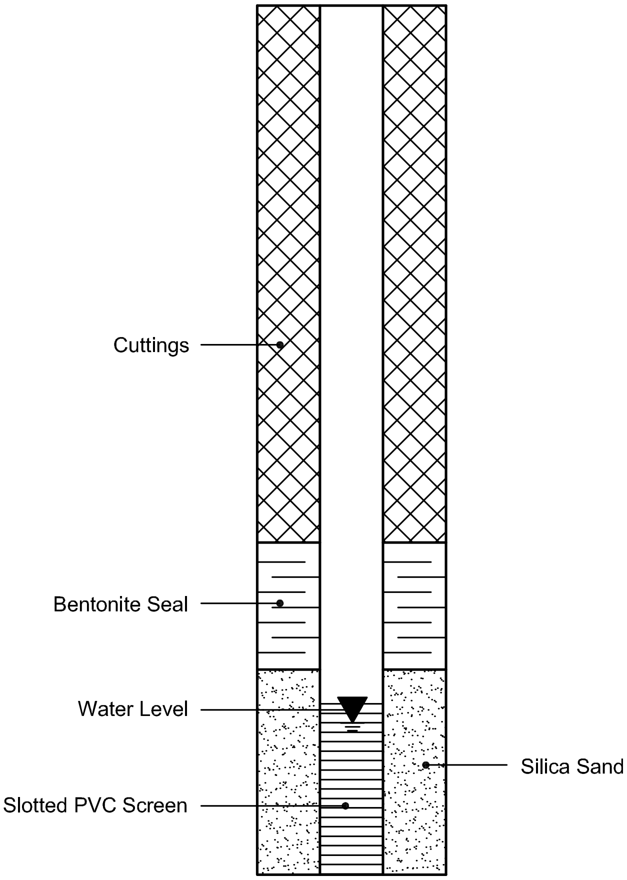


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 18-Nov-2021

Client: Paterson Group Consulting Engineers

Order Date: 12-Nov-2021

Client PO: 33413

Project Description: PG6018

	Client ID:	TP2-21 GS3	-	-	-
	Sample Date:	12-Nov-21 09:00	-	-	-
	Sample ID:	2146568-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	86.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.27	-	-	-
Resistivity	0.10 Ohm.m	147	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	9	-	-	-

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
142 Cardevco Road
Ottawa, Ontario

DATUM Finished floor elevation of existing building. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Excavator

DATE September 12, 2018

FILE NO. PG4672

HOLE NO. TP 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	99.04		
FILL: Compact, brown to grey sand and gravel, trace silt 0.46		G	1						
Compact, light brown fine **SILTY SAND** 0.91		G	2						
Compact brown **SAND** trace gravel and silt 1.07		G	3			1	98.04		
End of Test Pit Practical refusal to excavation at 1.07m depth (Groundwater not encountered at the completion of excavation)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
142 Cardevco Road
Ottawa, Ontario

DATUM Finished floor elevation of existing building. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Excavator

DATE September 12, 2018

FILE NO. PG4672

HOLE NO. TP 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	98.87		
FILL: Compact, grey to brown sand and gravel	▨	G	1						
0.76									
Compact, light brown fine **SILTY SAND**		G	2						
0.91									
Compact, light brown **SAND** trace silt						1	97.87		
		G	3						
1.52									
End of Test Pit Practical refusal to excavation at 1.52m depth (Groundwater not encountered at the completion of excavation)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
158 Cardevco Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE May 20, 2022

FILE NO. PG6233

HOLE NO. TP 1-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	117.44		
FILL: Dense to very dense brown silty sand with gravel and crushed stone	▨	G	1						
0.52									
FILL: Dense brown silty sand with gravel, crushed stone, asphalt and concrete	▨					1	116.44		
	▨	G	2						
1.71	▨	G	3						
End of Test Pit Refusal to excavation in dense fill at 1.71 m depth (Open hole GWL 1.6 m depth)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
158 Cardevco Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE 2022 May 20

FILE NO. PG6233

HOLE NO. TP 1A-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	117.44
FILL: Dense to very dense brown silty sand with gravel and crushed stone 0.52							
FILL: Dense brown silty sand with gravel, crushed stone, asphalt and concrete 2.02						1	116.44
Compact brown **SILTY SAND** with gravel, trace cobbles 2.31		G	1			2	115.44
End of Test Pit Refusal to excavation on bedrock surface at 2.31 m depth (Open hole GWL 1.6 m depth)							

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

○ **Water Content %**

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
158 Cardevco Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Excavator

DATE 2022 May 20

FILE NO. PG6233

HOLE NO. TP 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	117.58		
FILL: Dense brown silty sand with gravel and crushed stone	▨	G	1						
0.55									
FILL: Compact brown silty sand with gravel, crushed stone and brick fragments	▨					1	116.58		
		G	2						
						2	115.58		
2.20									
Compact brown **SILTY SAND** with gravel, trace cobbles		G	3						
						3	114.58		
3.12									
End of Test Pit Refusal to excavation on bedrock surface at 3.12 m depth (Open hole GWL at 1.8 m depth)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
158 Cardevco Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

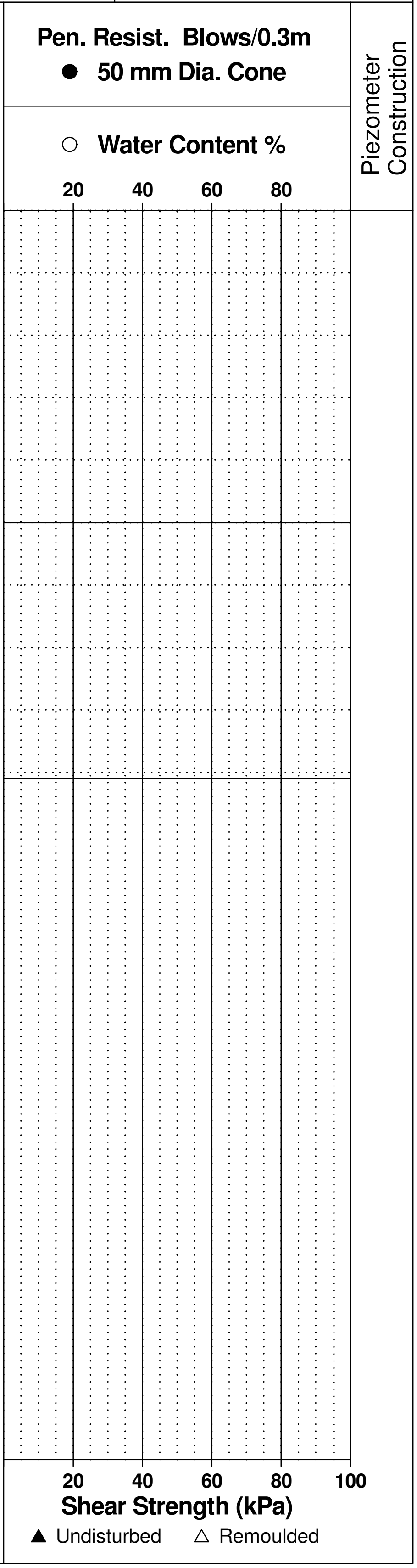
BORINGS BY Excavator

DATE 2022 May 20

FILE NO. **PG6233**

HOLE NO. **TP 3-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	117.34
FILL: Dense to very dense granular crushed stone, some sand	▨	G	1				
0.28							
FILL: Dense brown silty sand with gravel, asphalt and concrete	▨	G	2				
						1	116.34
2.00							
Compact brown **SILTY SAND** with gravel, trace cobbles		G	3				
2.06							
End of Test Pit Refusal to excavation on inferred bedrock at 2.06 m depth (Open hole GWL at 1.82 m depth)							



APPENDIX 2

FIGURE 1 – KEY PLAN

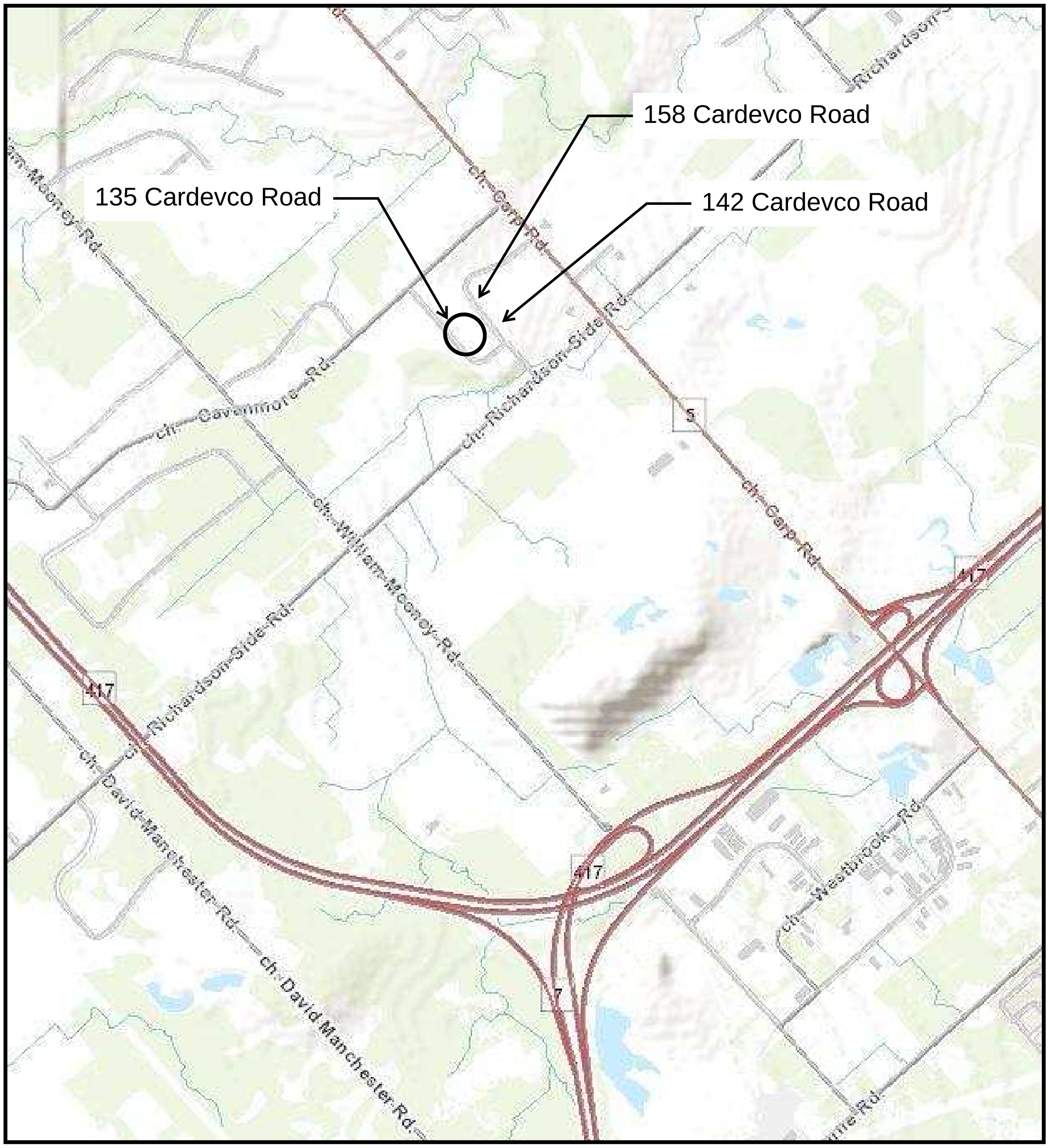
FIGURE 2 – RETAINING WALL CROSS SECTION LOCATION

FIGURE 3A AND 3B – GLOBAL SLOPE STABILITY ANALYSIS FOR STATIC AND SEISMIC LOADING CONDITIONS

FIGURE 4 – UNDERPINNING PROGRAM DETAIL

FIGURE 5 – UNDERPINNING PROGRAM SECTION A-A

DRAWING PG6018-1 - TEST HOLE LOCATION PLAN



158 Cardevco Road

135 Cardevco Road

142 Cardevco Road

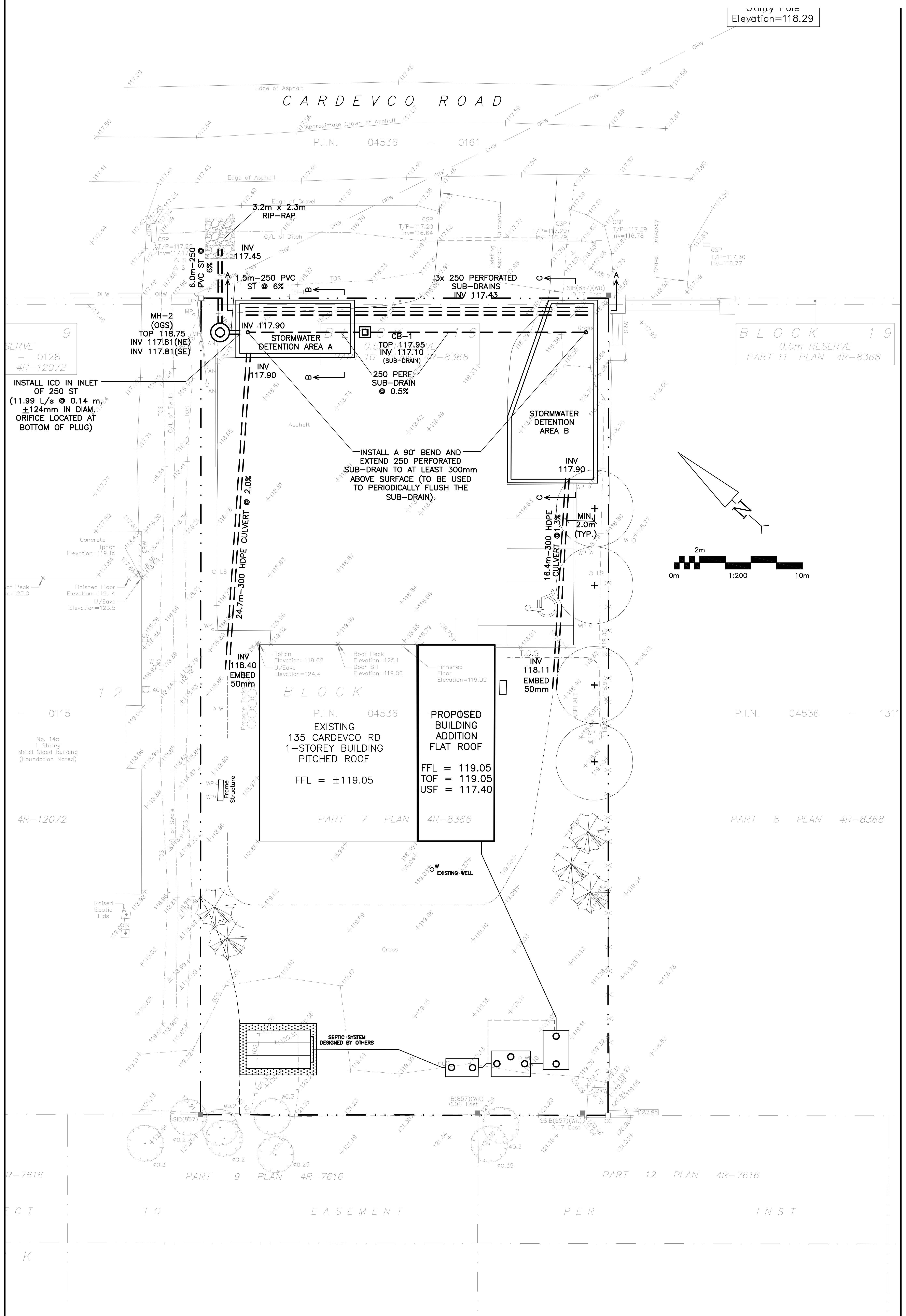
FIGURE 1

KEY PLAN

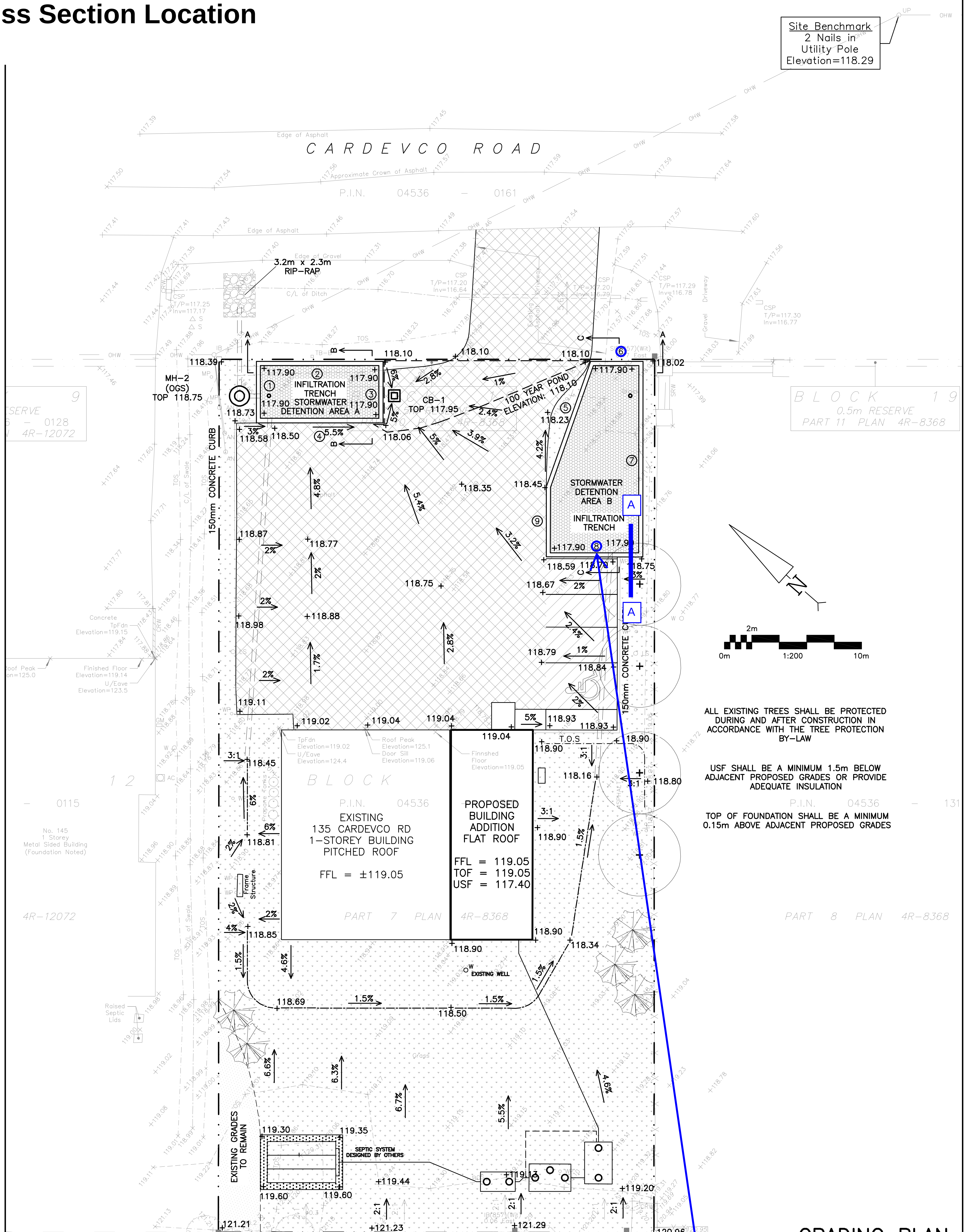
PATERSON GROUP

REFER TO NOTES & DETAILS
ON DRAWINGS C-2 & C-4

Figure 2 - Retaining Wall Cross Section Location



SITE SERVICING PLAN



ALL EXISTING TREES SHALL BE PROTECTED DURING AND AFTER CONSTRUCTION IN ACCORDANCE WITH THE TREE PROTECTION BY-LAW

USF SHALL BE A MINIMUM 1.5m BELOW ADJACENT PROPOSED GRADES OR PROVIDE ADEQUATE INSULATION

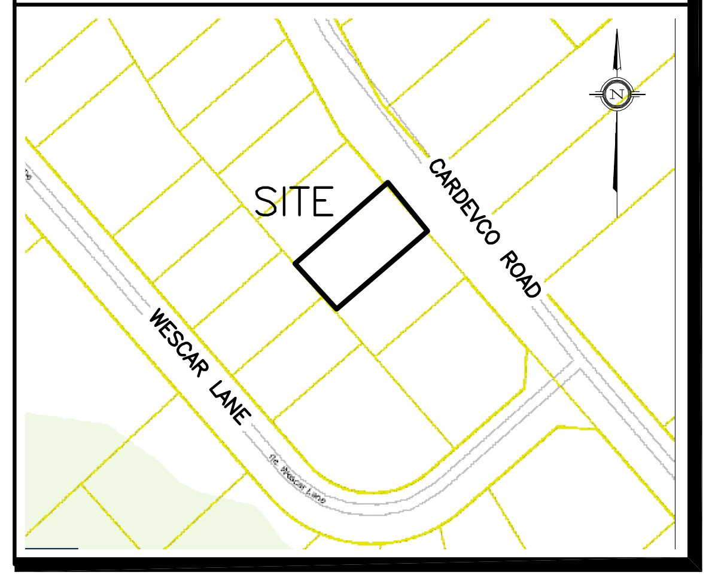
TOP OF FOUNDATION SHALL BE A MINIMUM 0.15m ABOVE ADJACENT PROPOSED GRADES

GRADING PLAN

LEGEND

FFL	FINISHED FLOOR ELEVATION
TOF	TOP OF FOUNDATION
USF	UNDERSIDE OF FOOTING
— · · — · · —	PROPERTY LINE
CB ▣	CATCH-BASIN
MH ◎	STORM MANHOLE
__ ST __	STORM SEWER
INV	INVERT OF PIPE
x99.99	EXISTING GRADE ELEVATION
+99.99	PROPOSED GRADE ELEVATION
2% →	EXISTING SLOPE OF GRADE
2% →	PROPOSED SLOPE OF GRADE
⇨	EMERGENCY OVERLAND FLOW
__ T.O.S __	TOP OF SLOPE
__ B.O.S __	BOTTOM OF SLOPE
—-—-—	CENTERLINE OF SWALE
············	SILT FENCE BARRIER
◐	ROOF DRAIN
▨	LIGHT-DUTY PAVEMENT
▩	HEAVY-DUTY PAVEMENT
⁘	LANDSCAPE

KEY PLAN



RETAINING WALL SCHEDULE

REF	TOP OF WALL	BOTTOM OF WALL	HEIGHT	NOTES	REF	TOP OF WALL	BOTTOM OF WALL	HEIGHT	NOTES
①	118.88 TO 118.25	117.90	0.98m TO 0.35m	TOP OF WALL TO BE 0.15m ABOVE PROPOSED GRADE DESIGNED BY STRUCTURAL	⑤	118.60 TO 118.25	117.90	0.70m TO 0.35m	TOP OF WALL TO BE 0.15m ABOVE PROPOSED GRADE DESIGNED BY STRUCTURAL
②	118.10	117.90	0.20m	TOP OF WALL TO BE USED AS OVERFLOW OUTLET DESIGNED BY STRUCTURAL	⑥	118.25 TO 118.17	117.90	0.35m TO 0.27m	TOP OF WALL TO BE 0.15m ABOVE PROPOSED GRADE DESIGNED BY STRUCTURAL
③	118.25 TO 118.21	117.90	0.35m TO 0.31m	TOP OF WALL TO BE 0.15m ABOVE PROPOSED GRADE DESIGNED BY STRUCTURAL	⑦	118.17 TO 118.90	117.90	0.27m TO 1.00m	TOP OF WALL TO BE 0.15m ABOVE PROPOSED GRADE DESIGNED BY STRUCTURAL
④	118.21 TO 118.88	117.90	0.31m TO 0.98m	TOP OF WALL TO BE 0.15m ABOVE PROPOSED GRADE DESIGNED BY STRUCTURAL	⑧	118.90 TO 118.74	117.90	1.00m TO 0.84m	TOP OF WALL TO BE 0.15m ABOVE PROPOSED GRADE DESIGNED BY STRUCTURAL
					⑨	118.74 TO 118.60	117.90	0.84m TO 0.70m	TOP OF WALL TO BE 0.15m ABOVE PROPOSED GRADE DESIGNED BY STRUCTURAL

PATERSON GROUP

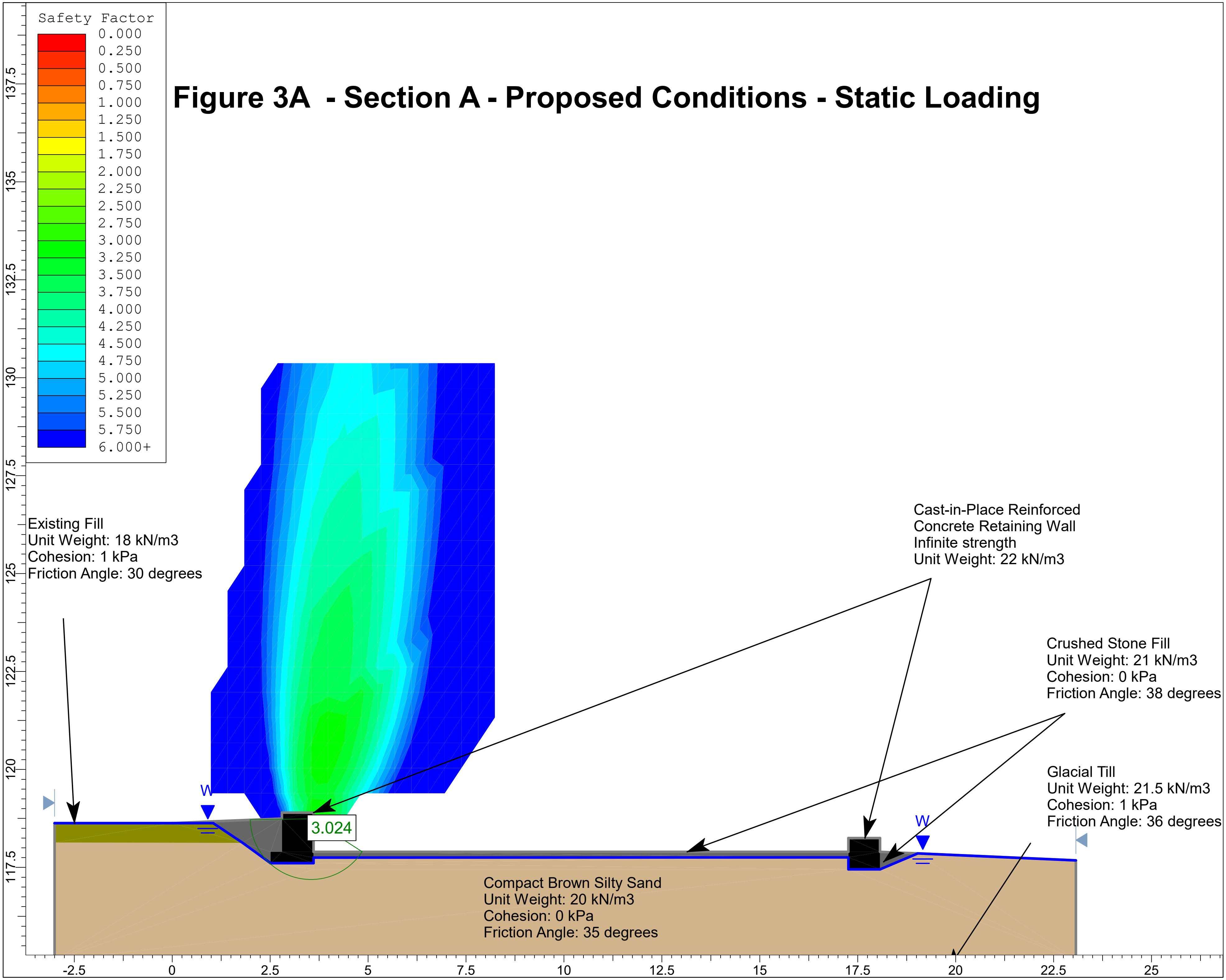


Figure 3A - Section A - Proposed Conditions - Static Loading

Existing Fill
Unit Weight: 18 kN/m3
Cohesion: 1 kPa
Friction Angle: 30 degrees

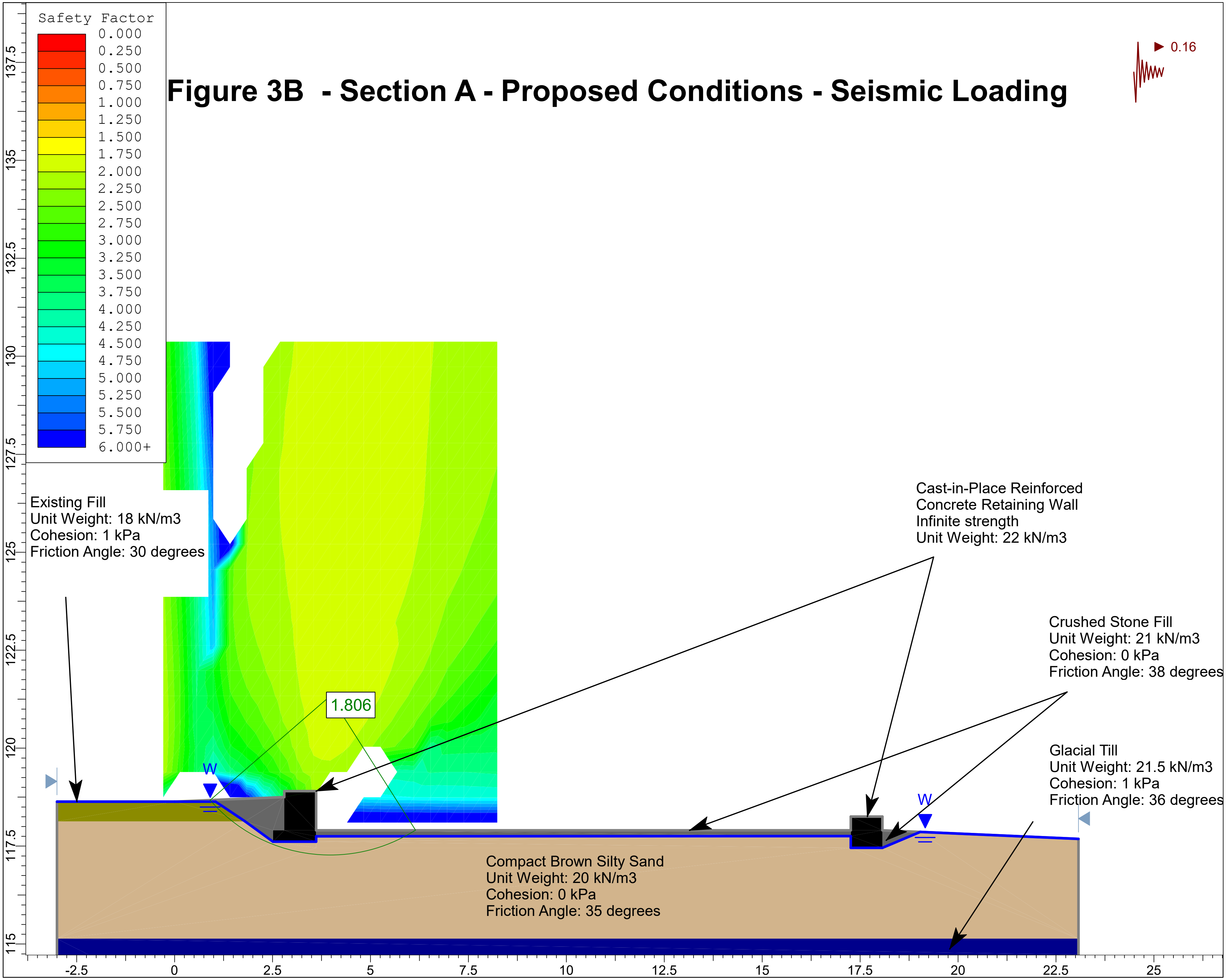
Cast-in-Place Reinforced
Concrete Retaining Wall
Infinite strength
Unit Weight: 22 kN/m3

Crushed Stone Fill
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees

Glacial Till
Unit Weight: 21.5 kN/m3
Cohesion: 1 kPa
Friction Angle: 36 degrees

Compact Brown Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 degrees

Figure 3B - Section A - Proposed Conditions - Seismic Loading



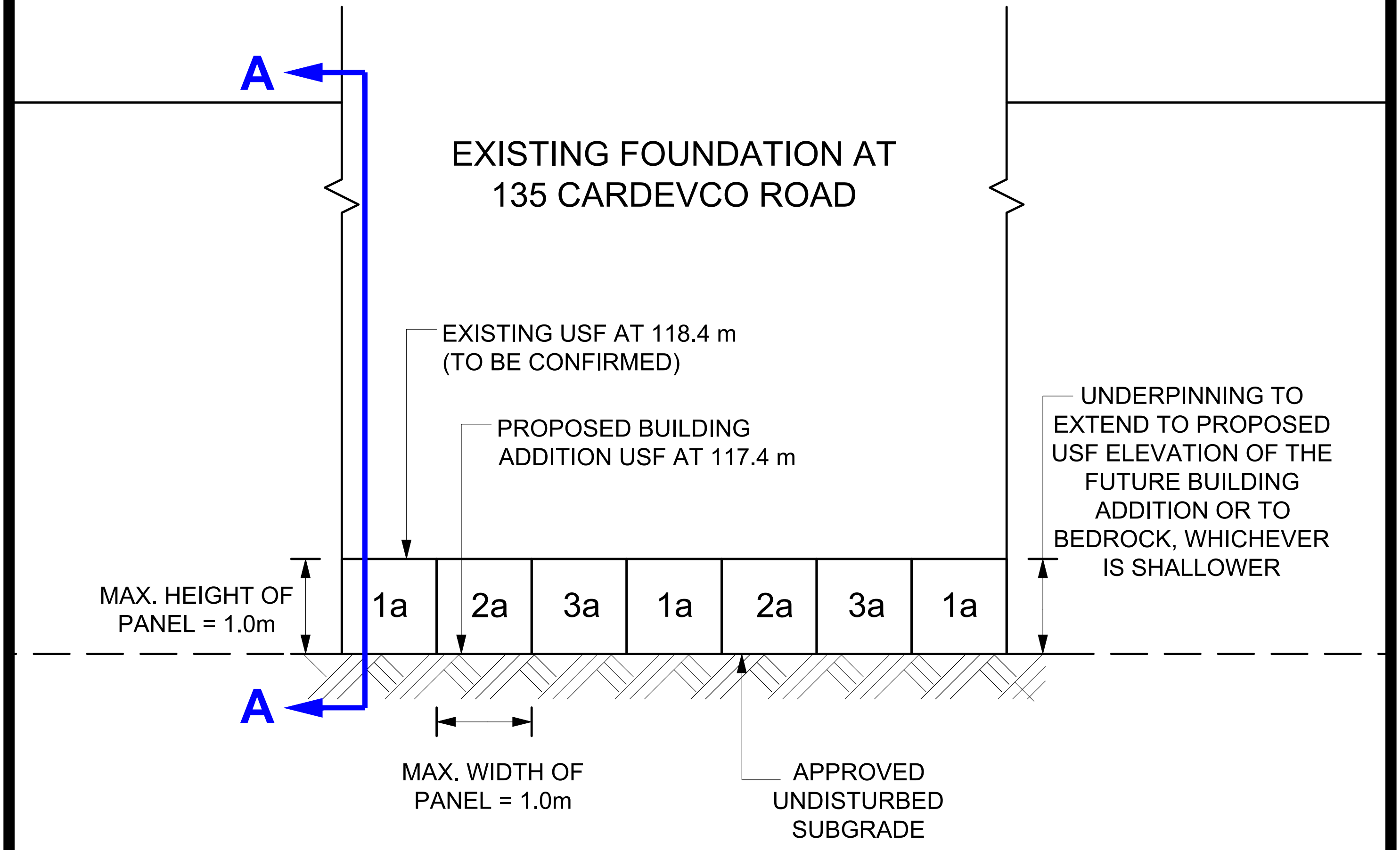
Existing Fill
Unit Weight: 18 kN/m3
Cohesion: 1 kPa
Friction Angle: 30 degrees

Cast-in-Place Reinforced
Concrete Retaining Wall
Infinite strength
Unit Weight: 22 kN/m3

Crushed Stone Fill
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Friction Angle: 38 degrees

Glacial Till
Unit Weight: 21.5 kN/m3
Cohesion: 1 kPa
Friction Angle: 36 degrees

Compact Brown Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 degrees



PATERSON GROUP

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

PREMIER BUS LINE INC.

PROPOSED INDUSTRIAL REDEVELOPMENT
135 CARDEVCO ROAD
OTTAWA, ONTARIO

Title:

UNDERPINNING PROGRAM DETAIL

Date: **11/2023**

Report No.: **PG6018-1**

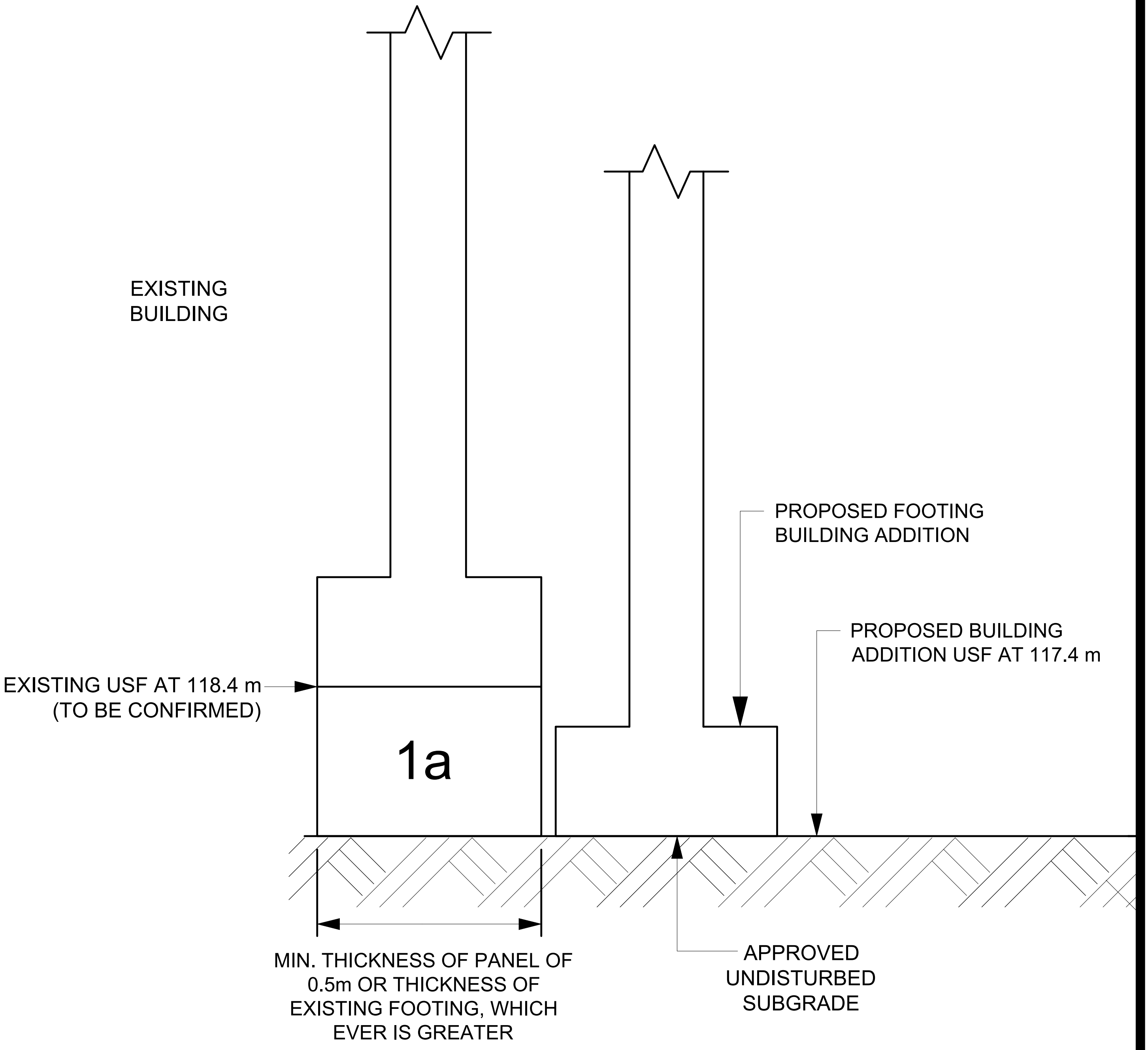
Scale: **N.T.S.**

Drawing No.: **FIGURE 4**

Drawn by: **ZS**

Checked by: **MS**

p:\autocad drawings\geotechnical\pg60xx\pg6018\pg6018-4 underpinningdetails.dwg



PATERSON GROUP

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

PREMIER BUS LINE INC.

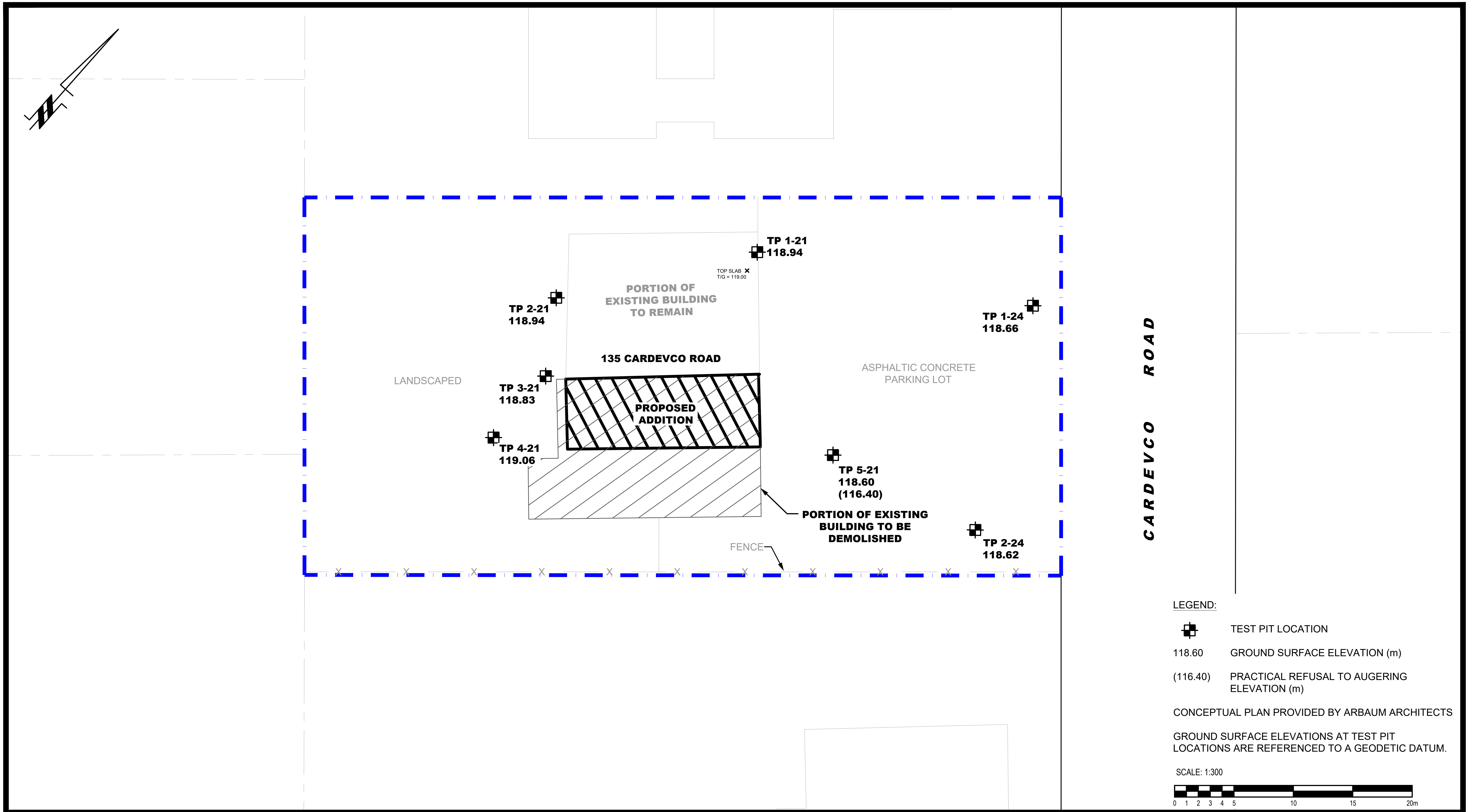
PROPOSED INDUSTRIAL REDEVELOPMENT
135 CARDEVCO ROAD
OTTAWA, ONTARIO

Title:

UNDERPINNING PROGRAM
SECTION A-A

Date: **11/2023**	Report No.: **PG6018-1**
Scale: **N.T.S.**	Drawing No.: **FIGURE 5**
Drawn by: **ZS**	Checked by: **MS**

p:\autocad drawings\geotechnical\pg60xx\pg6018\pg6018-4 underpinningdetails.dwg



LEGEND:

⌖	TEST PIT LOCATION
118.60	GROUND SURFACE ELEVATION (m)
(116.40)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY ARBAUM ARCHITECTS

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

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

NO.	REVISIONS	DATE	INITIAL
1	ADDED 2024 TEST PITS	10/07/2024	OB

PREMIER BUS LINES INC.
GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL REDEVELOPMENT
135 CARDEVCO ROAD
OTTAWA (CARP), ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:300
Date: 11/2021
Drawn by: JM
Report No.: PE6018-1
Checked by: MS
Dwg. No.: PG6018-1
Approved by: DJG
Revision No.: 1

11x17

p:\autocad drawings\geotechnical\pg60xx\pg6018\pg6018-1-test hole location plan (rev.01).dwg

APPENDIX 3

PHOTOGRAPHS FROM TEST PITS – NOVEMBER 2021

Photo 1: TP 1-21

Photo 2: TP 3-21

Photo 3: TP 4-21

Photo 4: TP 5-21

APPENDIX 4

RELEVANT MEMORANDUMS

re: **Response to City Comment**
Proposed Industrial Redevelopment
135 Cardevco Road – Carp, Ottawa, Ontario
to: Premier Bus Lines Inc. – **Eric Hochgeschurz**
to: City of Ottawa – Lead Planner – **Jerrica Gilbert**
date: March 24, 2025
file: PG6018-MEMO.06 Revision 1

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide a response to the geotechnical-related comment from the City of Ottawa listed in the letter dated March 12, 2025 (File No. D07-12-22-0173) regarding the proposed industrial redevelopment at the aforementioned site. This memorandum should be read in conjunction with the Geotechnical Investigation Report (Paterson Group Report PG6018-1 Revision 7 dated March 24, 2025) and our response memorandum to the City comments (PG6018-MEMO.05 Revision 1, dated March 24, 2025), which have been prepared for the proposed development at the aforementioned site.

Geotechnical-Related City Comment

Comment 6: *No formal response to the City comments, pertaining to the Geotechnical investigation, was received from Paterson Group.*

The response, from D.B.Gray Engineering, to last set of City comments pertaining to the Geotechnical investigation, states that no subdrains are required for the retaining walls. To the contrary, the Section 6.8 Retaining walls (Backfill Material) still references a need for longitudinal drains and weep holes.

If no drains are required, please revise the mentioned section and explicitly state in the report that the retaining wall drainage is not required.

Response: This comment has been acknowledged. Reference should be made to the above-mentioned geotechnical report (Paterson Group Geotechnical Report PG6018-1 Revision 7, dated March 24, 2025), and our response memorandum to the City comments (PG6018-MEMO.05 Revision 1, dated March 24, 2025).

Comment 9: *Previous City comment (8b) was not addressed:*
“The Geotechnical report prepared by Paterson Group Inc., dated November 24, 2023; Revision 4 (and also latest revision 5), requires a 1H:1V slope from the ground surface to the bottom of the retaining wall or a minimum of 1m setback from the back of the wall, for a proper placement and compaction of the backfill material.

This requirement appears not to have been met for either of the two infiltration trenches. Please provide dimensions between the SWM features and the adjacent property lines on the Servicing & Grading plan to confirm the required backfill set-back design requirements.

Please also coordinate with the Arbaum architects who prepared the Stormwater Detention retaining Wall Details plan (S300), which shows the distance to the property line, as 1 foot and 0 inches.

If the retaining wall geotechnical backfill requirement of a minimum 1 m cannot be accommodated within the property and a conventional construction 1:1 slope offset for placement of the OGS, then encroachment agreements with the affected adjacent property owners will be required, prior to the SPA approval". Please note that excavations deeper than 30cm within the public ROW require municipal consent.

Response: Our geotechnical response to this comment has already been provided. Reference should be made to our above-mentioned geotechnical report (Paterson Group Geotechnical Report PG6018-1 Revision 7, dated March 24, 2025), and our response memorandum to the City comments (PG6018-MEMO.05 Revision 1, dated March 24, 2025).

Comment 12: *Previous City comment (10b) was not addressed:*
"Cross-Sections show setback behind the retaining walls between 0.15 m – 0.35 m. Please see comments above pertaining to the minimum backfill requirement and impacts to the adjacent properties".

Minimum space required behind the retaining well for the proper compaction of the backfill material is 1 m, in accordance with the geotechnical requirements. That condition appears to not have been met, consequently encroachment agreements with the affected adjacent property owners will be required, prior to SPA approval.

Please note that excavations deeper than 30cm within the public ROW require municipal consent.

Response: Our geotechnical response to this comment has already been provided. Reference should be made to our above-mentioned geotechnical report (Paterson Group Geotechnical Report PG6018-1 Revision 7, dated March 24, 2025), and our response memorandum to the City comments (PG6018-MEMO.05 Revision 1, dated March 24, 2025).

We trust that this memorandum satisfies your requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
March 24, 2025
Z.W. AL-MOSELLY
100194385
PROVINCE OF ONTARIO

Zubaida Al-Moselly, Ph.D., P.Eng.

David J. Gilbert, P.Eng.

PATERSON GROUP

memorandum

re: # Response to City Comments

Proposed Industrial Redevelopment
135 Cardevco Road – Carp, Ottawa, Ontario
to: Premier Bus Lines Inc. – **Eric Hochgeschurz**
to: City of Ottawa – Lead Planner – **Jerrica Gilbert**
date: March 24, 2025
file: PG6018-MEMO.05 Revision 1

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide responses to the geotechnical-related comments from the City of Ottawa listed in the letter dated December 9, 2024 (File No. D07-12-22-0173) regarding the proposed industrial redevelopment at the aforementioned site. This memorandum should be read in conjunction with Paterson Group Geotechnical Report PG6018-1 Revision 7, dated March 24, 2025.

The following drawings prepared by others have been reviewed by Paterson in preparation for this memorandum:

- ❒ Stormwater Detention Retaining Wall Details – Cardevco Warehouse – Drawing No. S300 – Revision 5 dated February 10, 2025, Prepared by Abraum Architects.
- ❒ Site Servicing Plan and Grading Plan – Proposed Addition 135 Cardevco Road – Job No. 21081 – Drawing No. C-1 to C-4 – Revision 6 Dated February 10, 2025 – Prepared by D. B. Gray Engineering Inc.

Geotechnical-Related City Comments

Comment 5.a: *Please update the report and provide a conservatively established, geodetically referenced, long-term GWT elevation applicable at the infiltration trenches' locations, to avoid ambiguity for the Civil design considerations. At the moment it is not clear if it is 1.6 m – 1.7 m bgs. (latest In-Situ Infiltration Testing, dated August 20, 2024, by Paterson Group) or 2.0 m – 3.0 m bgs. (Geotechnical report long term GWT elevation assumption).*

This newly determined GWT elevation would indicate that the expected separation between the invert of the infiltration sand medium of the infiltration trench and the GWT elevation, would be approximately 0.5 m. The Geotechnical report and the City LID guideline require that it is not less than 1 m. The City guideline additionally states that if the 1 m separation cannot be met, then groundwater mounding analysis may be required and the report should be clear if one is needed in this case.

Please clarify the GWT elevation at the infiltration trenches A and B, determine if the groundwater mounding analysis is required, and provide the analysis if it is required.

Response: This comment has been acknowledged. Reference should be made to Paterson Group Geotechnical Report PG6018-1 Revision 7, dated March 24, 2025. The variability in ground surface elevations across the site influences groundwater depth measurements, as such the groundwater elevations have been reported as geodetic values above sea level (ASL). The groundwater elevations measured in July 2024 ranged between the maximum and minimum geodetic groundwater levels recorded in November 2021. The groundwater elevations encountered onsite on November 12, 2021, and July 9, 2024, ranged from the geodetic elevation of 116.70 to 117.06 m ASL.

Based on the observed groundwater elevations and infiltration depths, the long-term groundwater table is anticipated to range between the geodetic elevation of 117.0 m and 116.0 m ASL. It should be noted that the value provided for the long-term groundwater table does not account for seasonal high groundwater levels. The recorded groundwater infiltration depths are provided in the applicable Soil Profile and Test Data sheet in Appendix 1 of the above-noted updated geotechnical report.

Comment 5.b: *The Civil design (Plan C-1) proposed the infiltration trenches retaining walls' subdrain drainage to be placed behind the walls, at the building USF elevation (117.4), which is 1.4 m above the retaining walls' footing, while the retaining wall design plan S-300 (by Arbaum Architects) shows the drainage subdrains at the walls' footing (at the elevation of approximately 116.0 m).*

The report does not address this aspect and it is not clear at which elevation, along the back of the walls, the subdrains should be placed, to work as geotechnically intended, especially in the high GWT condition where the walls' footing is anticipated to be below the GWT elevation. Please confirm in the updated report, where the subdrains need to be placed, to prevent misinterpretation.

Response: Considering the site's groundwater elevation and the proposed footing elevation of the unheated retaining wall structure, the 100 mm diameter drainage pipe (sub-drain) located behind the retaining walls (i.e., along the back of the retaining walls) at the USF elevation of the proposed retaining walls is not required from a geotechnical perspective. Please refer to the updated geotechnical report mentioned above.

Comment 7.d: *2024 Paterson Group In-Situ Infiltration testing TP 1-24 for infiltration pond Area A identified GWT elevation at 116.96 m (geodetic elevation of ground surface minus 1.7m) and TP 2-24 at 116.92m (geodetic elevation of ground surface minus 1.7m) for Infiltration Area B. This data was obtained in July of 2024, which would not normally be considered the most conservative time period for GWT identification, hence the GWT elevation might be even higher during early and late year time periods.*

While working down from the top of the Infiltration trench A at the geodetic elevation of (117.95 – 116.96) – (0.05+0.15+0.3) = 0.49 m (separation between the bottom of the sand infiltration medium and the GWT).

While working down from the top of the Infiltration trench B at the geodetic elevation of (117.95 – 116.92) – (0.05+0.15+0.3) = 0.53 m (separation between the bottom of the sand infiltration medium and the GWT).

The report rationale on page 3 states that the latest GWT elevations were derived form In-Situ investigation test pits TP 1-24 and TP 2-24 and provides assurance that underside of infiltration trenches is between 0.75 m and 1.45 m, which does not match the findings of the said study,directly at the location of the infiltration trenches, and seems to overestimate the true separation.

If the previous rational (assuming the ditch invert elevation of 116.70 m to be indicative of the GWT elevation and top of trench at 117.9 m) were to be considered (in absence of the latest geotechnical GWT data) the result would have been 0.75 m, which still would have been below required 1 m separation to the bottom of the sand medium and therefore water mounding analysis would be required, to ensure that the trench would work as designed.

It is not clear at the moment if the latest GWT elevations (1.6m-1.7m bgs.) noted at the trenches' locations supersede the original geotechnical report's assumption of 2-3 m GWT bgs or apply in determining the separation between the infiltration trenches and the GWT elevation.

This concern has also been included in the Geotechnical report comments and requires clarification, and potentially groundwater mounding analysis, from the Geotechnical consultant.

Response: Reference should be made to our response to Comment 5.a in the current memorandum.

Comment 8.b: *The Geotechnical report prepared by Paterson Group Inc., dated November 24, 2023; Revision 4, requires a 1H:1V slope from the ground surface to the bottom of the retaining wall or a minimum of 1m setback from the back of the wall, for a proper placement and compaction of the backfill material.*

This requirement appears not to have been met for either of the two infiltration trenches. Please provide dimensions between the SWM features and the adjacent property lines on the Servicing & Grading Plan to confirm the required backfill set-back design requirements. Please also coordinate with the Arbaum architects who prepared the Stormwater Detention retaining Wall Details Plan (S300), which shows the distance to the property line, as 1 foot and 0 inches.

If the retaining wall geotechnical backfill requirement of a minimum 1 m cannot be accommodated within the property and a conventional construction 1:1 slope offset for placement of the OGS, then encroachment agreements with the affected adjacent property owners will be required, prior to the Site Plan Control approval.

Response: This comment has been acknowledged. Please refer to the updated geotechnical report mentioned above. Alternative backfill materials have been proposed in cases where there is a limited setback between the proposed retaining wall and the property line.

Additionally, to prevent encroachment onto adjacent properties, protect personnel working in trenches with steep or vertical sides, minimize the risk of soil collapse, and ensure compliance with safety regulations, the use of trench boxes is recommended for the excavation of the proposed retaining wall at all times in areas where conventional open-cut excavation is not feasible due to limited setback.

Comment 10.b: *Cross-Sections show setback behind the retaining walls between 0.15 m – 0.35 m. Please see the comments above pertaining to the minimum backfill requirement and impacts to the adjacent properties.*

Response: Reference should be made to our response to Comment 8.b in the current memorandum.

We trust that this memorandum satisfies your requirements.

Best Regards,

Paterson Group Inc.

Zubaida Al-Moselly, Ph.D., P.Eng.

LICENSED PROFESSIONAL ENGINEER
March 24, 2025
Z.W. AL-MOSELLY
100194385
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

PATERSON GROUP

memorandum

re: **Grading and Site Servicing Plan Review and Geotechnical Recommendations**
Proposed Industrial Redevelopment
135 Cardevco Road – Carp, Ottawa, Ontario
to: Premier Bus Lines Inc. – **Mr. Eric Hochgeschurz**
to: Arbaum Architects – **Ms. Mariana Palos** – marianapalos@arbaum.com
date: November 24, 2023
file: PG6018-MEMO.04

Further to your request and authorization, Paterson Group (Paterson) prepared the current memorandum to document our grading plan and site servicing plan review for the proposed Industrial Redevelopment to be located at 135 Cardevco Road in the City of Ottawa, Ontario. The following memorandum should be read in conjunction with Paterson Group Report PG6018-1 Revision 4 dated November 24, 2023.

Grading and Site Servicing Plan Review

Paterson reviewed the following grading and site servicing plan prepared by D. B. Gray Engineering Inc. for the aforementioned development:

☐ Grading Plan and Site Servicing Plan – Proposed Building Addition – 135 Cardevco Road - Job No. 21081 - Drawing No. C-1 – Revision 4 - dated June 9, 2023.

Based on our review of the above-noted drawing and the subsurface conditions present at the subject site, the proposed grading is considered acceptable from a geotechnical perspective. A silty clay deposit was not encountered during the geotechnical investigation and therefore permissible grade raise restrictions are not applicable to the subject site. Tree planting setbacks, based on the City of Ottawa “Tree Planting in Sensitive Marine Clay Soils - 2017 Guideline”, are not required as well.

The proposed underside of footing elevations (USF) for the proposed building addition is expected to be at an elevation of 117.40 m which is lower than the existing warehouse. Based on our review of the above-noted drawing, sufficient frost cover will be provided for the proposed footings (minimum 1.5 m below finished grade for heated structures).

For unheated structures, such as stairs, servicing pipes, and retaining walls, a minimum frost cover of 2.1 m below the finished grade is required to provide sufficient frost protection. Based on our review of the grading and site servicing plan, proposed retaining walls, culverts, catch basin, manhole, and stormwater pipes were noted to be provided with a reduced soil cover to footings against frost action.

Protection of Footings Against Frost Action

It should be noted that to accommodate the absence of sufficient frost cover (minimum 2.1 m for heated footings) for the proposed footings, a different form of frost protection should be provided. This can be achieved by means of rigid insulation.

Any portion of proposed retaining walls, culverts, catch basin, manhole, and stormwater pipes installed at a depth of 2.1 m below finished grade or deeper is considered to have sufficient soil cover for frost protection. Where insufficient soil cover is present above the invert of stormwater pipe, manhole, catch basin, or USF elevation of culverts and retaining walls, the following frost protection criteria should be followed:

Table 1 – Frost Protection Recommendations for Reduced Soil Cover

Thermal Condition	Soil Cover Provided (mm)	Insulation Dimensions	
		Thickness (mm)	Extension (mm)
Unheated	900-1200	75	Extend 1200 mm horizontally beyond edge of footing or pipe face
	600-900	100	Extend 1800 mm horizontally beyond edge of footing or pipe face
	300-600	150	Extend 2100 mm horizontally beyond edge of footing or pipe face
	0-300	200	Extend 2100 mm horizontally beyond edge of footing or pipe face

Underpinning

Based on our review of the grading plans, and on information collected from our geotechnical investigation, it is understood that the USF for the proposed building addition and the eastern foundation wall are at approximate geodetic elevation of 117.40 and 118.40 m, respectively. Furthermore, there will be no horizontal setback between the proposed building addition and the eastern foundation wall of the existing warehouse. Therefore, the underpinning of the eastern foundation wall of the existing warehouse should be underpinned.

❑ The underpinning program should be completed in sections (panels) by excavating each panel individually in a piano key fashion to maintain adequate lateral support for the existing footings.

☐ A maximum 1.0 m horizontal spacing is required between each excavated panel.
☐ The maximum height of excavation per stage is .1.0m.
☐ Each panel should be excavated using suitable excavation equipment and infilled with a minimum 15 MPa (28-day compressive strength) concrete once the forms are secured in place. Concrete infilling will be done through the cored holes in the floor slab.
☐ For each excavated panel, place 0.75 to 1.0 m forms below the top of the existing footings down to the bottom of the excavation at each stage. The forms should be firmly secured in place prior to pouring concrete.
☐ Once the concrete in the first set of panels has set (12 to 24 hours), the second set of panels can be completed. The process is then repeated in consecutive order to maintain adequate lateral support during the duration of the underpinning program.
☐ In cold weather conditions the concrete should be sufficiently protected with insulated tarps, until the concrete attains its design strength.
☐ The subsequent courses of panels should be offset from the previous course.
☐ The underpinning program should extend down to the USF elevation of the proposed building addition.

Further details regarding the proposed underpinning program are provided in Figure 3 and 4 in appendix 2 of the above noted geotechnical report.

We trust that this memorandum satisfies your requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
November 24, 2023
M. SALEH
100507739
PROVINCE OF ONTARIO

Yashar Ziaeimehr, M.A.Sc.

Maha K. Saleh, P.Eng.