PATERSON GROUP

Geotechnical Investigation

Proposed Commercial Development

2300 Bank Street
Ottawa, Ontario

Prepared for Trinity Group

Report PG7467-1 Revision 1 dated December 18, 2025

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Trinity Group to conduct a geotechnical investigation for the proposed commercial development to be located at 2300 Bank Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

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

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

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

Investigating for the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on available drawings, it is understood that the proposed development to be located at the subject site will consist of a single-story, slab-on-grade commercial building with an approximate footprint of 2,700 ft^2. The remainder of the site will generally be occupied by vehicle parking areas and access lanes with landscaped margins.

It is further anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field investigation was carried out on March 14, 2025, and consisted of advancing a total of 3 boreholes to a maximum depth of 7.6 m below the existing ground surface. The borehole locations were determined in the field by Paterson personnel taking into consideration existing site features and underground services. The locations of the boreholes are shown on Drawing PG7467-1 – Test Hole Location Plan, presented in Appendix 2.

The boreholes were advanced 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, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered 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 and subsequently placed in sealed plastic bags.

All samples were 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.

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

The overburden thickness was evaluated by completing a dynamic cone penetration test (DCPT) at borehole BH 3-25. 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.

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

Groundwater

Flexible polyethylene standpipes were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the field program. The measured groundwater levels are presented and discussed in Section 4.3 and are also provided on the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a handheld GPS unit, and referenced to a geodetic datum. The borehole locations and elevations are presented on Drawing PG7467-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of 1 shrinkage test, 1 grain size distribution analysis, and 2 Atterberg Limits tests were completed on selected soil samples. The results are presented in Section 4.2 and on the Grain Size Distribution and Hydrometer Testing Results, Atterberg Limit Results and Shrinkage Test Results sheets presented in Appendix 1

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a single-story commercial building along with associated vehicle parking areas, access lanes, and landscaped margins. Further, there is a watercourse located adjacent to the west side of the site, and the side slope along the watercourse contains some mature trees.

The site is bordered by Daze Street to the north, Bank Street to east, and by commercial properties to west and south. The existing ground surface across the site is relatively level and at grade with surrounding properties and roadways at an approximate geodetic elevation of 86.6 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of asphalt and fill overlying a deep clay deposit. The fill was generally observed to consist of brown silty sand with varying amounts of gravel, clay and crushed stone, extending to an approximate depth of 0.5 to 1.45 m below the existing ground surface.

A stiff, brown silty clay crust was encountered underlying the fill at all borehole locations. The brown silty clay crust was observed to contain some sand throughout the majority of the layer, and have intermittent layers of brown silty sand. A firm to stiff, grey silty clay deposit was encountered below the brown silty clay at approximate depths of 2.2 to 3.0 m below the existing ground surface. The grey silty clay deposit was observed to have intermittent sandy silt to silty sand seams and an increasing silt content with depth.

Practical DCPT refusal was encountered in borehole BH 3-25 at a depth of 19.7 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, bedrock within the north-east portion of the site consists of shale of the Billings formation, while bedrock within the south-west portion of the site consists of shale of the Carlsbad formation. The overburden drift thickness is estimated to be between 25 and 50 m.

Laboratory Testing

Atterberg Limits testing, as well as associated moisture content testing, was completed on a select silty clay samples. The results of the Atterberg Limits tests are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1. The results of the moisture content testing are presented on the Soil Profile and Test Data Sheets in Appendix 1. The tested silty clay samples classify as inorganic clays of high plasticity (CH) and inorganic silts of high plasticity (MH) in accordance with the Unified Soil Classification System (USCS).

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
BH 2-25	1.07	52	24	28	24.2	CH
BH 3-25	1.83	82	44	38	44.1	MH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content;
CH: Inorganic Clays of High Plasticity, MH: Inorganic Silts of High Plasticity

Grain size distribution analysis was completed on 1 selected silty clay sample. The result of the grain size distribution analysis is presented in Table 2, and on the Grain Size Distribution sheet in Appendix 1.

Table 2 – Grain Size Distribution Results

Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 3-25	1.83	0.0	15.0	21.7	63.3

Linear shrinkage testing was completed on a sample recovered from 1.83 m depth from borehole BH 1-25 and yielded a shrinkage limit of 19.67 and a shrinkage ratio of 1.753 The results of the shrinkage testing are presented on the Linear Shrinkage sheet in Appendix 1.

4.3 Groundwater

The groundwater levels were measured in the piezometers on March 21, 2025. The observed groundwater levels are summarized in Table 3 on the next page.

Table 3 - Summary of Groundwater Level Readings				
Test Hole Number	**Ground Surface Elevation (m)**	**Groundwater Level (m)**	**Groundwater Elevation (m)**	**Recording Date**
BH 1-25	86.63	1.67	84.96	March 21, 2025
BH 2-25	86.65	4.53	82.12	
BH 3-25	86.51	7.23	79.28	
Note: Ground surface elevations at borehole locations are referenced to a geodetic datum.				

It should be noted that groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater levels can also be estimated from the observed colour, moisture content and consistency of the recovered samples.

Based on these observations, the long-term groundwater level is expected to range between approximately 2 to 3 m below the existing ground surface. However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered suitable for the proposed development, from a geotechnical perspective. It is recommended that the proposed building be founded on conventional spread footings placed on an undisturbed, stiff silty clay bearing surface.

Due to the presence of the silty clay deposit, a grade raise restriction will apply to the subject site. Permissible grade raise recommendations are discussed in Section 5.3.

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

5.2 Grading and Preparation

Stripping Depth

Asphalt, topsoil and fill, such as those containing organic or deleterious material, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures. It is anticipated that the existing fill within the proposed building footprint, free of deleterious material and significant amounts of organics, can be left in place below the proposed floor slab. However, it is recommended that the existing fill layer be proof-rolled several times **under dry conditions and in above freezing temperatures,** using suitable compaction equipment, and approved by Paterson personnel at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved engineered fill. The existing fill should be removed entirely below the proposed footings.

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

Fill Placement

Engineered fill used for grading beneath the proposed building 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 building and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

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

5.3 Foundation Design

Bearing Resistance Values

Strip footing up to 3 m wide and pad footings up to 5 m wide, placed on undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**. A geotechnical factor of 0.5 was incorporated to the bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one 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 silty clay soil bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post-construction total and differential settlement of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a silty clay medium when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil of the same or higher capacity as that of the bearing medium.

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **1.0 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 construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class XD**. If a higher seismic site class is required (such as **Class XC**), a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic classification for foundation design of the proposed building, as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2024 for a full discussion of the earthquake design requirements.

5.5 Slab on Grade Construction

With the removal of any topsoil and fill containing construction debris or significant amounts of deleterious materials, the existing fill or native soil subgrade approved by the geotechnical consultant at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for floor slab construction.

It is recommended that the slab-on-grade subgrade be proof-rolled with a suitably sized roller making several passes under dry conditions, prior to fill placement, and which is approved by the geotechnical consultant. Any poor performing areas should be removed and replaced with an engineered fill, such as OPSS Granular A or B Type II.

It is recommended that the upper 200 mm of sub-floor fill consist of OPSS Granular A crushed stone. All backfill material required to raise the grade within the footprint of the proposed building should be placed in a maximum 300 mm thick loose lifts and compacted to 98% of it's SPMDD.

5.6 Pavement Design

It is expected that where the existing pavement structures are in good condition, they can be re-used for the proposed development. However, where there is extensive fatigue cracking of the existing asphalt, the following is recommended:

- ❑ Remove the existing asphalt.
- ❑ Proof-roll the subgrade with several passes of a vibratory drum roller, under the observation of Paterson. Any soft and/or loose areas detected during the proof-rolling should be excavated and replaced with OPSS Granular B Type II material, which is placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD
- ❑ Supply and place an asphaltic wear course consisting of 50 mm of HL-3 asphaltic concrete for car-only parking areas, or supply and place one 50 mm lift of HL-8 asphaltic concrete (binder course) and one 40 mm lift of HL-3 asphaltic concrete (wear course) for access lanes and heavy truck parking/loading areas.

For new areas to be paved, the proposed pavement structures presented in Tables 4 and 5 below can be considered, should they be required at the subject site.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 5 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Truck Parking/Loading 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
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE – Existing fill, or OPSS Granular B Type I or II material placed over in situ soil or engineered fill	

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material, placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Since the building will consist of slab-on-grade construction, a perimeter foundation drainage system is considered optional throughout the portions of the proposed building footprint where the exterior finished surface will consist of landscaping. In areas where hard-scaping or pavement structures will abut the building footprint, it is recommended to implement a foundation drainage system. The system should consist of a 150 mm diameter perforated corrugated plastic pipe wrapped in a geosock and surrounded on all sides by 150 mm of 19 mm clear crushed stone. 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.

The pipe should be placed at the footing level around the exterior perimeter of the structure if the backfill between the founding depth and finished grade will consist of crushed stone fill or site-generated soil backfill placed in conjunction with a composite foundation drainage board.

Foundation Backfill

Backfill against the exterior sides of the foundation walls may consist of free-draining, non-frost susceptible imported crushed stone or clean sand fill. Alternatively, consideration may be given to placing approved soil fill as described in Section 5.2 of this report as backfill against the foundation walls.

If the building's perimeter drainage pipe is located at footing level, a composite foundation drainage board should be placed against the foundation walls to ensure satisfactory drainage of the backfill layer to the perimeter drainage pipe. All fill placed as foundation backfill should be placed in maximum 300 mm thick loose lifts, compacted using suitable compaction equipment (suitably sized smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill) and tested for compaction efforts at the time of construction by Paterson personnel.

Concrete and Brick Sidewalks Adjacent to Buildings

To avoid differential settlements within the proposed concrete and brick sidewalks adjacent to the proposed building, it is recommended that the upper 600 mm of backfill placed below the concrete sidewalks adjacent to the building footprints to consist of non-frost susceptible material such as OPSS Granular A or Granular B Type II. The granular material should be placed in maximum 300 mm loose lifts and compacted to a minimum of 98% of the material's SPMDD using suitable compaction equipment.

The subgrade material should be shaped to promote positive drainage towards the building's perimeter drainage system and placed directly against the foundation walls composite foundation drainage board layer. Consideration should be given to placing a minimum 50 mm thick layer of rigid extruded polystyrene insulation below the granular fill layer, however, should be detailed by Paterson once design drawings are being complete by others.

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.

6.3 Excavation Side Slopes

The side slopes of shallow excavations anticipated at this site should either be cut back at acceptable slopes or should be retained by temporary shoring systems from the start of the excavation until the structure is backfilled. It is anticipated 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. A flatter slope (i.e. 3H:1V) is required for excavation below the groundwater level.

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

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

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

6.4 Pipe Bedding and Backfill

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

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. The bedding should extend to the spring line of the pipe. Cover material, 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 pipes). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

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 95% of the material’s SPMDD.

6.5 Groundwater Control

Based on the results of the geotechnical investigation, it is anticipated that groundwater infiltration into the excavations will be minimal and should be controllable using open sumps. However, 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

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

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of 3 to 4 weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan, to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. Should a Permit to Take Water be required, a minimum of 5 to 6 months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

Impacts on Neighboring Structures

Based on the observed existing groundwater level and anticipated shallow depth of foundation, it is not anticipated that the proposed excavation for the proposed building will extend below the groundwater table. Therefore, no adverse effects from short-term or long-term dewatering are expected for surrounding structures.

6.6 Winter Construction

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost 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.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg Limits testing was completed for select recovered silty clay samples from the subject site. The soil samples were recovered from between an elevation below the anticipated design underside of footing elevation and approximately 3.5 m depth below anticipated finished grade. The results of our testing are presented in Table 1 in Section 4.2 and in Appendix 1.

Based on the results of the Atterberg Limits testing mentioned above, the plasticity index was found to be less than 40% in the tested silty clay samples.

The following tree planting setbacks are therefore recommended for the low to medium sensitivity silty clay deposit throughout the subject site.

Large trees (mature height over 14 m) can be planted within these areas provided that a tree to foundation setback equal to the full mature height of the tree can be provided. Tree planting setback limits are **4.5 m** for small (mature tree height up to 7.5 m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the conditions noted below are met.

- ❑ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade for footings within 10 m from the tree, as measured from the center of the tree trunk and verified by means of the Grading Plan.

- ❑ A small tree must be provided with a minimum of 25 m^3 of available soils volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

- ❑ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

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

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

It is well documented in literature, and in 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 Assessment

The current slope stability analysis for the slope adjacent to the watercourse located on the south-west side of the site, was completed using topographical survey information obtained in the field by Paterson. One (1) slope cross-section (A-A') was studied as the worst-case scenario for the proposed development. The cross-section location is presented on Drawing PG7467-1 - Test Hole Location Plan in Appendix 2.

The slope conditions were reviewed on-site by Paterson at the time of the field investigation. At that time, the slope adjacent to the subject site was observed to be vegetated with grass and some mature trees with no sign significant signs of erosion observed along the toe of the slope. The bottom portion of the slope was observed to be partially covered with rip rap.

The slope stability analysis of the proposed site conditions was conducted using SLIDE, a computer program which permits a two-dimensional stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. A horizontal acceleration of 0.16 g (50% of PGA = 0.32g) was utilized for the seismic analysis.

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 (F.o.S.) 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 F.o.S. greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum F.o.S. of 1.5 is generally recommended for static analysis conditions and a minimum F.o.S. of 1.1 is generally recommended for seismic analysis conditions, where the failure of the slope would endanger permanent structures.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered from the test holes. The effective strength soil parameters used for static analysis are presented in Table 6 below.

Table 6 – Effective and Total Stress Soil Parameters			
Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay	17	33	5
Grey Silty Clay	16	33	10
Fill	18	33	1
Glacial Till	20	33	0
Bedrock	22	NA	NA

The total strength soil parameters used for seismic analysis were also chosen based on the subsoil information recovered within the boreholes. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 7:

Table 7 – Effective and Total Stress Soil Parameters			
Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay	17	0	100
Grey Silty Clay	16	0	50
Fill	18	33	1
Glacial Till	20	33	0
Bedrock	22	NA	NA

The analyses were completed by conservatively assuming fully-saturated groundwater conditions extending up to the ground surface

Limit of Hazard Lands Setback

The results for the slope stability analyses under static and seismic conditions at cross-sections A-A’ are shown on Figures 2A and 2B, which are attached to the current report in Appendix 2.

The results of the slope stability analysis under static conditions at cross-section A-A’ indicate a suitable factor of safety exceeding 1.5, while the results of the slope stability analysis under seismic conditions at cross-section A-A’ indicate a suitable factor of safety exceeding 1.1.

Accordingly, a Limit of Hazard Lands setback of 11 m is recommended for the proposed development. This 11 m Limit of Hazard Lands setback consists of a 5 m toe erosion allowance, and a 6 m erosion access allowance. The setbacks are also shown on the cross-sections (Figures 2A and 2B) and on Drawing PG7467-1 - Test Hole Location Plan, included in Appendix 2. The Limit of Hazard Lands does not conflict with the proposed development at the site, as shown on Drawing PG7467-1 – Test Hole Location Plan.

It should be noted that the various regulating authorities may subject the proposed development to additional setbacks beyond the Limit of Hazards Lands provided herein.

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 detailed Grading Plan from a geotechnical perspective.
- ❑ Observation of all bearing surfaces prior to the placement of concrete and fill.
- ❑ Sampling and testing of the engineered fill materials.
- ❑ Observation of proof rolling of existing fill, if applicable.
- ❑ 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.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

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

8.0 Statement of Limitations

The recommendations provided herein 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 Trinity Group, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Nicole R.L. Patey, P.Eng.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ Trinity Group (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS RESULTS

GRAIN SIZE ANALYSIS RESULTS

SHRINKAGE TEST RESULTS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2300 Bank Street, Ottawa, ON

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 371325.20 **NORTHING:** 5024263.77 **ELEVATION:** 86.63

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

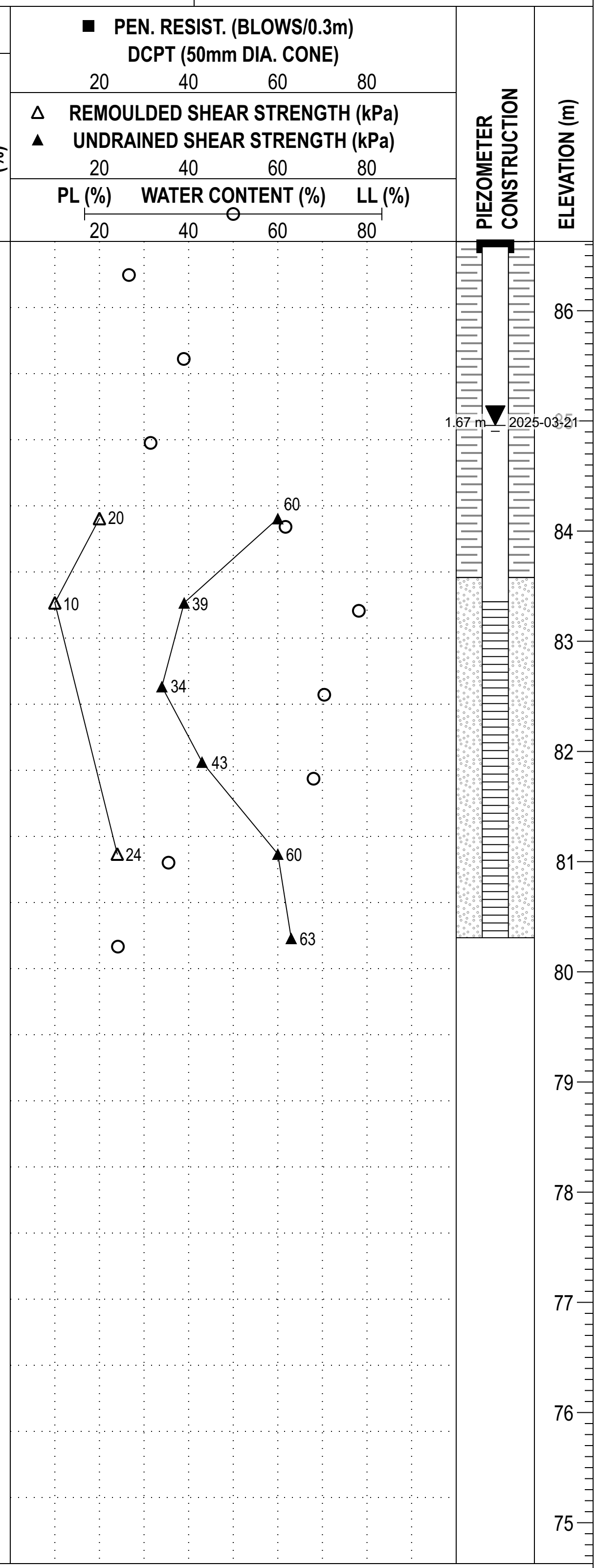
REMARKS:

DATE: March 14, 2025

FILE NO. : PG7467

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION (GROUND SURFACE)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
ASPHALT 0.05m [86.58m]				
FILL: Compact, brown silty clay with sand, gravel and crushed stone 0.53m [86.10m]	AU 1			27
Stiff, brown **SILTY CLAY** - Intermittent silty sand seams	SS 2	92	6-4-5-4 9	39
	SS 3	100	3-2-3-2 5	31
2.97m [83.66m]	SS 4	100	P	62
Firm to stiff, grey **SILTY CLAY** - Intermittent sandy silt to silty sand seams	SS 5	96	P	78
	SS 6	83	P	70
	SS 7	67	P	68
	SS 8	75	P	35
6.71m [79.92m]	SS 9	79	P	24
End of Borehole				
(GWL at 1.67 m depth - March 21, 2025)				



DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2300 Bank Street, Ottawa, ON

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 371303.85 **NORTHING:** 5024281.56 **ELEVATION:** 86.65

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 14, 2025

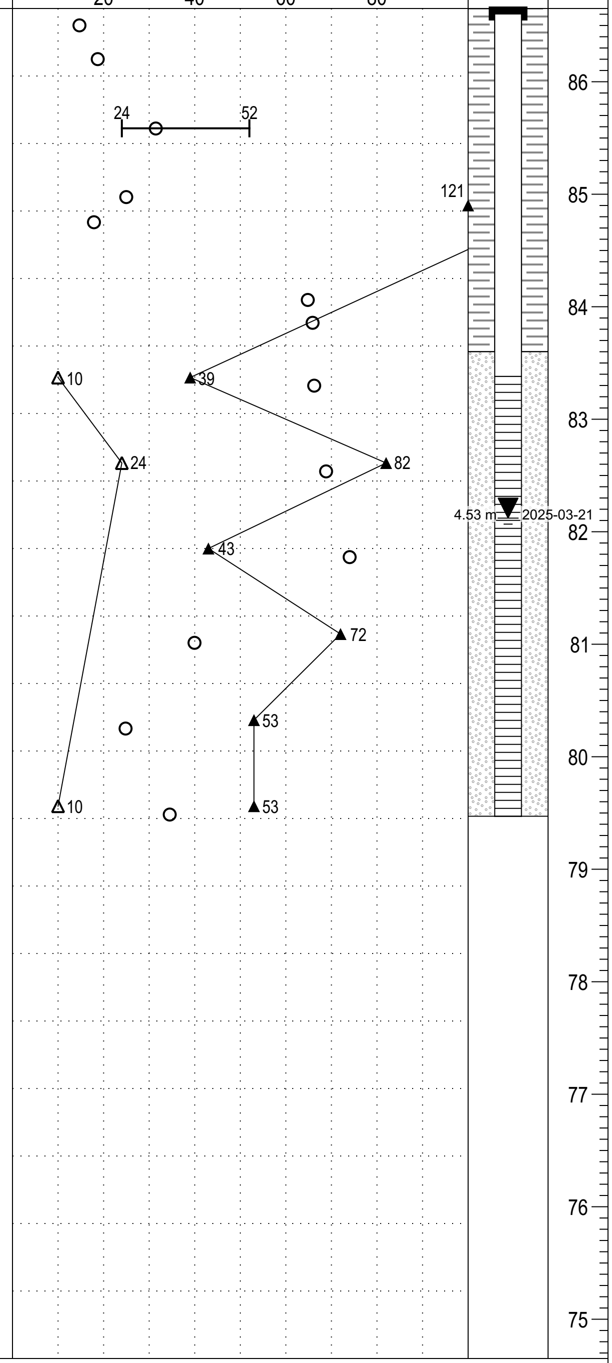
FILE NO. : PG7467

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	DEPTH / ELEVATION
GROUND SURFACE	
ASPHALT	0.05m [86.60m]
FILL: Compact, brown silty sand with silty clay, gravel and crushed stone	0.69m [85.96m]
Very stiff, brown **SILTY CLAY**	1.83m [84.82m]
Compact, brown **SILTY SAND** with **SILTY CLAY**	2.21m [84.44m]
Firm to stiff, grey **SILTY CLAY** - Intermittent sandy silt to silty sand seams at 3.89 m depth	7.47m [79.18m]
End of Borehole (GWL at 4.53 m depth - March 21, 2025)	

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			15 19
SS 2	83	5-3-4-4 7	31
SS 3	67	P	25 18
SS 4	54	2-1-1-1 2	65 66
SS 5	33	P	66
SS 6	96	P	69
SS 7	83	P	74
SS 8	100	P	40
SS 9	75	P	25
SS 10		P	35

■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa)
PL (%) WATER CONTENT (%) LL (%)



STRATA PLOT | DEPTH (m) | PIEZOMETER CONSTRUCTION | ELEVATION (m)

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SOIL PROFILE AND TEST DATA

PATERSON GROUP

Geotechnical Investigation

2300 Bank Street, Ottawa, ON

COORD. SYS.: MTM ZONE 9 **EASTING:** 371284.06 **NORTHING:** 5024246.17 **ELEVATION:** 86.51

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 14, 2025

FILE NO. : PG7467

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION (GROUND SURFACE)	Strata boundary depth [elevation]
ASPHALT	0.08m [86.43m]
FILL: Compact, brown silty sand with gravel and crushed stone, trace clay - Very dense by 0.76 m depth	1.45m [85.06m]
Stiff, brown **SILTY CLAY**, with some sand - Silty sand seam by 2.44 m depth	2.97m [83.54m]
Firm, grey **SILTY CLAY** - Sandy silty to silty sand seams by 3.81 m depth - Silt content increasing with depth	7.62m [78.89m]
Dynamic Cone Penetration Test commenced at 7.62 m depth	

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	REMOULDED SHEAR STRENGTH (kPa)	UNDRAINED SHEAR STRENGTH (kPa)	PL (%)	LL (%)
AU 1			8				
			11				
SS 2	98	50-/-/-/ 50/0.13	15				
SS 3	83	2-3-3-2 6	50			44	82
SS 4	100	P	69	20	60		
SS 5	75	P	71	10	43		
SS 6	100	P	76	10	39		
SS 7	83	P	73	10	48		
SS 8	92	P	24		63		
SS 9	92	P	28	14	48		
SS 10	67	P	35	14	48		

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12

ELEVATION (m): 86 85 84 83 82 81 80 79 78 77 76 75

PIEZOMETER CONSTRUCTION: 7.23 m 2025-03-21

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2300 Bank Street, Ottawa, ON

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 371284.06 **NORTHING:** 5024246.17 **ELEVATION:** 86.51

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

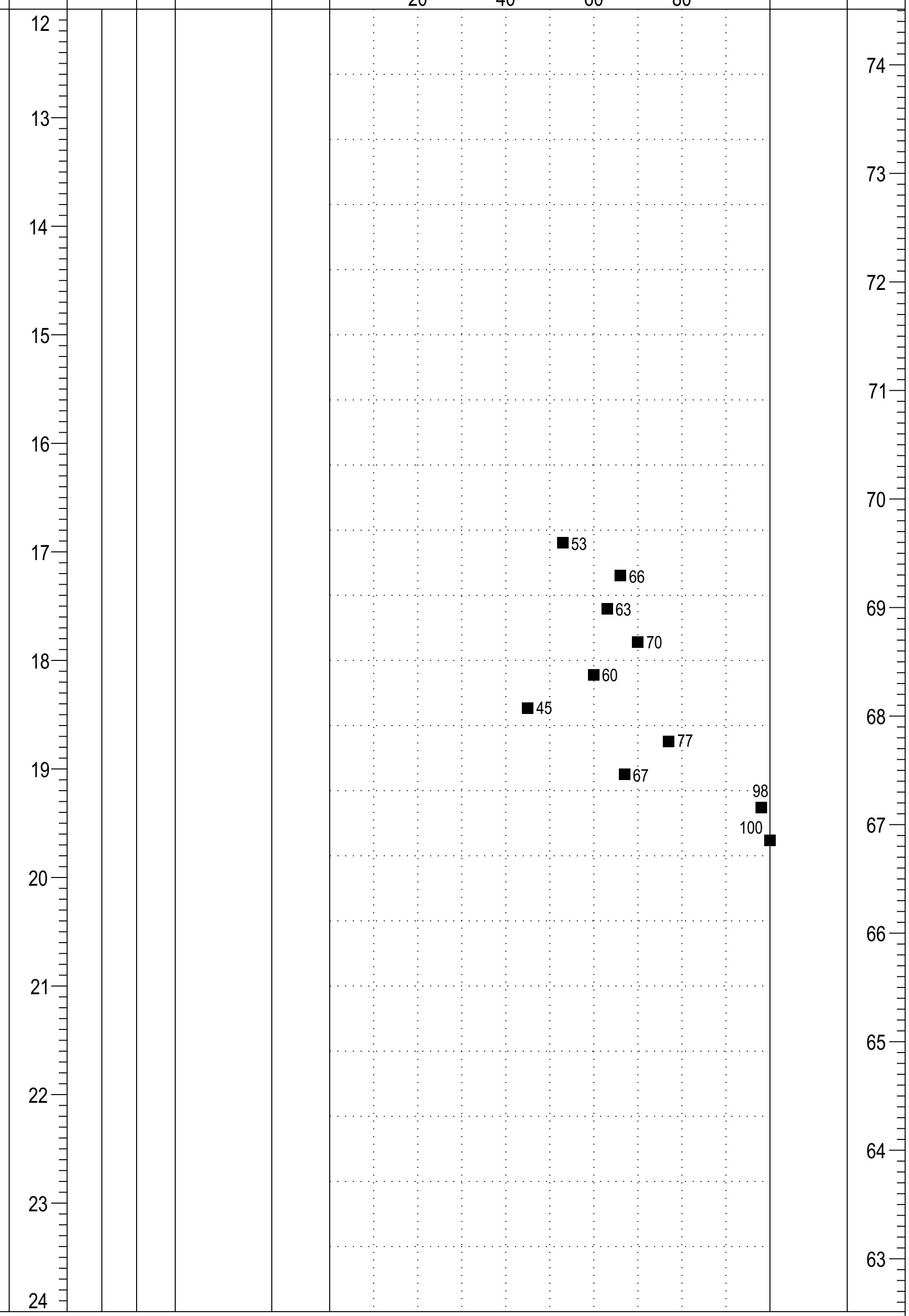
REMARKS:

DATE: March 14, 2025

FILE NO. : PG7467

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
19.69m [66.82m]		12 – 19.69							74 – 66.82
End of Borehole Cone pushed from 7.62 to 16.76 m depth Practical refusal to DCPT at 19.69 m depth on inferred bedrock (GWL at 7.23 m depth - March 21, 2025)		19.69 – 24							66.82 – 63



DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water 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 at 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

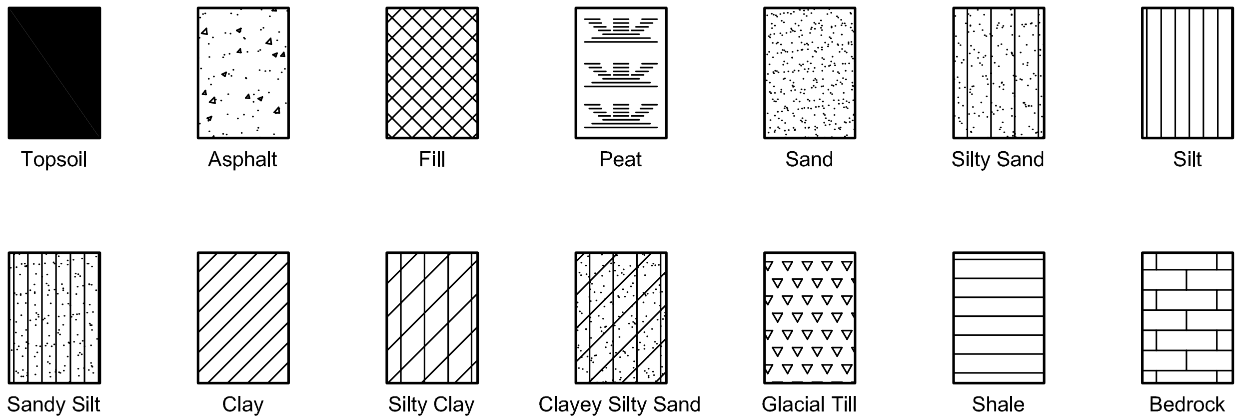
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

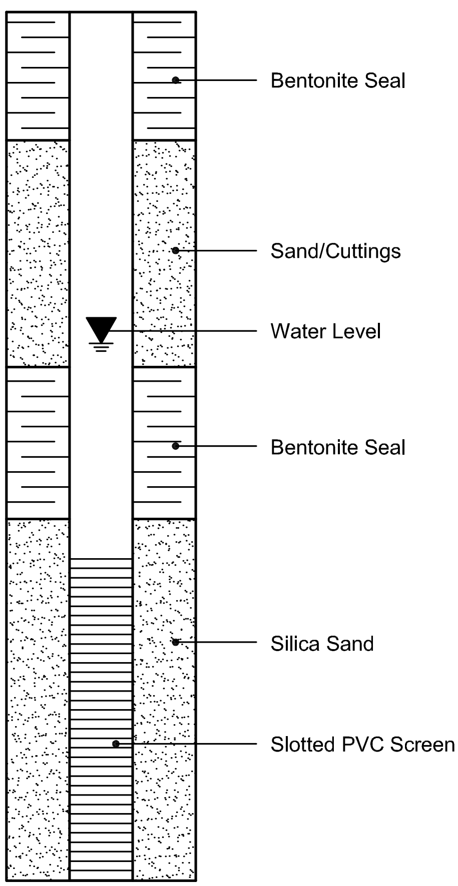
SYMBOLS AND TERMS (continued)

STRATA PLOT

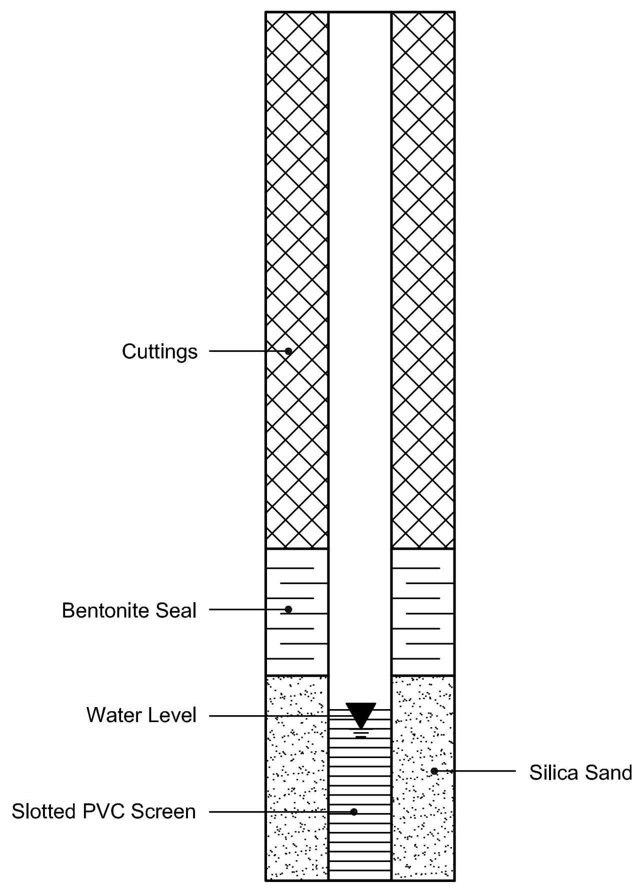


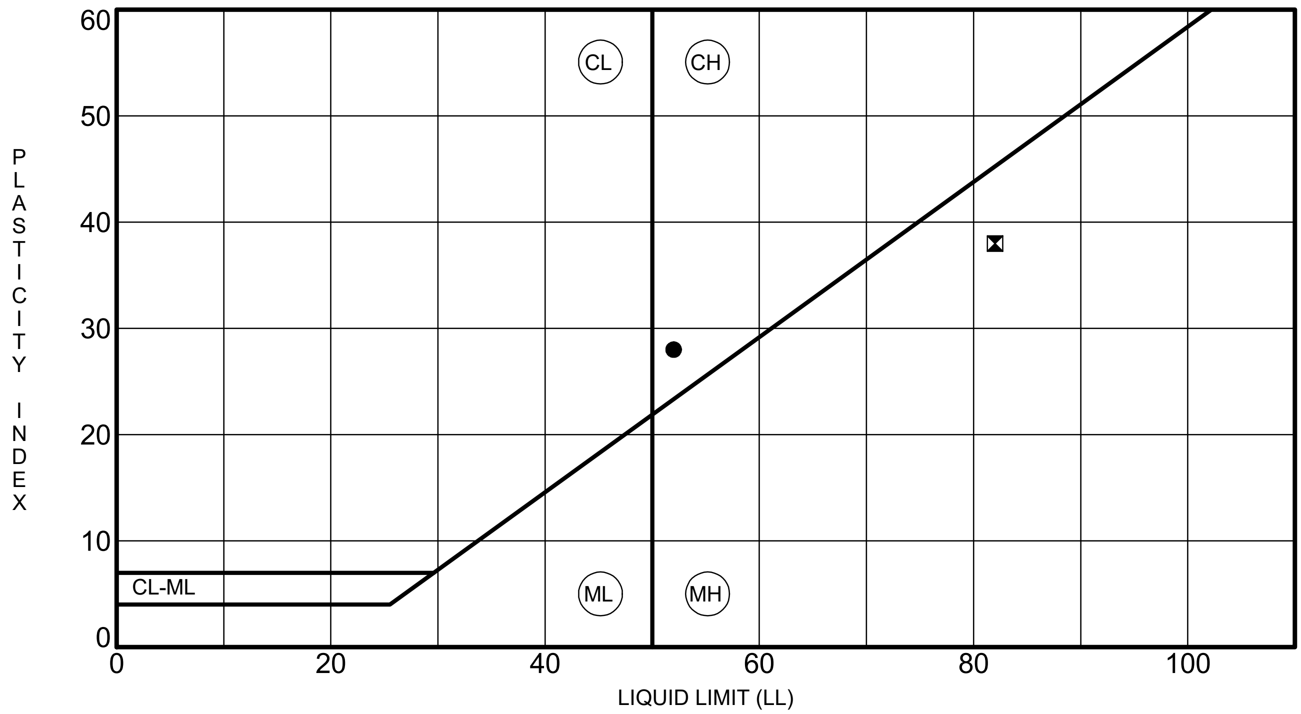
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 2-25**	**SS2**	**52**	**24**	**28**		**CH - Inorganic clays of high plasticity**
⧗	**BH 3-25**	**SS3**	**82**	**44**	**38**		**MH - Inorganic silts of high plasticity**

CLIENT **Trinity Development Group Inc.**

PROJECT **Geotechnical Investigation -**
2300 Bank Street, Ottawa, Ontario

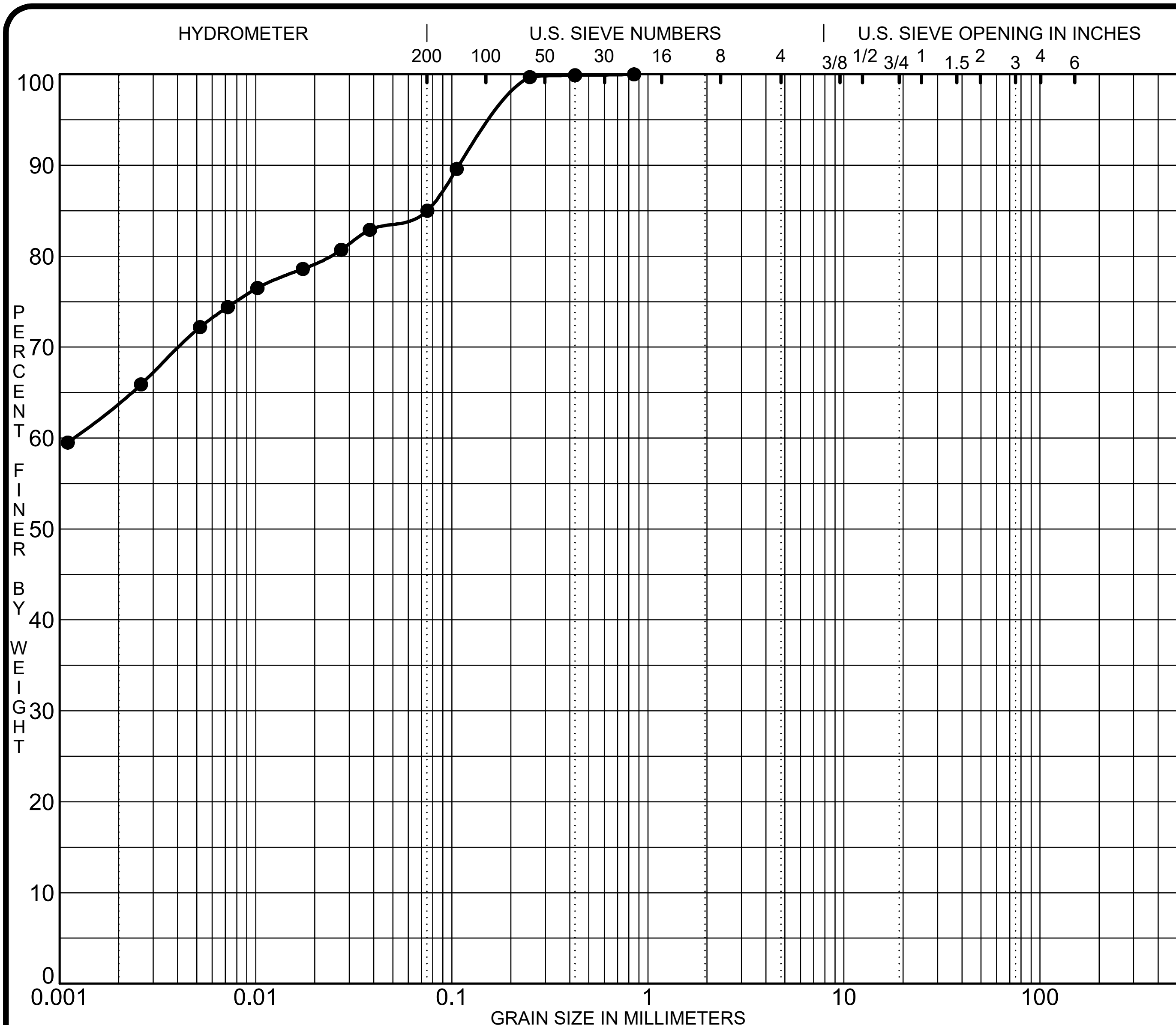
FILE NO. **PG7467**

DATE **21 Mar 25**

PATERSON GROUP

9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

ATTERBERG LIMITS' RESULTS



CLAY	SILT	SAND			GRAVEL		COBBLES
		fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 3-25**	**SS3**	**MH - Inorganic silts of high plasticity**	**38.9**	**82**	**44**	**38**		
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 3-25**	**SS3**	**0.85**	**0.00**			**0.0**	**15.0**	**21.7**	**63.3**
⊠										
▲										
★										

CLIENT **Trinity Development Group Inc.**
FILE NO. **PG7467**

PROJECT **Geotechnical Investigation -**
2300 Bank Street, Ottawa, Ontario
DATE **21 Mar 25**

PATERSON GROUP

9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

GRAIN SIZE DISTRIBUTION

PATERSON GROUP

Linear Shrinkage
ASTM D4943-02

CLIENT:	Trinity Development GrouP	DEPTH	5' - 7'	FILE NO.:	PG7667
PROJECT:	2300 Bank Street, Ottawa, ON	BH OR TP No:	BH1-25 SS3	DATE SAMPLED	-
LAB No:	59084	TESTED BY:	C.P	DATE RECEIVED	17-Mar-25
SAMPLED BY:	K.S.	DATE REPORTED:	21-Mar-25	DATE TESTED	18-Mar-25

LABORATORY INFORMATION & TEST RESULTS

Moisture	No. of Blows(7)	Calibration (Two Trials)	Tin NO.(A1)	
Tare	4.92	Tin	4.76	4.77
Soil Pat Wet + Tare	60.78	Tin + Grease	4.92	4.92
Soil Pat Wet	55.86	Glass	43.24	43.24
Soil Pat Dry + Tare	34.24	Tin + Glass + Water	85.66	85.66
Soil Pat Dry	29.32	Volume	37.50	37.5
Moisture	**90.52**	**Average Volume**	**37.50**	

Soil Pat + String	29.44
Soil Pat + Wax + String in Air	34.56
Soil Pat + Wax + String in Water	12.08
Volume Of Pat (Vdx)	22.48

RESULTS:

Shrinkage Limit	**19.67**
Shrinkage Ratio	**1.753**
Volumetric Shrinkage	**124.186**
Linear Shrinkage	**23.591**

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

PARACEL LABORATORIES LTD.

Order #: 2512074

Certificate of Analysis

Report Date: 21-Mar-2025

Client: **Paterson Group Consulting Engineers (Ottawa)**

Order Date: 17-Mar-2025

Client PO: 62614

Project Description: PG7467

	Client ID:	BH1-25-SS3	-	-	-	-	-
	Sample Date:	14-Mar-25 09:00	-	-	-		
	Sample ID:	2512074-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	75.3	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.49	-	-	-	-	-
Resistivity	0.1 Ohm.m	3.6	-	-	-	-	-
Anions							
Chloride	10 ug/g	1410	-	-	-	-	-
Sulphate	10 ug/g	214	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 A - SECTION A-A - PROPOSED CONDITIONS - STATIC ANALYSIS

FIGURE 2 B - SECTION A-A - PROPOSED CONDITIONS - SEISMIC ANALYSIS

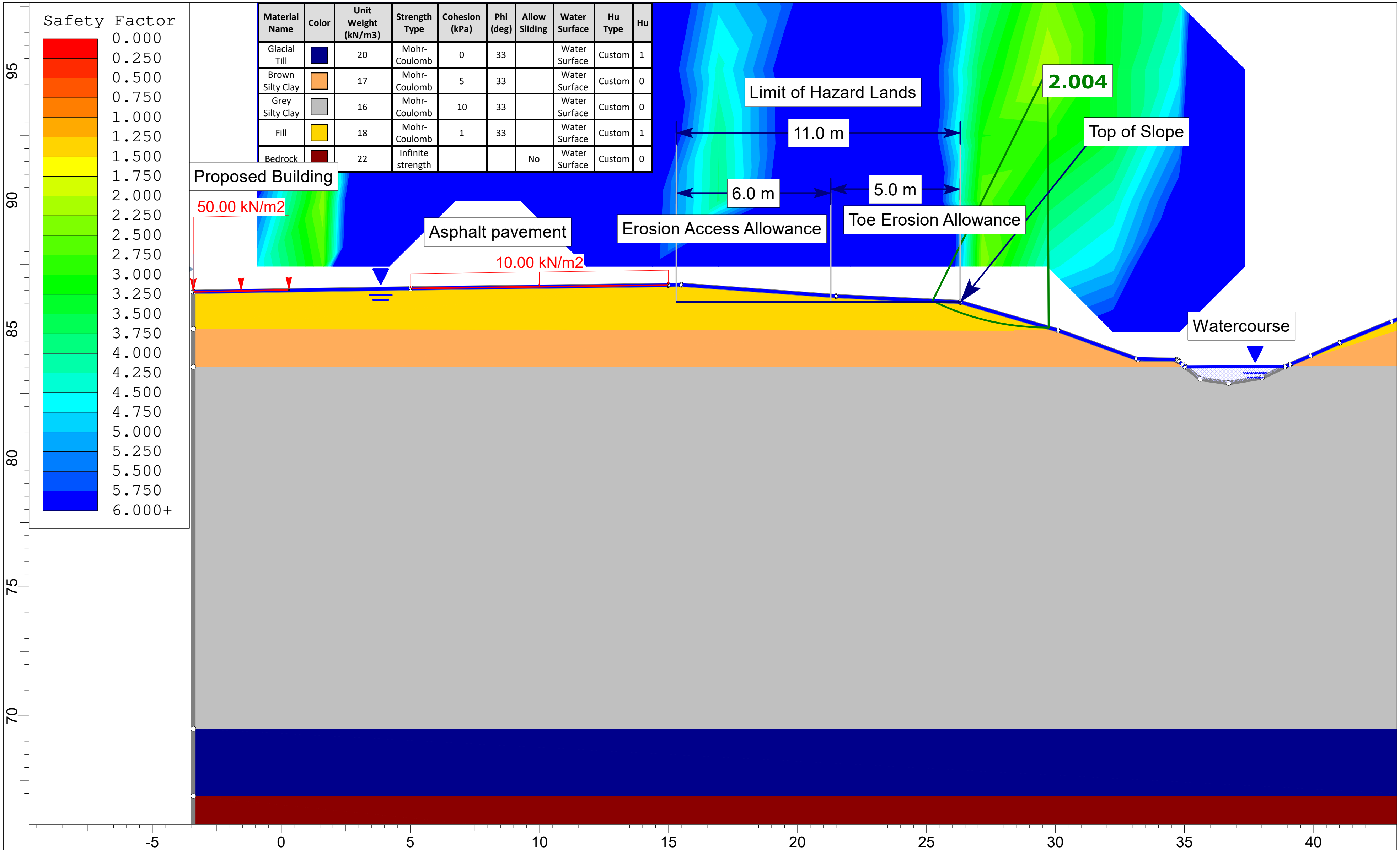
PHOTOS 1 TO 4 – PHOTOGRAPHS FROM SITE VISIT

DRAWING PG7467-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP

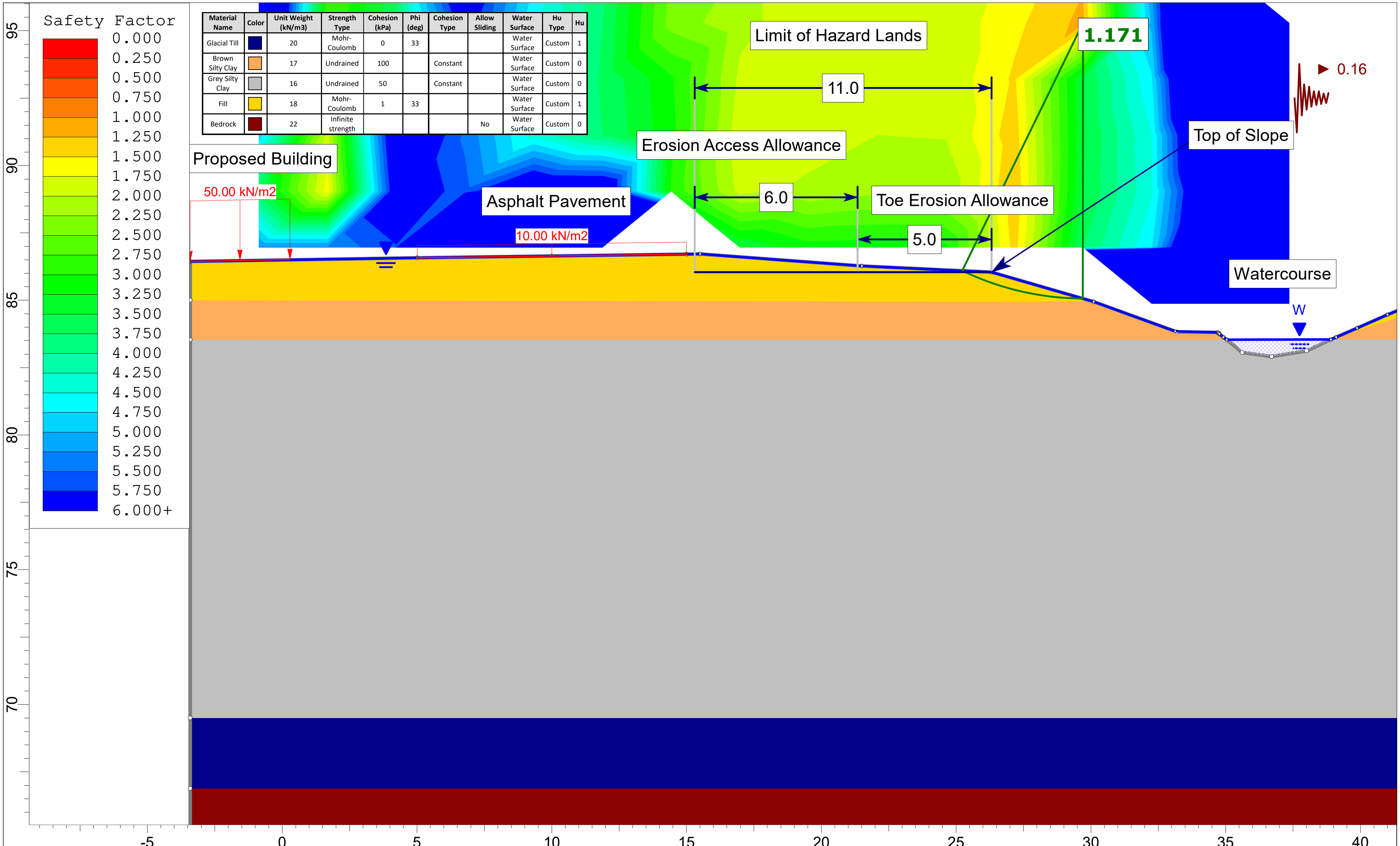


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu
Glacial Till		20	Mohr-Coulomb	0	33		Water Surface	Custom	1
Brown Silty Clay		17	Mohr-Coulomb	5	33		Water Surface	Custom	0
Grey Silty Clay		16	Mohr-Coulomb	10	33		Water Surface	Custom	0
Fill		18	Mohr-Coulomb	1	33		Water Surface	Custom	1
Bedrock		22	Infinite strength			No	Water Surface	Custom	0

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG7467 - 2300 Bank Street		
Slope Section	A	*Scenario*	Proposed Static Loading
Drawn By	DR	*Company*	Trinity Group
Date	2025-04-03	*File Name*	Figure 2A



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding	Water Surface	Hu Type	Hu
Glacial Till		20	Mohr-Coulomb	0	33			Water Surface	Custom	1
Brown Silty Clay		17	Undrained	100		Constant		Water Surface	Custom	0
Grey Silty Clay		16	Undrained	50		Constant		Water Surface	Custom	0
Fill		18	Mohr-Coulomb	1	33			Water Surface	Custom	1
Bedrock		22	Infinite strength				No	Water Surface	Custom	0

PATERSON GROUP

SLIDEINTERPRET 9.025

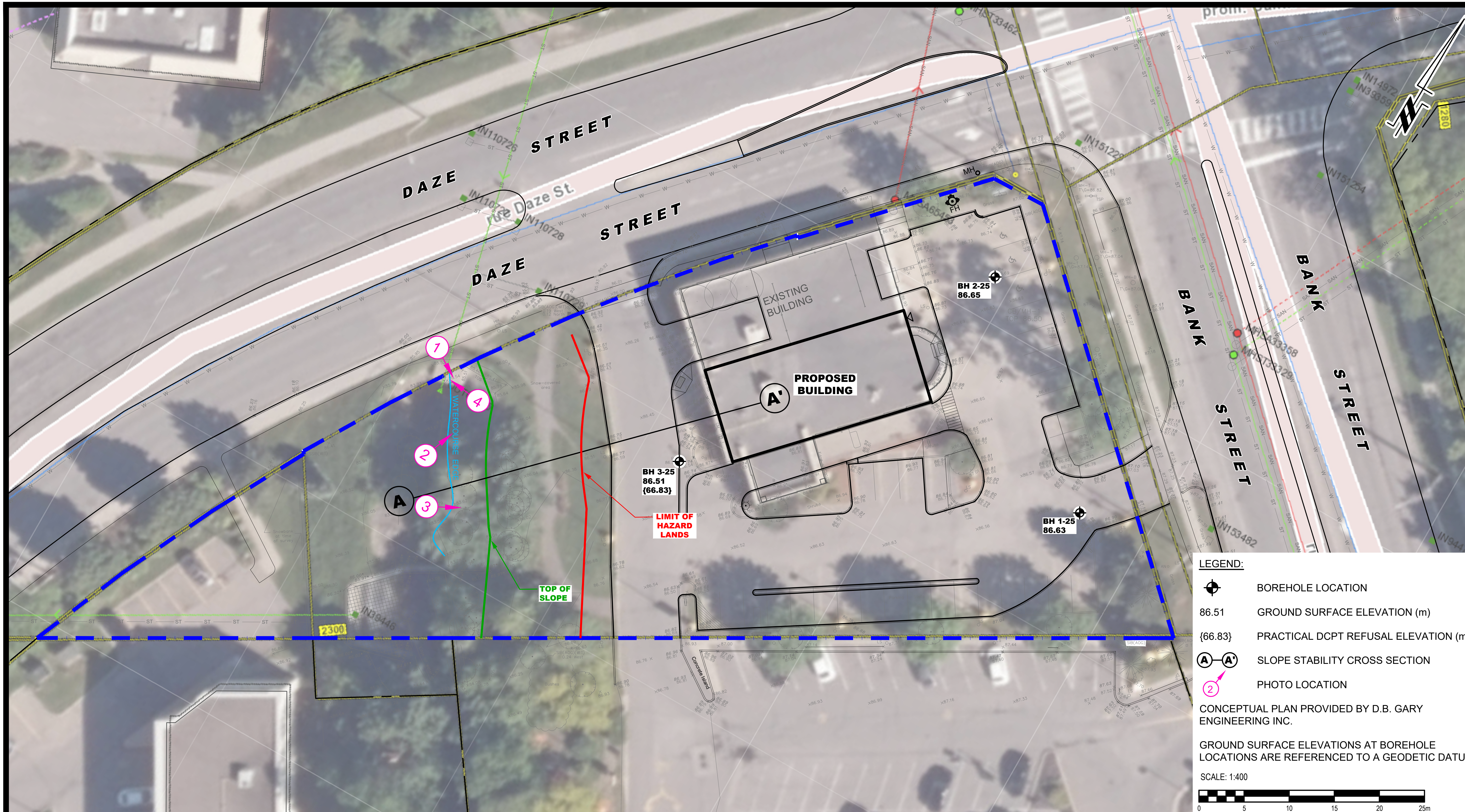
Project	PG7467 - 2300 Bank Street		
Slope Section	A	*Scenario*	Proposed Seismic Loading
Drawn By	DR	*Company*	Trinity Group
Date	2025-04-03	*File Name*	Figure 2B

Photo 1: Photograph of running water Creek and toe of slope taken at the northwestern portion of the site towards the south illustrating crushed stone riprap with few tress, grass and shrubs covered side slopes, no toe erosion was observed.

Photo 2: Photograph of creek, facing north illustrating crushed stone riprap with few tress, grass and shrubs covered side slopes.

Photo 3: Photograph of Creek, facing north east illustrating crushed stone riprap with few tress, grass and shrubs covered side slopes.

Photo 4: Photograph of creek with culvert, facing west illustrating crushed stone riprap with grass and shrubs covered side slopes.



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

NO.	REVISIONS	DD/MM/YYYY	INITIAL
1	UPDATED CONCEPTUAL PLAN	17/12/2025	NP

TRINITY GROUP
GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
2300 BANK STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 04/2025
Drawn by: YA	Report No.: PG7467-1
Checked by: NP	Dwg. No.: PG7467-1
Approved by: SD	Revision No.: 1

p:\autocad drawings\geotechnical\pg74xx\pg7467\pg7467-1-test hole location plan (rev.01).dwg

11x17