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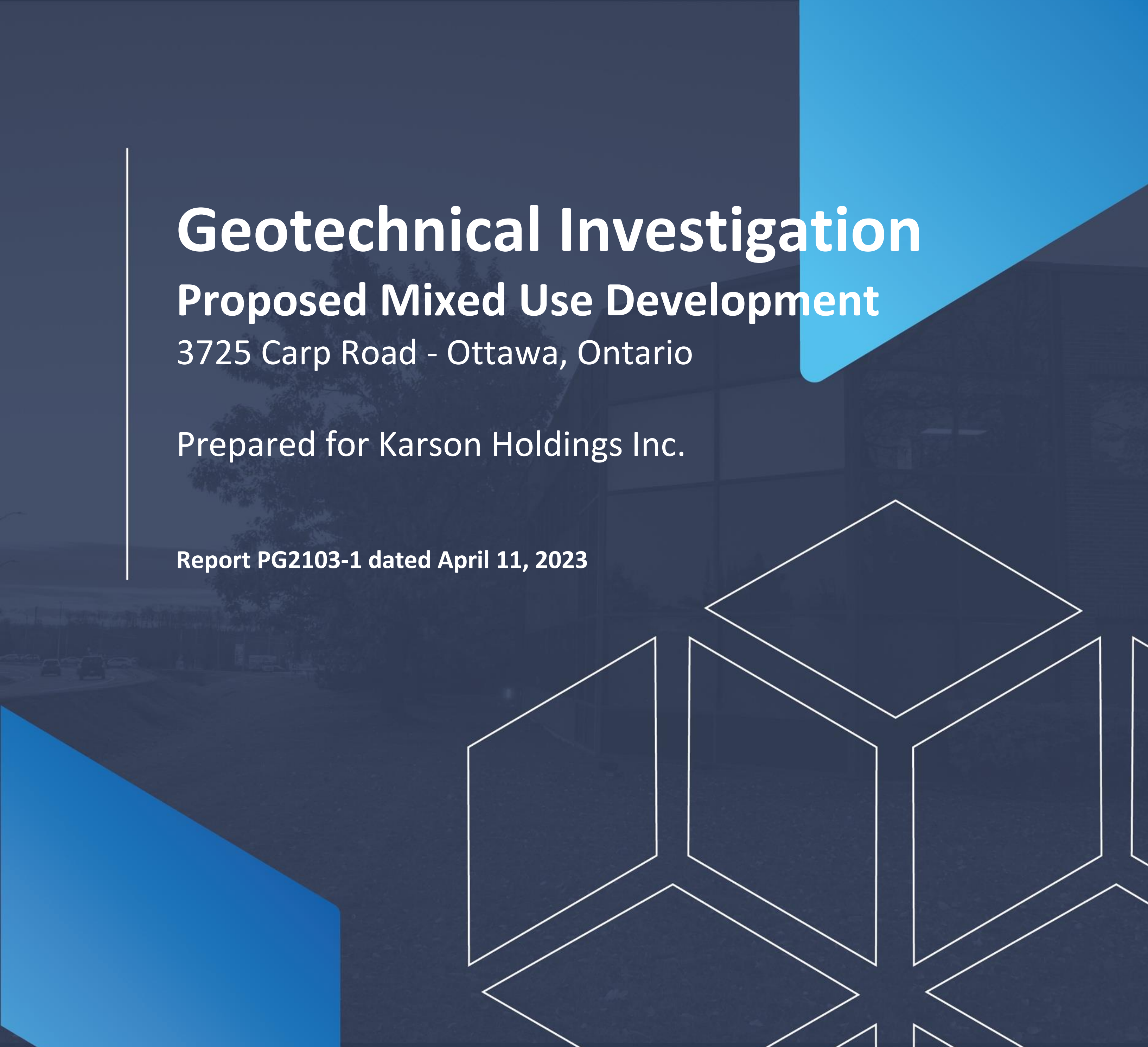

Geotechnical Investigation

Proposed Mixed Use Development

3725 Carp Road - Ottawa, Ontario

Prepared for Karson Holdings Inc.

Report PG2103-1 dated April 11, 2023

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Mr. Karson Holdings Inc. to conduct a geotechnical investigation for the proposed mixed-use development to be located at 3725 Carp Road, in the City of Ottawa, Ontario (reference should be made to Figure 1 - Key Plan presented in Appendix 2).

The investigation objective was to:

- Determine the subsurface soil and groundwater conditions by means of test holes.
- Provide geotechnical recommendations pertaining to 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. The report contains the findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as 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

The proposed development is understood to consist of a series of stacked unit structures of slab-on-grade construction with mixed-use residential/commercial blocks anticipated throughout several blocks. Associated paved parking, access lane areas and landscaped areas are also expected. It is expected that the proposed buildings will be municipally serviced.

It is also understood that previously existing buildings located throughout the subject site have been demolished at the time of preparing this report.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out between January 5 to January 10, 2023. At that time, 10 boreholes were advanced to a maximum depth of 8.2 m below the existing ground surface. A previous investigation by Paterson was undertaken on the subject site on October 18 and 19, 2010. During that time, a total of 10 boreholes were advanced to a maximum depth of 8.9 m below ground surface. The subsurface soil profiles encountered by Paterson are presented on the Soil Profile and Test Data Sheets in Appendix 1.

The test hole locations were placed in a manner to provide general coverage taking into consideration site features and underground utilities. The test hole locations for the current investigation are presented on Drawing PG2103-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track-mounted power 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 from the geotechnical division. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In-Situ Testing

Soil samples were recovered from the auger flights or collected using a 50 mm diameter split-spoon sampler. All soil samples were classified on site, placed in sealed plastic bags and transported to the laboratory for further review. The depths at which the auger and split-spoon samples were recovered from the test holes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Testing (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 sample 300 mm into the soil after a 150 mm initial penetration with a 63.5 kg hammer falling from a height of 760 mm.

The overburden thickness was evaluated during the investigation by dynamic cone penetration test (DCPT) completed at boreholes BH 1-23, BH 2-23 and BH 8-23. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip and 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 every 300 mm.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils and as based on our assessment of subsurface conditions at the time of the current field investigation. It should be noted that several intervals were not assessed using a vane apparatus, however, the frequency of testing was considered satisfactory for the nature of the deposit and consistency of the cohesive soil encountered at the time of our investigation.

Due to the preliminary nature of the previous investigation undertaken in 2010, undrained and remolded shear strength values of cohesive soils were not measured at that time.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1.

Groundwater

Monitoring wells were installed at BH 1-23, BH 2-23, BH 3-23, BH 6-23, BH 7-23, BH 8-23 and BH 9-23, and flexible polyethylene standpipes were installed within the remaining boreholes of the current investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Flexible polyethylene standpipes were installed in BH 4, BH 5, BH 8, BH 9 and BH 10 of the previous investigation, while the remaining boreholes were assessed visually within the open borehole column prior to being backfilled. It should be noted that piezometers installed during the previous investigation are no longer present throughout the subject site. Given this, groundwater levels could not be measured at the previous test hole locations provided with a piezometer.

Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1 of this report.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- Slotted 32 mm diameter PVC screen at the base of each borehole.
- 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No.3 silica sand backfill within annular space around screen.
- Bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

Sample Storage

All samples from the current investigation will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless directed otherwise.

3.2 Field Survey

The test hole locations were selected in the field by Paterson personnel to provide general coverage of the subject site with consideration to existing site features. The borehole locations and ground surface elevations were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The test hole locations and ground surface elevation at each test hole location are presented on Drawing PG2103-1 – 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 logs. A total of two (2) grain size distribution analysis tests, two (2) Atterberg limit test, and one (1) shrinkage test were completed on selected samples. The results of the testing are discussed in Section 4.2 and are provided 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 results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is generally vacant with the exception of a shed located towards the west boundary of the subject site. It should be noted that historical aerial photographs of the site indicate a series of buildings were previously located throughout the area and subject site. Figure 2 to Figure 7, included in Appendix 2 of this report, depict the historical photographs of the subject site.

The ground surface throughout the subject site is currently relatively flat with a slight downslope towards the west and south portion of the subject site. The ground surface along the north and east property boundary generally matched the grade of the surrounding roadways and properties, whereas the western and southern portions were approximately 500 to 800 mm lower than the adjacent roadways. The site is bordered by Donald B. Munro Drive to the north, and Carp Road to the east, Carp River along the south and west and further by undeveloped lands.

Based on the historical photos and field observations, a drainage ditch was present between the 50s to the 70s within the central portion of the site. The ditch was then replaced by a concrete storm pipe and backfilled to accommodate new developments. The abandoned, existing storm pipe was noted along the central portion of the slope face and expected to have been discharging its contents into the Carp River. The pipe is currently abandoned but is expected to be insufficiently decommissioned. It should further be noted that the slope was excavated to accommodate the placement of the storm pipe which created the existing bulge in the slope face.

Due to the presence of Carp River bordering the subject site, a slope stability assessment was carried out considering the slope conditions present in the subject site and along the sidewalls of the aforementioned watercourse. The results of the slope stability assessment are discussed further in Subsection 6.8 of this report.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations, consisted of fill underlain by a layer of peat or topsoil which was observed to be underlain by a layer of silty clay at some test hole locations and further by a deposit of silty sand.

The silty sand was observed to be underlain by a deposit of glacial till at the locations of BH 1-23. Glacial till was also observed at BH 4 and BH 10.

The fill material was observed to generally consist of varying amounts of crushed stone, brown to grey silty sand and/or silty clay, and trace amounts of organics, gravel, bricks and/or asphaltic concrete. The fill was observed to extend to depths ranging between 0.6 and 4.5 m below the existing ground surface.

The fill material was underlain by peat or topsoil at several borehole locations. The peat layer was observed to be approximately 0.6, 1.2, 1.4, 1.4 and 2.0 m thick at BH 10, BH 6-23, BH 7-23, BH 9 and BH 10-23, respectively. Peat was encountered at the deepest sampling interval of BH 10-23, BH 7 and BH 8, however, was not investigated further.

The silty clay layer generally consisted of a very stiff to firm, brown to grey silty clay with varying amounts of occasional sand and gravel. The silty clay layer was observed to extend to depths ranging between 1.5 and 6.7 m below existing ground surface.

The silty sand deposit was generally observed to consist of a loose to compact, brown to grey silty sand with trace amounts of clay and gravel. A running sand condition was noted at most boreholes at depths ranging between 2.3 and 8.2 m below ground surface. The silty sand deposit was observed to be underlain by a glacial till deposit at BH 1-23. The glacial till deposit generally consisted of a compact, grey silty sand with gravel and occasional cobbles.

Reference to the Soil Profile and Test Data sheets in Appendix 1 for details of the soil profiles encountered at each test hole location.

Laboratory Testing Results

Atterberg Limits and Moisture Content Test

Atterberg limits testing, as well as associated moisture content testing, was completed on the recovered silty clay samples at selected locations throughout the subject site. The results of the Atterberg limits are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results

Test Hole	Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 4-23	SS6	4.11	59	14	45	CH
BH 5-23	SS5	3.35	44	18	26	CL

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; CL: Inorganic Clay of Low Plasticity; CH: Inorganic Clay of High Plasticity

The results of the moisture contest test are presented on the Soil Profile and Test Data Sheet in Appendix 1.

Shrinkage Test

The results of the shrinkage limit test indicate a shrinkage limit of 17.3 and a shrinkage ratio of 1.85.

Grain Size Distribution Results

Two (2) sieve analysis tests were completed by Paterson to classify the selected soil sample according to the Unified Soil Classification System (USCS). The results are summarized in Table 2.

Table 2 – Summary of Grain Size Distribution Analysis

Test Hole	Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 3-23	SS7	4.8	10.2	86.1	3.6	0.0
BH 9-23	SS8	5.6	0.0	43.8	29.2	27.0

Reference to the Soil Profile and Test Data sheets in Appendix 1 for details of the soil profiles encountered at each test hole location.

Remediation by Others

It is understood that an environmental remediation program was completed between September and October 2016 throughout the subject site. The aforementioned remediation program consisted of excavating two areas of potential environmental contamination located on the north and west portion of the subject site to depths ranging between 2 and 3 m below ground surface. It is understood the lower-most sections of these excavations were in-filled with 6 inch-plus sized rock and river stone followed by environmentally clean soil generated from the site clean-up excavations.

Reference should be made to the Remediate of Petroleum Hydrocarbon Impacted Soil report prepared by Amec Foster Wheeler Environment & Infrastructure dated

November 2016 for additional information regarding those works and their associated subsurface profile findings.

Bedrock

Practical refusal to DCPT was encountered at BH 1-23, BH 2-23 and BH 8-23 at depths of 10.6, 24.3 and 23.4 m, respectively. Based on available geological mapping, the bedrock in the subject area consists of Paleozoic interbedded limestone and shale of the Verulam formation, with an overburden drift thickness of 15 to 50 m depth.

4.3 Groundwater

Groundwater levels were measured on January 17, 2023, within the monitoring wells and piezometers installed as part of the current investigation. Measurements from the previous investigation have also been summarized below. The measured groundwater levels are presented in Table 3.

Table 3 – Summary of Groundwater Level Readings

Test Hole Number	Observation Method	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
			Depth (m)	Elevation (m)	
BH 1-23	Monitoring Well	93.29	1.59	91.70	January 17, 2023
BH 2-23	Monitoring Well	93.32	1.74	91.58	
BH 3-23	Monitoring Well	92.77	1.05	91.72	
BH 4-23	Piezometer	91.89	0.74	91.15	
BH 5-23	Piezometer	92.72	1.29	91.43	
BH 6-23	Monitoring Well	92.71	0.97	91.74	
BH 7-23	Monitoring Well	92.47	1.22	91.25	
BH 8-23	Monitoring Well	92.58	0.92	91.66	
BH 9-23	Monitoring Well	92.41	0.77	91.64	
BH 10-23	Piezometer	92.46	1.43	91.03	
BH 1	Open Hole	92.18	2.10	90.08	October 18, 2010
BH 2	Open Hole	90.81	Dry	N/A	
BH 3	Open Hole	92.18	Dry	N/A	
BH 4	Piezometer	92.52	1.31	91.21	October 25, 2010
BH 5	Piezometer	92.83	0.80	90.83	
BH 6	Open Hole	92.67	4.30	88.37	October 18, 2010
BH 7	Open Hole	92.58	Dry	N/A	October 19, 2010
BH 8	Piezometer	92.53	1.26	91.27	October 25, 2010
BH 9	Piezometer	92.67	5.28	87.39	
BH 10	Piezometer	92.95	0.90	92.05	
BH 11	Open Hole	93.85	Dry	N/A	October 19, 2010

Note: The ground surface elevation at each borehole location was surveyed by Paterson using a handheld GPS and was referenced to a geodetic datum

It should also be noted that running sand was encountered at sampling intervals between 3.0 to 6.5, 3.7 to 5.1, 2.3 to 5.1 and 3.8 to 8.2 at BH 1-23, BH 2-23, BH 3-23 and BH 8-23, respectively. Further, running sand was encountered at approximate depths of 3.5 and 3.9 m at BH 10 and BH 6-23, respectively.

Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximate elevations of **88.5 to 89.5 m** (Approximate depths ranging from **3 to 4 m** below 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.

The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered satisfactory for the proposed development, however, a ground improvement program and/or ground replacement techniques will need to be undertaken to provide adequate bearing conditions for the future buildings. In summary, four different kinds of bearing medium improvement methods would need to be considered throughout the subject site:

- **Method A – Shallow Removal of Existing Unsuitable Soil Layers**: In this scenario, an acceptable in-situ, undisturbed soil bearing medium (such as very stiff brown silty clay, compact brown silty sand and/or firm grey silty clay) is anticipated to be encountered within approximately 1 to 2 m below the existing ground surface. In this scenario, it would be recommended to place footings upon the native soil underlying the fill, topsoil and/or peat layers.

  This may be accomplished by either placing footings directly upon the soil layer (may require extending footings depths to extend below varying fill layer thickness) or placing footings upon a trench of lean concrete extending between a design USF and down to the suitable bearing layer. However, this methodology would not be feasible for areas below building slabs, as such, a different method would need to be considered for the overall building footprint below slab structures.

- **Method B – Deep Removal of Existing Unsuitable Soil and Subgrade Replacement**: Due to the presence of highly compressible layers of topsoil and peat located throughout the majority of the site, this material is recommended to be removed from below building footprints. It is anticipated dewatering by the use of well-points in advance of excavating would need to be undertaken prior to considering this option.

  Re-use of the existing material overlying the topsoil/peat layers may be considered provided a suitable subgrade replacement program designed and overseen by Paterson is undertaken. This would generally consist of removing the existing fill, segregating acceptable material for re-use as backfill, and then re-using the material in conjunction with several lifts of bi-axial geogrid spreading across the footprint of the replaced fill and beyond the building footprints. A 4 to 6 month settlement monitoring program would be recommended to be undertaken upon completion of the fill replacement program.

- **Method C – Ground Improvement Program Designed by Specialized Contractor (Areas Free of Topsoil and Peat at Depth)**: a ground improvement program, which is anticipated to consist of a combination of rapid compaction, dynamic compaction and/or dynamic replacement programs would be undertaken by a contractor that specializes in these programs. An example contractor that would undertake this type of ground improvement method would be Menard Canada.

  In this scenario, the ground improvement contractor would be designed by the specialized contractor and approved by Paterson prior to being undertaken. Once the ground improvement program is complete, soil improvement will be verified by means of pressuremeter and/or DCPT investigations, which would be undertaken by the contractor. Once the results have been verified and considered acceptable, the contractor will certify the improved soils suitability to support the proposed structures. Once certified, it is anticipated conventional construction techniques may be carried out for foundation and building pad construction throughout the building footprints.

- **Method D – Deep Foundations and Soil Replacement or Improvement Below Slabs**: Alternatively to improving bearing conditions for the foundations, consideration could be given to founding the structures using a deep foundation, such as end-bearing piles, extending to the bedrock surface.

  However, since the building slabs would be susceptible to long-term settlement of the underlying existing soil, consideration could be given to completing the replacement of the subgrade material as described in Method B or evaluating potentially feasible ground improvement strategies with a specialized contractor for the supporting the building slabs.

Areas where hardscaping, such as parking areas and access roads, where services would not be located, are proposed along areas underlain by fill and further by topsoil/peat may be improved as described in Method C. It is expected that the topsoil/peat soils below the existing fill materials can be left in place within these proposed parking areas. However, settlement due to decomposition of the organic layer is expected and periodic maintenance of the pavement surface is anticipated to be required.

Due to the groundwater level within the sand layer and proximity to the Carp River, it is expected that a significant groundwater in-flux would be observed for the installation of infrastructure below the groundwater table. Excavations within cohesive soils in close proximity to non-cohesive soils below the groundwater table as well as in non-cohesive soils have the potential for basal heave if groundwater is not controlled during excavation work.

Based on this, a series of well points may be required to control groundwater in-flow for service trenches that extend below the water table. This would need to be undertaken to mitigate high levels of infiltration and the potential for basal heave in excavations.

It is assumed that the excavations will be carried out within the confines of a fully braced steel trench box or other acceptable shoring systems designed by a qualified engineer to resist the design lateral earth pressures and the potential for basal heave.

Due to the presence of a silty clay layer, proposed grading throughout the subject site will be subjected to a permissible grade raise restriction. Our permissible grade raise recommendations are discussed in Subsection 5.3. It should be understood that the current grade raise recommendations do not consider the use of more than 1 m of crushed stone fill below building footprints. Therefore, should subgrade improvement measures and/or grade raise requirements consider more than 1 m of crushed stone fill below and around a building footprint, Paterson should review the applicability of the current restrictions and/or the reduce the recommendations provided herein accordingly.

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

5.2 Site Grading and Preparation

Stripping Depth

As noted in Subsection 5.1, several different methodologies could be considered at each building location. In the case that a ground improvement program would be undertaken, the existing fill is anticipated to remain in place throughout the subject site. For the other options, all topsoil and deleterious fill, such as those containing significant amounts of organic and inorganic material, should be removed from within the footprint of any settlement sensitive structures.

Fill Placement – Ground Improvement Program by Specialized Contractor

Once ground improvement techniques are completed, fill used for grading beneath the proposed building pads should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II and/or select subgrade material. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and concrete pads 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. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD.

Fill Replacement – Ground Improvement by Replacing Existing Fill

In-filling operations are anticipated to be completed using approved existing fill and imported fill. The fill should be free of significant amounts of organics, deleterious materials and approved by Paterson at the source location prior to importing to the subject site. Further, fill generated from building excavations at the subject should be reviewed at the time of excavating by Paterson personnel to determine suitability of those materials at that time.

Approved fill should be placed in maximum 300 mm loose lifts and compacted by suitably sized compaction equipment making several passes under dry conditions and in above freezing temperatures. In general, soil fill is recommended to compacted using a suitably sized vibratory sheepsfoot roller. It is expected groundwater infiltration will be encountered in deeper excavations.

Where issues arise placing soil fill in areas where groundwater is not controlled feasibly, larger imported crushed rock fill (i.e., 300 mm minus blast-rock) in conjunction with a separation layer would be recommended to be placed in specific areas to raise the subgrade surface. The fill layer between the depth of the excavation and the desired pre-grade height would be provided with several lifts of bi-axial geogrid.

The design of this fill replacement programs would be designed at a detailed stage of design and is only considered preliminary at this time. The program would require daily full-time inspections by Paterson personnel at the time of construction, if undertaken.

Fill Placement – Remainder of Site

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in maximum 300 mm thick loose lifts and 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 maximum 300 mm thick loose lifts to at least 95% of the material's SPMDD. The placement of subgrade material should be reviewed at the time of placement by Paterson personnel.

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

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. 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 paved areas should be compacted to at least 100% of its SPMDD.

Lean-Concrete Trenches

Alternatively, where unacceptable fill is encountered below proposed underside of footing level consideration could be given to trenching through the existing fill materials to an undisturbed, very stiff to firm silty clay and/or compact silty sand bearing surface as described in Method A of Subsection 5.1. A zero entry trench, with near vertical walls could be excavated and backfilled with a 17 MPa lean concrete to underside of footing. Additional details are provided in Subsection 5.3.

Compacted Granular Fill Working Platform (Piled Foundation)

Should the proposed structure be supported on a pile foundation, the use of heavy equipment would be required to install the piles (i.e., pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance to the underlying soil.

A typical working platform could consist of 600 mm of OPSS Granular B, Type II crushed stone placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in maximum 300 mm thick lifts. The subgrade below the platform should be covered with a woven geotextile liner such as Terrafix 200W or equivalent followed by a biaxial geogrid liner such as Terrafix TBX1500 or equivalent. The geotextile and geogrid liners will provide a reinforcement for the subgrade soils and will reduce differential settlements and rutting of the asphaltic layer.

Once the piles have been driven and cut off, the working platform can be re-graded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for basement slab structure.

5.3 Foundation Design

5.3.1 Conventional Shallow Footings (No Ground Improvement/Replacement Techniques)

For foundations extending to undisturbed native soil, the following bearing resistance values may be considered for the proposed preliminary design:

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed stiff brown silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, founded on an undisturbed firm grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **75 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **110 kPa**.

Footings founded on an undisturbed compact silty sand bearing surface can be designed using a bearing resistance value at SLS of **120 kPa** and a factored bearing resistance value at ULS of **180 kPa**.

A geotechnical resistance factor of 0.5 was applied to the above bearing resistance values at ULS. The bearing resistance value at SLS should be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil or approved fill bearing surfaces. 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, under dry conditions, prior to the placement of concrete for footings.

Proof Rolling and Subgrade Improvement for Loose Sand Below Footings

Where the sand bearing surface for footings is considered loose by Paterson at the time of construction, it would be recommended to proof roll the bearing surface prior to forming for footings. Improving the bearing surface compaction will provide a suitable sand bearing medium.

Depending on the looseness and degree of saturation at the time of construction, other measures (additional compaction, dewatering, mud-slab, sub-excavation and reinstatement of crushed stone fill) may be recommended to accommodate site conditions at the time of construction. However, these considerations would be evaluated at the time of construction by Paterson on a footing-specific basis.

Lean-Concrete In-Filled Trenches

Where in-situ, undisturbed suitable soil is encountered below the design underside of footing elevation, consideration may be given to lowering the bearing surface to a suitable soil bearing medium by placing the footings on a lean-concrete in-filled trench extending to suitable soil surface. Footings placed on a lean-concrete in-filled trench extending to suitable bearing medium may be designed using a bearing resistance value for the anticipated bearing medium. This may be accomplished by excavating near-vertical trenches to expose the underlying bearing surface and backfilling with lean concrete (minimum 15 MPa, 28-day compressive) to the design underside of footing level.

Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bearing surface. The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below 1.5 m depth). Once approved by Paterson, lean concrete can be poured up to the proposed founding elevation.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the subgrade surface. It is anticipated water infiltration will make this difficult where the subgrade surface is located below the groundwater table. Where water infiltration cannot be controlled using open sumps within the excavation footprints it is recommended to install a well point adjacent to excavation footprints to lower the water table in advance of sub-excavations.

The bearing capacity of the footings placed over concrete infilled trenches would follow the same bearing capacities provided for the native subgrade encountered at the bottom of the trench.

5.3.2 Fill Replacement Program

The design of the proposed fill replacement program would be undertaken at a later stage of design. However, footings would be placed upon an anticipated 1 to 6 m deep layer of replaced fill reinforced with several lifts bi-axial geogrid. The compaction program would be overseen on a full-time basis by Paterson personnel. Further, a settlement monitoring program would be undertaken for a duration of approximately 4 to 6 months upon completion of the fill replacement program before footings may be constructed. It would be anticipated that fill would be raised up to 300 mm above the future finished floor elevation for each building using this methodology. Different verification methods suggested by the ground improvement designers may also be considered.

Footings founded on the approved replaced fill bearing surface would be able to be designed using a bearing resistance value at SLS of **130 kPa** and a factored bearing resistance value at ULS of **200 kPa**. A geotechnical resistance factor of 0.5 was applied to the above bearing resistance value at ULS. The bearing resistance value at SLS should be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

5.3.3 Shallow Foundation (After Ground Improvement Techniques)

For foundations being placed in areas where ground improvement techniques would be employed and certified by a specialized contractor may consider the following for preliminary design:

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on silty clay and silty sand fill can be designed for a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) **150 kPa** can be used for footings placed on an engineered fill pad over a stiff silty clay/silty sand fill bearing surface upon successful completion of a ground improvement program.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

5.3.4 End Bearing Driven Piled Foundation

The following alternative is relatively expensive to the aforementioned options and has been provided for information purposes at this time.

A deep foundation method, such as end bearing piles, may be considered for the foundation support of the proposed buildings given the presence of fill, peat and topsoil throughout the subject site.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 3. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 4 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	925	1100	9	27
245	11	1050	1250	9	31
245	13	1200	1400	9	35

Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving. A full-time field review program should be conducted during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical).

The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

5.3.5 Remaining Considerations for Foundation Design

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 soil bearing medium when a plane extending vertically and horizontally from the footing perimeter, at a minimum of 1.5H:1V, only through in situ soil or engineered fill of the same or higher capacity as the soil.

Permissible Grade Raise Recommendations

Our permissible grade raise recommendations for the proposed mixed-use development are presented on Drawing PG2103-2 Permissible Grade Raise Plan in Appendix 2. A post-development groundwater lowering of 0.5 m was assumed for our calculations. Grade restrictions only apply in areas where structures and infrastructure are supported by the underlying clay deposit.

The current grade raise recommendations do not consider the use of more than 1 m of crushed stone fill below building footprints. Therefore, should subgrade improvement measures and/or grade raise requirements consider more than 1 m of crushed stone fill below and around a building footprint, Paterson should review the applicability of the current restrictions and/or the reduce the recommendations provided herein accordingly.

If higher than permissible grade raises are required, lightweight fill would be advised to be used around and throughout building footprints to accommodate proposed grading to mitigate unacceptable long-term post-construction total and differential settlements. This would be determined at the grading plan review stage.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class E** for the foundations considered. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Slab on Grade Construction

With the removal of all topsoil, soils containing significant amounts of organics and/or deleterious fill within the proposed building footprint, undisturbed, native soil surface or compacted fill approved by Paterson and/or the surface prepared by the ground improvement contractor will be considered an acceptable subgrade surface on which to commence backfilling for floor slab construction.

Any soft or loose areas should be removed and backfilled with appropriate backfill material, such as OPSS Granular B Type II placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the materials SPMDD. The upper 300 mm directly below the concrete slab should consist of OPSS Granular A crushed stone compacted to a minimum of 98% of the materials SPMDD.

5.6 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be designed for car only parking areas and access lanes.

Table 5 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 6 - Recommended 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
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either approved in situ sand or OPSS Granular B Type I or II material placed over in situ soil.	

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

It is recommended to proof-roll the subgrade surfaces using a sheepsfoot roller prior to placing the above-noted subbase layers where the subgrade consists of existing in-situ fill. 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.

Due to the varying overlying fill and peat/topsoil material throughout this site, consideration should be given to using a geogrid as a reinforcing layer between the base and subbase layers to improve the longevity of the pavement structure over these subgrades. The geogrid layer should consist of a Terrafix TBX2500. The placement of this layer should be reviewed at the time of construction by Paterson personnel.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter foundation drainage system is considered optional, due to the native in situ soils, for the proposed structure(s). It is recommended to consider the implementation of a foundation drainage system in areas of hardscaping against building footprints. If considered, the system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The majority of the site excavated materials could be placed as backfill against the foundation walls provided the site excavated material is approved by the geotechnical consultant at the time of construction. The native material should be tested prior to placement to ensure the material meets a free draining non frost susceptible material. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, can also be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection. The recommended minimum thickness of soil cover is 2.1 m (or equivalent).

6.3 Excavation Side Slopes

Excavations will be through fill, peat, silty clay and/or loose to compact silty sand. The side slopes of excavations in the overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

Sufficient room is anticipated to be available for the greater part of the excavation to be completed by open-cut methods (i.e., unsupported excavations). Temporary shoring may be required where sufficient space is unavailable.

Above the groundwater level, for excavations to depths of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Shallower slopes should be provided for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be constructed.

The slopes recommended above are only for temporary slopes. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be maintain a safe working distance from the excavation. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

The subsurface soil is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavation Base Stability

The base of supported excavations can fail by three (3) general modes:

- ☐ Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,
- ☐ Piping from water seepage through granular soils, and
- ☐ Heave of layered soils due to water pressures confined by intervening low permeability soils.

Shear failure of excavation bases is typically rare in granular soils if adequate lateral support is provided. Inadequate dewatering can cause instability in excavations made through granular or layered soils. The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems.

The factor of safety with respect to base heave, FS_b, is:

$$FS_b = N_b s_u / \sigma_z$$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 below.
s_u - undrained shear strength of the soil below the base level
σ_z - total overburden and surcharge pressures at the bottom of the excavation

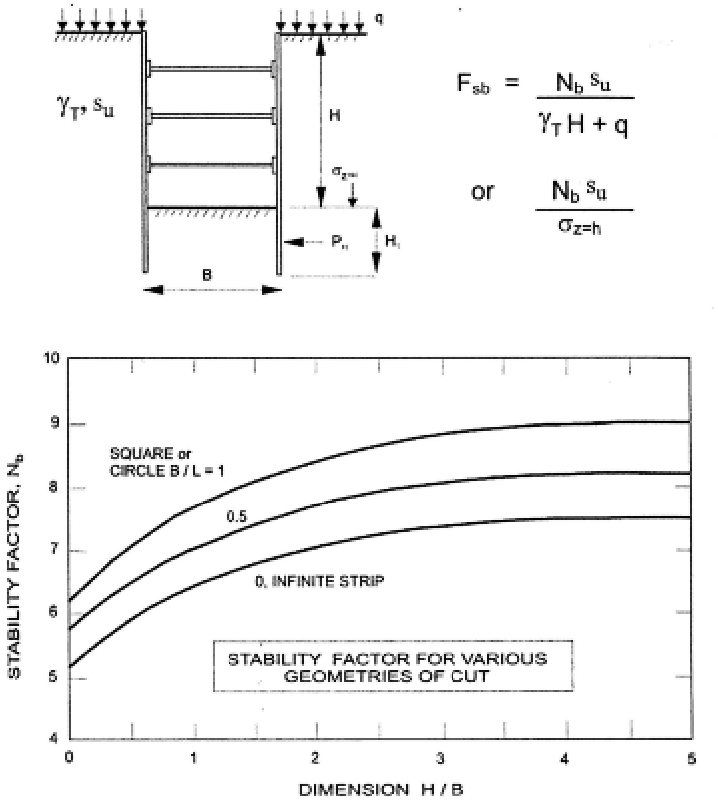

Figure 1 - Stability Factor for Various Geometries of Cut

In the case of soft to firm clays, a factor of safety of 2 is recommended for base stability.

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 City of Ottawa.

The pipe bedding for sewer and water pipes placed on a relatively dry , undisturbed subgrade surface should consist of a minimum of 150 mm of OPSS Granular A material. If the bedding is located within a layer of grey silty clay the thickness of the bedding material should be increased to a minimum of 300 mm.

The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

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

Generally, it should be possible to re-use the moist (not wet) brown silty clay and silty sand above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay material will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period. Peat and topsoil is not recommended for re-use below pavement structures.

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 with a sheepsfoot roller.

6.5 Groundwater Control

Due to the permeable silty sand to sandy silt deposit encountered within the groundwater table throughout the subject site, it is anticipated that conventional pumping with open sumps will be difficult to control the groundwater influx through the sides of the temporary excavations extending in proximity to and below the groundwater table.

It is recommended that a dewatering program, such as a series of well points designed and installed by a licensed contractor specializing in dewatering, be completed for service installations in proximity to and below the groundwater level.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbances 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

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

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

The trench excavations should be constructed to avoid the introduction of frozen materials, snow or ice into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving during construction. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

6.7 Corrosion Potential and Sulphate

The analytical results indicate the sulphate content is less than 0.1%. These results are indicative that Type 10 Portland cement (normal cement) would be appropriate. The results of the chloride content, pH and resistivity indicate the presence of a non-aggressive to slightly aggressive environment for exposed ferrous metals.

6.8 Landscaping Considerations

Tree Planting Restrictions

Paterson completed a subsurface soils review of the site to determine applicable tree planting setbacks, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines).

Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. Grain size distribution and hydrometer testing was also completed on selected soil samples. The above-noted test results were completed on samples taken at depths between the anticipated underside of footing elevation and a 3.5 m depth below finished grade. The results of our testing are presented in Tables 1 and 2 in Subsection 4.2 and in Appendix 1.

Based on the results of our review, two tree planting setback areas have been established (Area 1 and Area 2). The recommended tree planting setbacks should be reviewed by Paterson once the proposed grading plan has been prepared. The two areas are detailed below and have been outlined on Drawing PG2103-3 - Tree Planting Setback Recommendations presented in Appendix 2.

Area 1 – Low to Medium Sensitivity Clays

A clay soil with a low to medium potential for soil volume change was encountered in the areas outlined on Drawing PG2103-3 - Tree Planting Setback Recommendations in Appendix 2. Based on our Atterberg limits test results, the modified plasticity index does not exceed 40% in these areas. The following tree planting setbacks are recommended for these areas:

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

- ❐ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied 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 as indicated procedural changes below.

❐ A small tree must be provided with a minimum 25 m^3 of available soil 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).

Area 2 - High Sensitivity Clays

A clay soil with a high potential for soil volume change was encountered in the areas outlined on Drawing PG6087-3 - Tree Planting Setback Recommendations in Appendix 2. Based on our Atterberg limits test results, the modified plasticity index generally equals or exceeds 40% in these areas. The following tree planting setbacks are recommended for these areas:

Large trees (mature height over 14 m) can be planted within these areas provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits are **7.5 m** for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m), provided that the following conditions are met:

❐ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied 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 as indicated procedural changes below.

❐ A small tree must be provided with a minimum 25 m^3 of available soil 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).

6.9 Slope Stability Assessment

The Carp River borders the south and west boundary of the subject site. The existing slope conditions within the sidewalls of the river were reviewed by Paterson field personnel as part of the geotechnical investigation on January 6, 2023. Four (4) slope cross-sections were studied as the worst-case scenarios. The cross-section locations are presented on Drawing PG2103-1 – Test Hole Location Plan in Appendix 2.

A slope stability analysis was carried out to determine the required geotechnical setback from the top of the slope based on a factor of safety of 1.5. Toe erosion and erosion access allowance were also considered in the determination of limits of hazard lands setback line and are discussed on the following sections.

Field Observations

The existing slope conditions along the south and west portions of the subject site are detailed below. Reference may also be given to photographs taken as part of our site review in Appendix 2.

Carp River was observed along the south and west site boundary. The general slope of the riverbank was observed to be approximately 2 to 3 m high and appeared to have a profile generally shaped 3H:1V with local sections shaped 2H:1V. The slope surface generally appeared to be heavily vegetated and consisted of grass and large, mature trees.

The Carp River adjacent to the subject site was observed to be up to approximately 10 m wide. The water flow rate was noted to be relatively high at the time of our visit. The majority of the watercourse bed appeared to be covered by sediment consisting of fine sand and gravel underlain by in-situ stiff, grey silty clay. Signs of active erosion were not identified along the river alignment at the time of our review.

As mentioned in Section 4.1, a bulge along the slope face can be observed to accommodate an existing and abandoned storm pipe. It is understood that the slope was excavated to accommodate the water flow through the existing storm sewer. Therefore, reinstatement of this area and proper decommissioning of the aforementioned pipe may be required depending on the development limits.

Slope Stability Analysis

The analysis of the stability of the upper slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several Limit Equilibrium Methods including but not limited to, the Bishop's and Morgen-Stern methods, which are widely used and accepted analysis method.

The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

Subsoil conditions at the cross-sections were inferred based on nearby boreholes completed during the aforementioned geotechnical investigation, our observations along the slopes during our site visits, and general knowledge of the area's geology. For a conservative review, the slope stability analysis was completed at each slope cross-section under worst-case-scenario by assigning cohesive soils under fully saturated, groundwater flow conditions. In developing the slope geometry, elevations were surveyed using a high-precision GPS unit. These elevations were used preferentially over contours, where present.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation. The effective strength soil parameters used for static analysis are presented in Table 7.

Table 7 – Effective Strength Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Fill Material	18	31	1
Peat	16	33	5
Grey Silty Clay	16	33	10
Silty Sand	20	35	0

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strength values recovered during the geotechnical investigation and based on our knowledge of the geology in the area. The strength parameters used for the seismic analysis at the slope cross-sections are presented in Table 8 below.

Table 8 – Total Strength Soil and Material Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
Fill Material	18	31	1
Peat	16	33	5
Grey Silty Clay	16	-	40
Silty Sand	20	35	0

Static Loading Analysis

The results are shown in Figures 8, 10, 12 and 14 in Appendix 2. The results indicate a slope with factors of safety exceeding 1.5 beyond the top of slope at section D. However, a slope with a factor of safety less than 1.5 was measured at Sections A, B and C at the existing slope.

Seismic Loading Analysis

An analysis considering seismic loading and the groundwater at ground surface was also completed. A horizontal acceleration of 0.16g was considered for all slopes. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the analysis including seismic loading are shown in Figures 9, 11, 13 and 15 in Appendix 2. The results indicate a slope with a factor of safety less than 1.1 beyond the top of slope for Sections A, B and C. However, a slope with a factor of safety exceeding 1.1 was measured at Section D.

Based on these results, a stable slope setback of 7, 7 and 9 m from the top of slope is required to achieve a factor of safety of at least 1.1 for the Limit of Hazard Lands in the area of Sections A, B and C, respectively. It should be noted that the stable slope setback associated with our seismic loading analysis governs the required stable slope setback required for static conditions for Sections A, B and C.

Toe Erosion and Erosion Access Allowances

Based on the soils observed at the time of our field review a toe erosion allowance of 5 m was applied from the watercourse edge. Further, an access allowance of 6 m is required from the top of slope or geotechnical setback.

Limit of Hazard Lands

Based on the above, a setback taken from the top of the current slope has been provided as based on the above-noted observations and analysis. Reference should be made to Drawing PG2103-1 – Test Hole Location Plan for the proposed Limit of Hazard Lands setback for development considerations at the subject site. The existing vegetation on the slope faces should not be removed as it contributes to the stability of the slope and reduces erosion.

It should be understood the construction of parking areas may be permitted within the currently proposed Limit of Hazard Lands setback area from a geotechnical perspective. The current setback may be reduced if considerations were given to providing erosion protection to the toe of slope and/or re-shaping the existing slope to achieve a minimum factor of safety of 1.5 and 1.1 in static and seismic loading conditions and assessed using the methodology described herein.

Re-shaping would generally consist of lessening the steepness of the existing slopes in conjunction with in-filling along the slope face with concentrated fill placed closest to the toe of slope. Additional details may be provided at a later date if these options were considered as part of the proposed design.

For the portion of the slope that recesses inwards towards the site and at the location of the previously abandoned private storm pipe outlet, consideration could be given to in-filling this section of the slope to reinstate the ground surface to match the surrounding grades. This would result in an adjusted Top of Slope alignment and further result in a modified Limit of Hazard Lands setback limit alignment throughout this area. The fill would generally be recommended to consist of cohesive backfill, such workable brown silty clay, placed in 300 mm thick loose lifts and compacted using a sheepsfoot roller.

This work would be recommended to be complete in the dry, as such, the area would be recommended to be dewatered and cut-off from influx of water from the Carp River. The subgrade is expected to be relatively saturated and covered with vegetation. As such, a provision would be recommended to be carried to remove the existing subgrade surface material to provide an adequate surface to place the fill upon.

Should high influxes of groundwater be experienced that would not be able to be controlled using conventional open sumps, a layer a of rip-rap and/or large blast-rock stone could be placed to raise the subgrade above the depth of groundwater influx. The large stone layer would be recommended to be surface with a geotextile layer to prevent migration of fines of the overlying fine-grained soil fill material.

The current Limit of Hazard Lands setback is based on existing conditions. Therefore, the proposed grading conditions will need to be evaluated using the methodology described herein.

Assessment of Retrogressive Landslide Potential

Based on our review, the subject site is not considered to be susceptible to retrogressive landslides. Landslides throughout the Ottawa area have been inventoried by the Geological Survey of Canada to have been triggered and have occurred in marine clay deposits. The most common natural, non-manmade, triggers for landslides in the Ottawa area consist of either earthquakes or bank erosion, or a combination of both of these triggers. Further, these types of landslides have historically only occurred in relatively deep marine clay deposits.

Due to the relatively thin layer of clay present between the existing fill/peat and underlying sand deposit, the in-situ clay layer is not considered to be sufficiently thick to be prone to a retrogressive landslide. Clay was not observed to form the sidewalls of the Carp River channel along the southern boundary of the subject site and is therefore not subject to erosion by the Carp River.

Further, despite the factors of safety being less than 1.1 for existing conditions in seismic loading, failure planes did not extend within the underlying clay deposit. This indicates the clay has sufficient shear strength to resist the loads imposed by an earthquake with a peak ground acceleration of 0.16g, which would imply the clay deposit would have a notably reduced potential for failure by an earthquake. The reduced potential for failure results in a significantly reduced potential for a retrogressive landslide to occur.

Should a slope failure occur within the clay deposit, the potential for retrogression was assessed by estimating the stability number, N_s, using the methodology outlined in *Flowsliding in Sensitive Soil* (Mitchell & Markell, 1974) and as detailed below:

$$N_s = yH/S_u$$

Where y = unit weight of the slopes overburden material (kN/m^3), H = bank height (m) and S_u = peak undrained shear strength (kPa). Analysis of forty landslides determined that N_s should be greater than or equal to 6 for the potential of retrogression to occur (Mitchell & Markell, 1974). Undrained shear strength was measured to range between 40 and 209 kPa and the slope height was estimated be between 1.5 and 2.4 m. Based on this, the worse-case scenario N_s values were estimated to range between 0.7 and 1.1 and are less than 6, which would not suggest the potential for a landslide to develop by retrogression.

Since there is no evidence of landslides being triggered in deposits of sand and/or peat throughout the Ottawa area, the potential for a retrogressive landslide to occur within these deposits is considered very unlikely and negligible from a geotechnical perspective. Based on this, since a slope failure within the silty clay deposit is not expected to occur by the two main triggers for landslides that have occurred throughout the Ottawa-area, it is not expected that the subject site would be susceptible to a retrogressive landslide triggered by a slope failure.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ➢ Review preliminary and detailed grading, servicing and structural plan(s) from a geotechnical perspective.
- ➢ Review of the geotechnical aspects of the ground improvement contractors proposed methodology, if applicable.
- ➢ Review of architectural plans pertaining to foundation drainage systems.

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

- ➢ Review and inspection of the installation of the foundation drainage systems and insulation surrounding the structures.
- ➢ Observation of all bearing surfaces prior to the placement of concrete.
- ➢ Sampling and testing of the concrete and fill materials.
- ➢ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ➢ Observation of all subgrades prior to backfilling.
- ➢ Field density tests to determine the level of compaction achieved.
- ➢ Sampling and testing of the bituminous concrete including mix design reviews.

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 soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The report recommendations are in accordance with the present understanding of the project. Paterson request permission to review the grading plan once available and recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and could only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The recommendations provided are intended for the design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractors construction methods, equipment capabilities and schedules.

The present report applies only to the project described in this document. This report for purposes other than those described herein or by person(s) other than Karson Holdings Inc. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Drew Petahtegoose, B. Eng.

Faisal I. Abou-Seido, P.Eng.

LICENSED PROFESSIONAL ENGINEER
April 11, 2023
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMITS TESTING RESULTS

ANALYTICAL TESTING RESULTS

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE January 5, 2023

FILE NO. PG2103

HOLE NO. BH 1-23

SOIL DESCRIPTION

GROUND SURFACE

TOPSOIL 0.20

FILL: Brown to grey silty sand, trace topsoil

- some gravel by 0.9m depth

1.83

Loose to compact, brown **SILTY SAND,** trace to some gravel

- some running sand from 3.0 to 6.5m depth

- grey by 4.1m depth

7.78

GLACIAL TILL: Compact, grey silty sand with gravel, occasional cobbles 8.23

Dynamic Cone Penetration Test commenced at 8.23m depth.

10.64

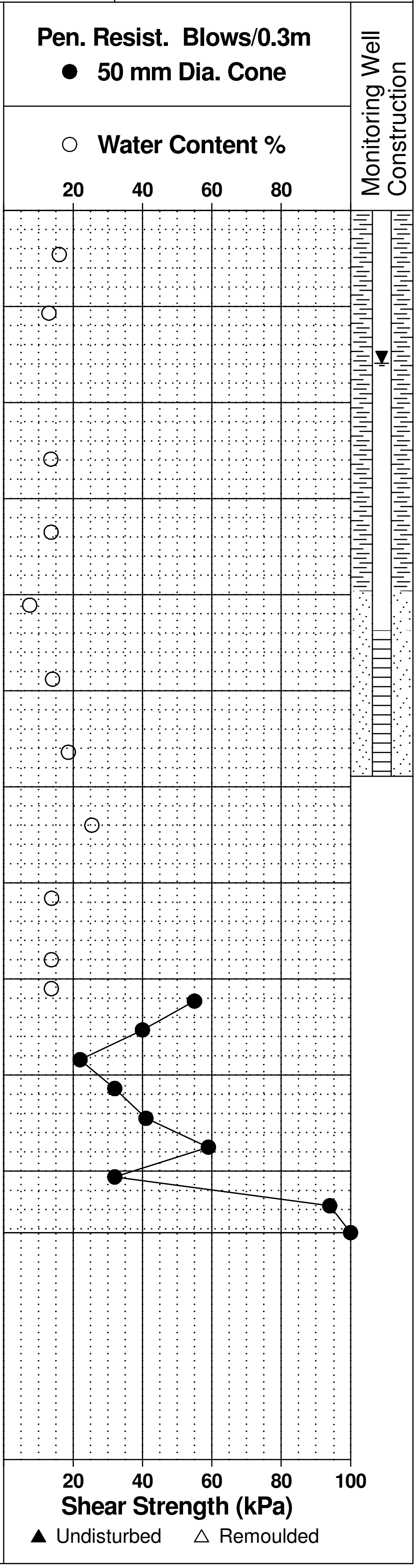
End of Borehole

Practical DCPT refusal at 10.64m depth.

(GWL @ 1.59m - Jan. 17, 2023)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	17	3
SS	3	0	4
SS	4	17	8
SS	5	17	7
SS	6	0	10
SS	7	4	8
SS	8	12	8
SS	9	63	17
SS	10	50	12
SS	11	50	20

DEPTH (m)	ELEV. (m)
0	93.29
1	92.29
2	91.29
3	90.29
4	89.29
5	88.29
6	87.29
7	86.29
8	85.29
9	84.29
10	83.29



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

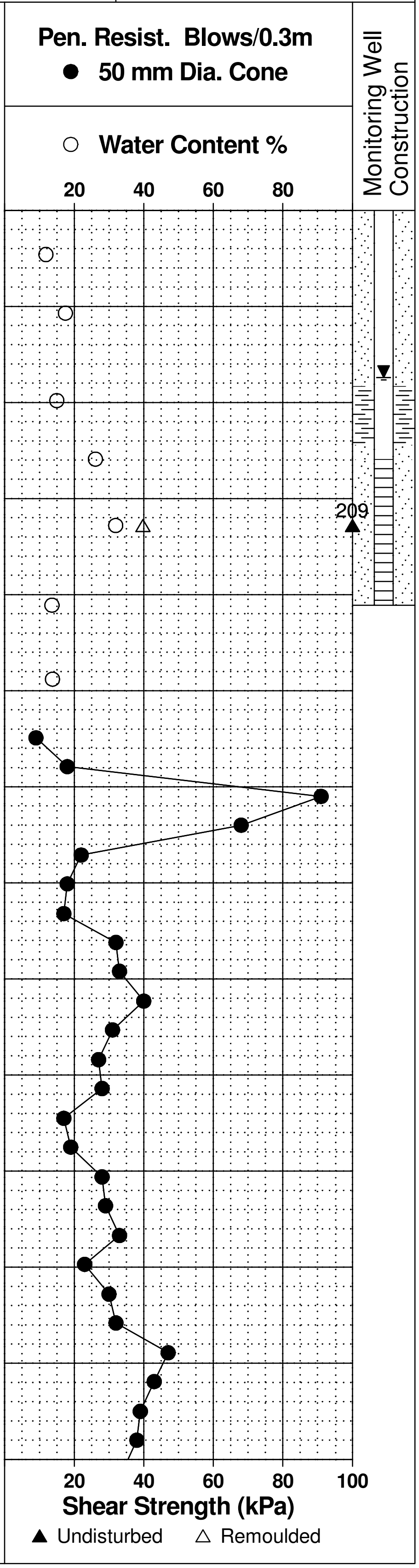
DATE January 5, 2023

FILE NO. PG2103

HOLE NO. BH 2-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.10				
FILL: Brown to grey silty sand, trace gravel	1.45	AU	1		
		SS	2	33	4
PEAT	1.68				
Compact, grey-brown **SILTY SAND**	2.21	SS	3	67	12
Hard, grey-brown **SILTY CLAY** with intermittent sand seams	3.66	SS	4	83	8
		G	5		
Compact, reddish brown **SILTY SAND,** some gravel - running sand from 3.7 to 5.1m depth	5.18	SS	6	75	17
		SS	7	58	16
Dynamic Cone Penetration Test commenced at 5.18m depth.					

DEPTH (m)	ELEV. (m)
0	93.32
1	92.32
2	91.32
3	90.32
4	89.32
5	88.32
6	87.32
7	86.32
8	85.32
9	84.32
10	83.32
11	82.32
12	81.32
13	80.32



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

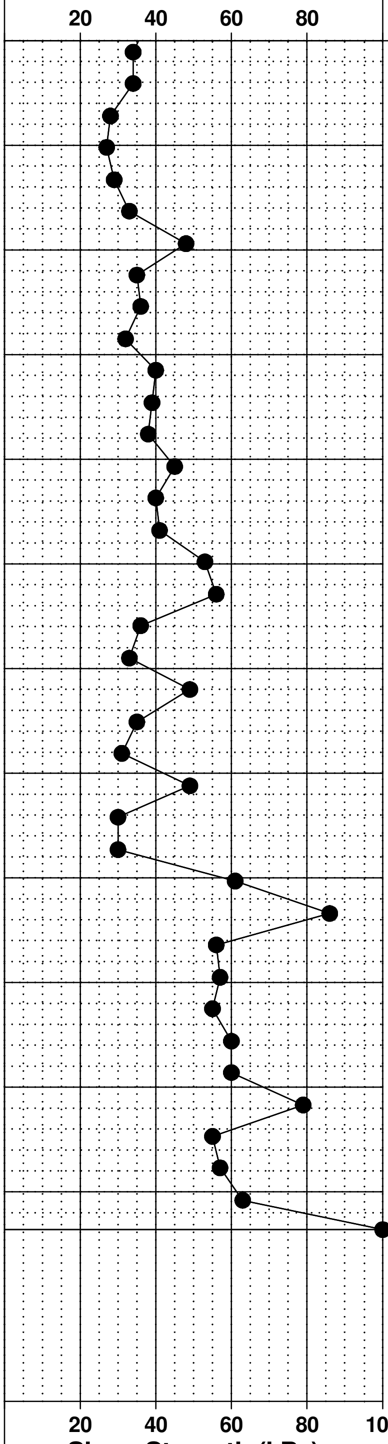
BORINGS BY Track-Mount Power Auger

DATE January 5, 2023

FILE NO. PG2103

HOLE NO. BH 2-23

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 %	Monitoring Well Construction
GROUND SURFACE									
Dynamic Cone Penetration Test commenced at 5.18m depth.						13	80.32		
						14	79.32		
						15	78.32		
						16	77.32		
						17	76.32		
						18	75.32		
						19	74.32		
						20	73.32		
						21	72.32		
						22	71.32		
						23	70.32		
						24	69.32		
24.36									
End of Borehole Practical DCPT refusal at 24.36m depth. (GWL @ 1.74m - Jan. 17, 2023)									



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE January 6, 2023

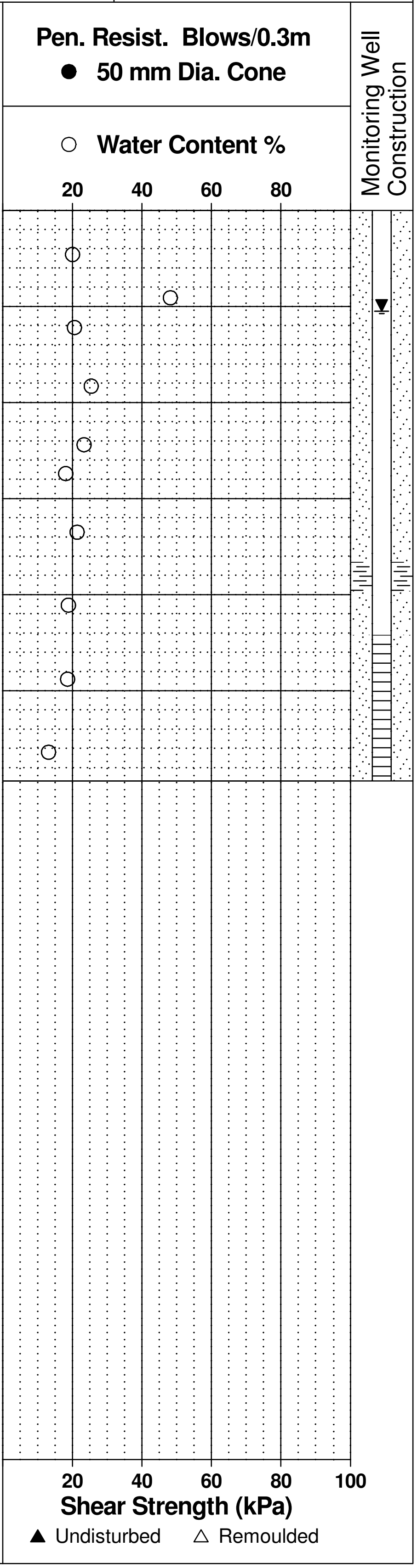
FILE NO. PG2103

HOLE NO. BH 3-23

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.30
FILL: Brown silty sand, trace clay and gravel	0.69
TOPSOIL	1.07
Very stiff, brown **SILTY CLAY,** trace sand and organics	1.52
Loose to compact, reddish brown **SILTY SAND,** trace clay - running sand from 2.3 to 5.1m depth - grey by 3.7m depth - gravel content increasing with depth	5.94
End of Borehole (GWL @ 1.05m - Jan. 17, 2023)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	4
SS	3	75	4
SS	4	67	7
SS	5	75	9
SS	6	83	7
SS	7	58	14
SS	8	25	14

DEPTH (m)	ELEV. (m)
0	92.77
1	91.77
2	90.77
3	89.77
4	88.77
5	87.77



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SOIL PROFILE AND TEST DATA

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Prop. Mixed-Use Development - 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

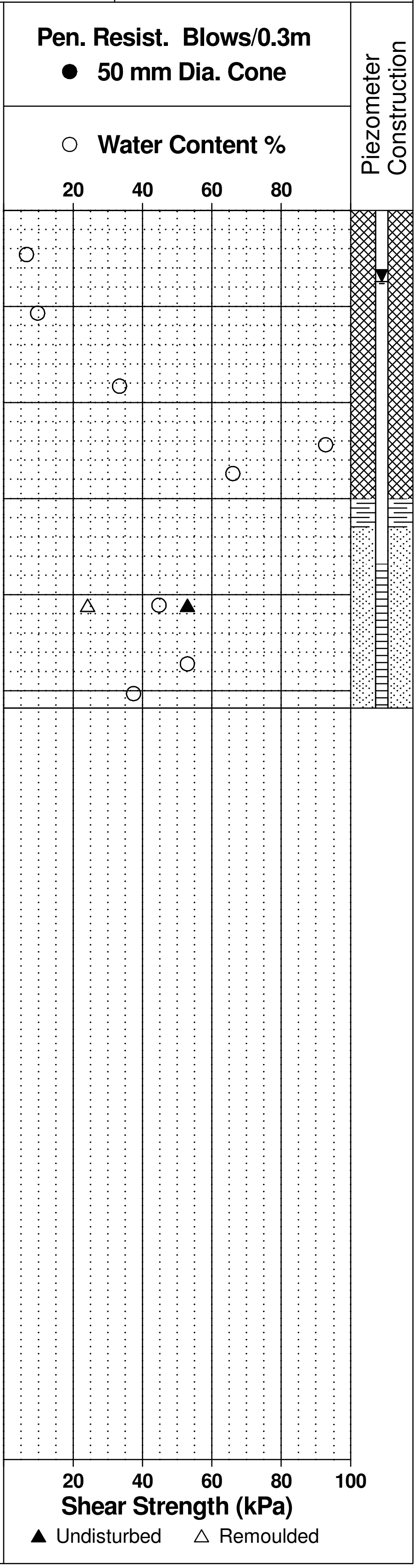
BORINGS BY Track-Mount Power Auger

DATE January 6, 2023

FILE NO. PG2103

HOLE NO. BH 4-23

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	91.89
FILL: Brown silty sand with gravel and crushed stone	AU	1				
	SS	2	67	23	1	90.89
1.45						
FILL: Brown silty clay with topsoil, some sand, trace wood	SS	3	17	6	2	89.89
2.59						
TOPSOIL	SS	4	67	3		
2.90						
Stiff, grey **SILTY CLAY,** trace sand	SS	5	0	3	3	88.89
	SS	6	100	1	4	87.89
4.82						
Grey **SILTY SAND**	SS	7	67	P	5	86.89
5.18						
End of Borehole						
(GWL @ 0.74m - Jan. 17, 2023)						



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SOIL PROFILE AND TEST DATA

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Prop. Mixed-Use Development - 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

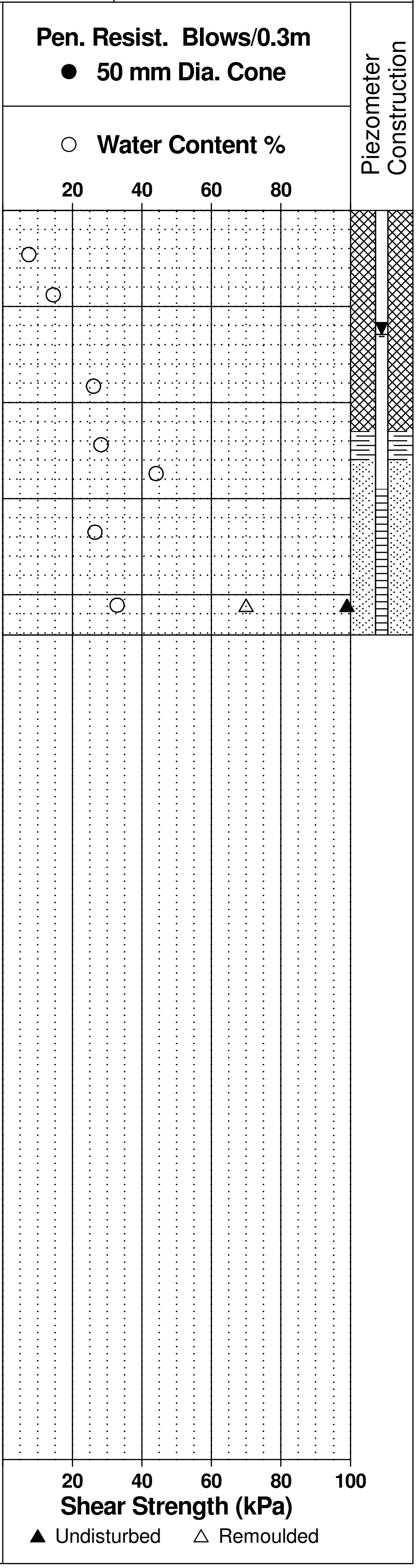
BORINGS BY Track-Mount Power Auger

DATE January 6, 2023

FILE NO. PG2103

HOLE NO. BH 5-23

SOIL DESCRIPTION	SAMPLE				DEPTH (m)	ELEV. (m)
	TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE					0	92.72
FILL: Grey gravel with crushed stone and sand 0.30	AU	1				
FILL: Brown silty sand with gravel, asphalt, some crushed stone - trace clay by 0.8m depth 1.45	SS	2	22	50+	1	91.72
FILL: Grey to brown silty clay, trace sand, gravel and organics 2.44	SS	3	83	7	2	90.72
TOPSOIL 2.97	SS	4	83	2		
Stiff, grey **SILTY CLAY,** trace sand	SS	5	92	1	3	89.72
4.42	SS	6	75	P	4	88.72
End of Borehole (GWL @ 1.29m - Jan. 17, 2023)						



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DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

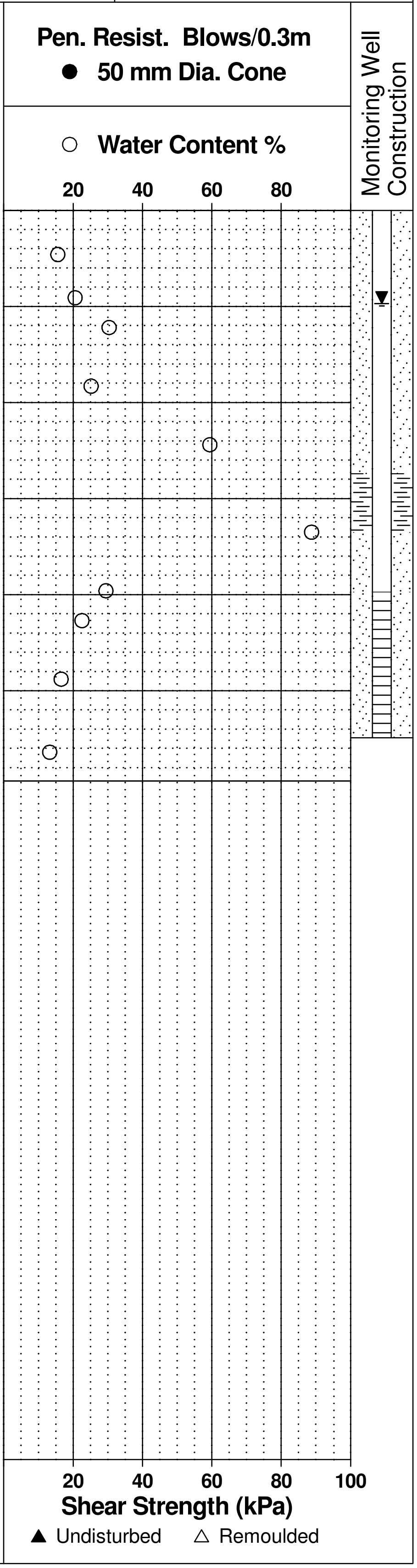
DATE January 6, 2023

FILE NO. PG2103

HOLE NO. BH 6-23

SOIL DESCRIPTION	Depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Grey gravel with sand and crushed stone	0.25				
FILL: Brown silty sand	1.17	AU	1		
		SS	2	75	10
FILL: Grey to brown silty clay, some sand, trace organics	2.49	SS	3	83	4
		SS	4	92	3
PEAT, trace clay	3.73	SS	5	100	1
Firm, grey **SILTY CLAY,** trace sand and gravel	3.96	SS	6	67	2
Very loose to dense, grey **SILTY SAND** - running sand encountered at 3.9m depth - gravel content increasing with depth	5.94	SS	7	8	6
		SS	8	25	48
End of Borehole (GWL @ 0.97m - Jan. 17, 2023)					

DEPTH (m)	ELEV. (m)
0	92.71
1	91.71
2	90.71
3	89.71
4	88.71
5	87.71



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Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE January 6, 2023

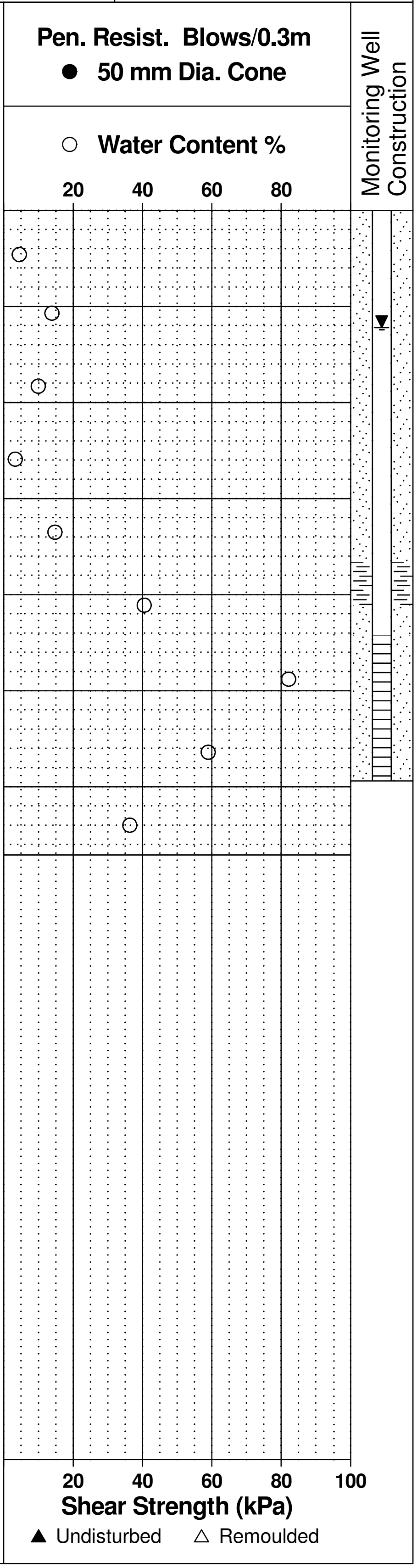
FILE NO. **PG2103**

HOLE NO. **BH 7-23**

SOIL DESCRIPTION	
GROUND SURFACE	
FILL: Crushed stone and gravel with silty sand	0.30
FILL: Brown silty sand, some gravel and crushed stone	0.69
FILL: Gravel with silty sand, trace clay, crushed stone and asphaltic concrete	1.45
FILL: Crushed stone and gravel, trace sand	3.66
FILL: Grey to brown silty clay, trace sand and organics	4.50
PEAT	5.94
Firm, grey organic **SILTY CLAY,** some sand, trace gravel	6.71
End of Borehole	
(GWL @ 1.22m - Jan. 17, 2023)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	33	14
SS	3	25	11
SS	4	12	7
SS	5	8	7
SS	6	33	4
SS	7	33	3
SS	8	83	2
SS	9	67	2

DEPTH (m)	ELEV. (m)
0	92.47
1	91.47
2	90.47
3	89.47
4	88.47
5	87.47
6	86.47



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE January 9, 2023

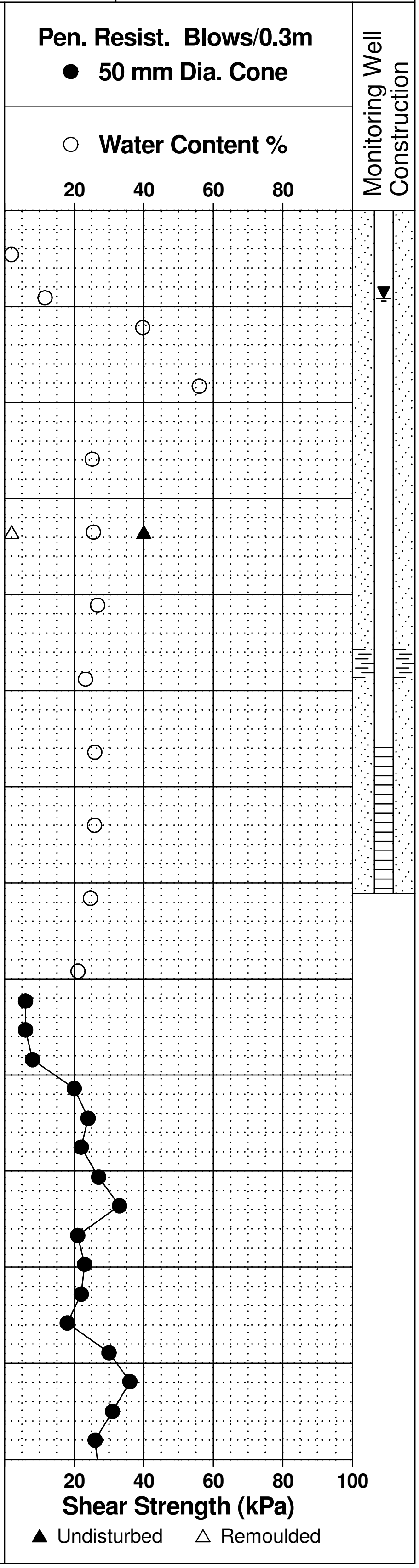
FILE NO. **PG2103**

HOLE NO. **BH 8-23**

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Crushed stone and gravel with silty sand	0.30
FILL: Brown silty sand with gravel and crushed stone	1.07
TOPSOIL	1.45
Firm, brown **SILTY CLAY,** some to trace organics - grey by 2.2m depth	3.73
Loose to very loose, grey **SILTY SAND** - some running sand from 3.8m to 8.2m depth	8.23
Dynamic Cone Penetration Test commenced at 8.23m depth.	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	7
SS	3	67	4
SS	4	100	4
SS	5	83	P
SS	6	12	P
SS	7	67	9
SS	8	33	1
SS	9	33	1
SS	10	33	1
SS	11	58	3

DEPTH (m)	ELEV. (m)
0	92.58
1	91.58
2	90.58
3	89.58
4	88.58
5	87.58
6	86.58
7	85.58
8	84.58
9	83.58
10	82.58
11	81.58
12	80.58
13	79.58

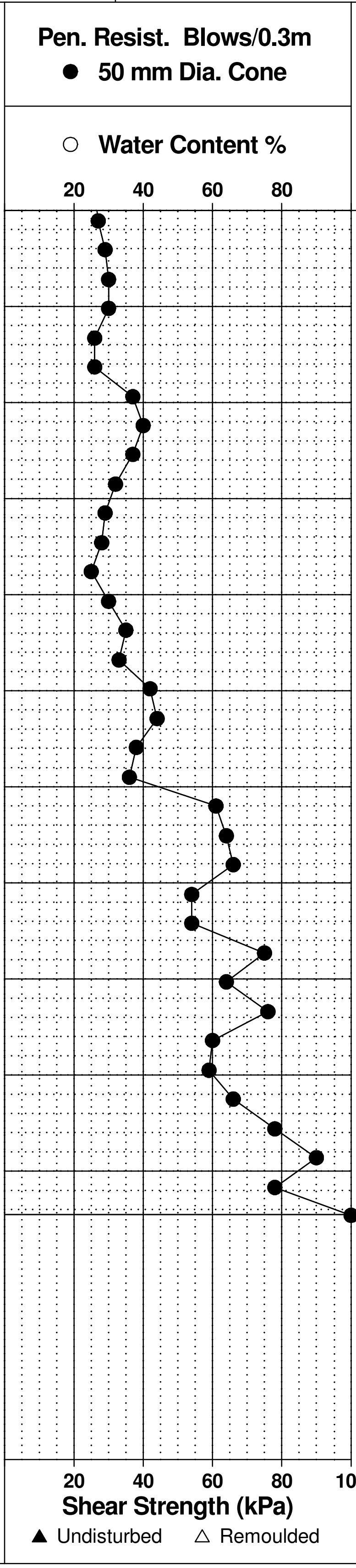


SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE January 9, 2023
FILE NO. **PG2103**
HOLE NO. **BH 8-23**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
GROUND SURFACE								
Dynamic Cone Penetration Test commenced at 8.23m depth.						13	79.58	
						14	78.58	
						15	77.58	
						16	76.58	
						17	75.58	
						18	74.58	
						19	73.58	
						20	72.58	
						21	71.58	
						22	70.58	
						23	69.58	
23.45								
End of Borehole Practical DCPT refusal at 23.45m depth. (GWL @ 0.92m - Jan. 17, 2023)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

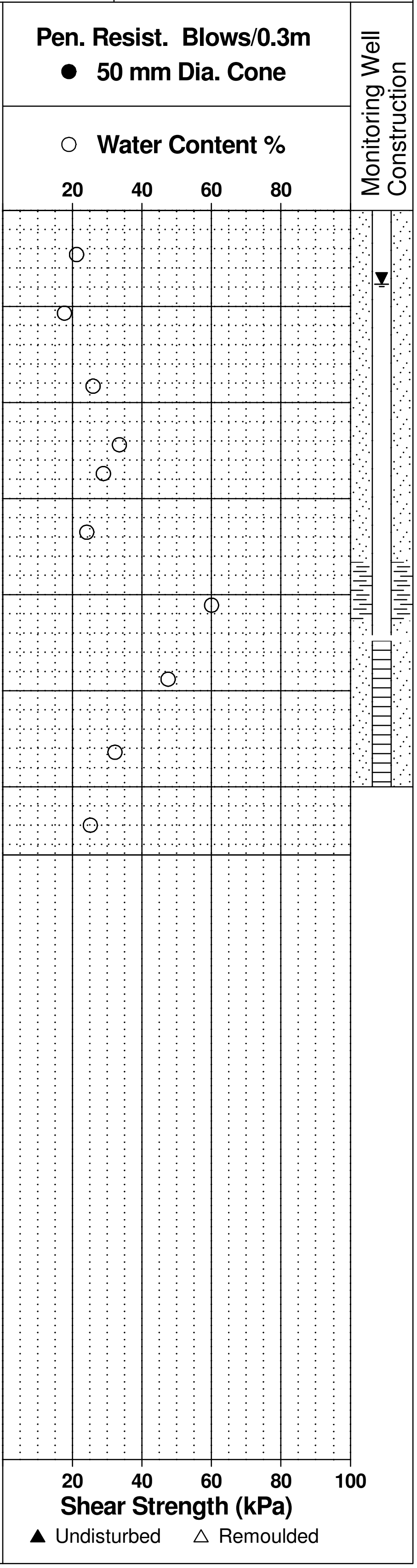
DATE January 10, 2023

FILE NO. PG2103

HOLE NO. BH 9-23

SOIL DESCRIPTION	Depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Crushed stone and gravel with silty sand	0.05 0.46	AU	1		
TOPSOIL					
FILL: Crushed stone with clay and sand, some brick	1.45	SS	2	8	3
FILL: Grey to brown silty clay, some sand	3.35	SS SS	3 4	67 92	4 4
FILL: Grey silty sand	3.73	SS	5	4	1
Firm, grey organic **SILTY CLAY,** trace sand and wood	5.26	SS SS	6 7	100 100	1 1
Very loose, grey **SILTY SAND** with clay - trace gravel by 6.0m depth	6.71	SS SS	8 9	50 42	1 1
End of Borehole (GWL @ 0.77m - Jan. 17, 2023)					

DEPTH (m)	ELEV. (m)
0	92.41
1	91.41
2	90.41
3	89.41
4	88.41
5	87.41
6	86.41



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Development - 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE January 10, 2023

FILE NO. PG2103

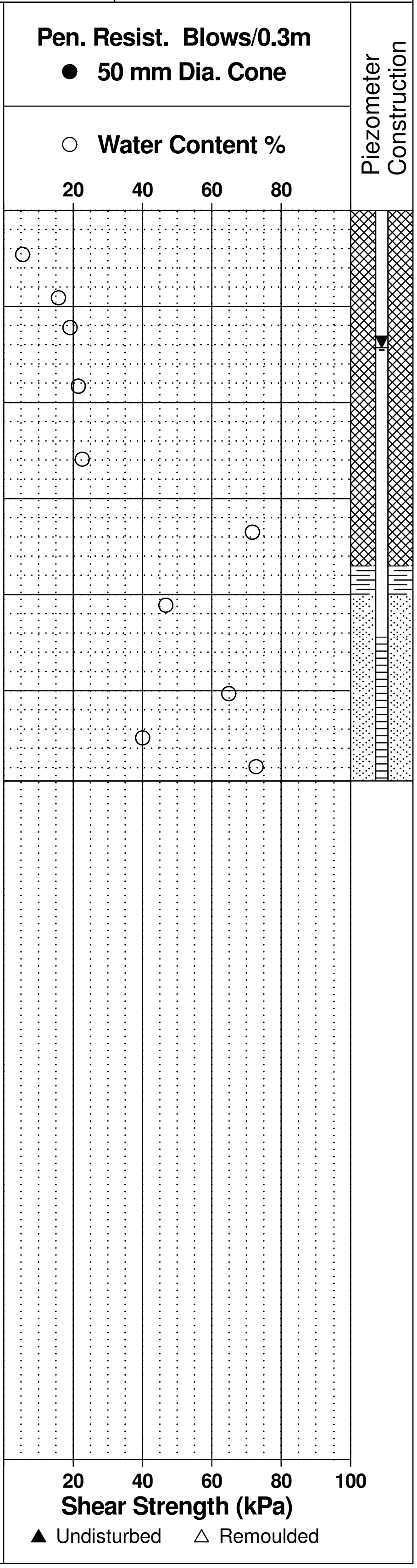
HOLE NO. BH10-23

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Crushed stone and gravel with silty sand	0.15	AU	1		
FILL: Brown silty sand with gravel and crushed stone	1.22	SS	1	38	11
FILL: Brown to grey silty clay, some sand, some to trace topsoil and organics	2.21	SS	3	58	4
FILL: Grey silty sand, some clay	2.97	SS	4	17	P
PEAT, some clay and sand, trace wood		SS	5	100	2
		SS	6	92	2
	5.03	SS	7	67	2
Stiff to firm, grey **SILTY CLAY to CLAYEY SILT**	5.54				
PEAT with clay, some sand	5.94	SS	8	100	P

End of Borehole

(GWL @ 1.43m - Jan. 17, 2023)

DEPTH (m)	ELEV. (m)
0	92.46
1	91.46
2	90.46
3	89.46
4	88.46
5	87.46



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development, 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 45 Power Auger

DATE October 18, 2010

FILE NO. PG2103

HOLE NO. BH 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	92.18		
FILL: Brown silty sand with gravel 0.60		AU	1						
FILL: Grey-brown silty clay		SS	2	25	3	1	91.18		
2.20		SS	3	42	5	2	90.18		∇
TOPSOIL 2.50									
Stiff, brown **SILTY CLAY** 2.90		SS	4		2				
End of Borehole									
(Open hole GWL @ 2.1m depth)									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development, 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 45 Power Auger

DATE October 18, 2010

FILE NO. PG2103

HOLE NO. BH 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	90.81		
FILL: Brown silty sand with gravel, some clay	⊠	AU	1						
		SS	2	25	10	1	89.81		
1.45									
Grey-brown **SILTY CLAY**	▨	SS	3	50	5	2	88.81		
2.13									
End of Borehole (BH dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development, 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 45 Power Auger

DATE October 18, 2010

FILE NO. PG2103

HOLE NO. BH 3

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	92.18		
FILL: Brown to black silty sand, some gravel		AU	1						
0.60		AU	2						
FILL: Black silty sand with clay and gravel		SS	2	42	22	1	91.18		
1.45									
Grey-brown **SILTY CLAY**		SS	3	50	6	2	90.18		
2.90		SS	4	50	3				
End of Borehole									
(BH dry upon completion)									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development, 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

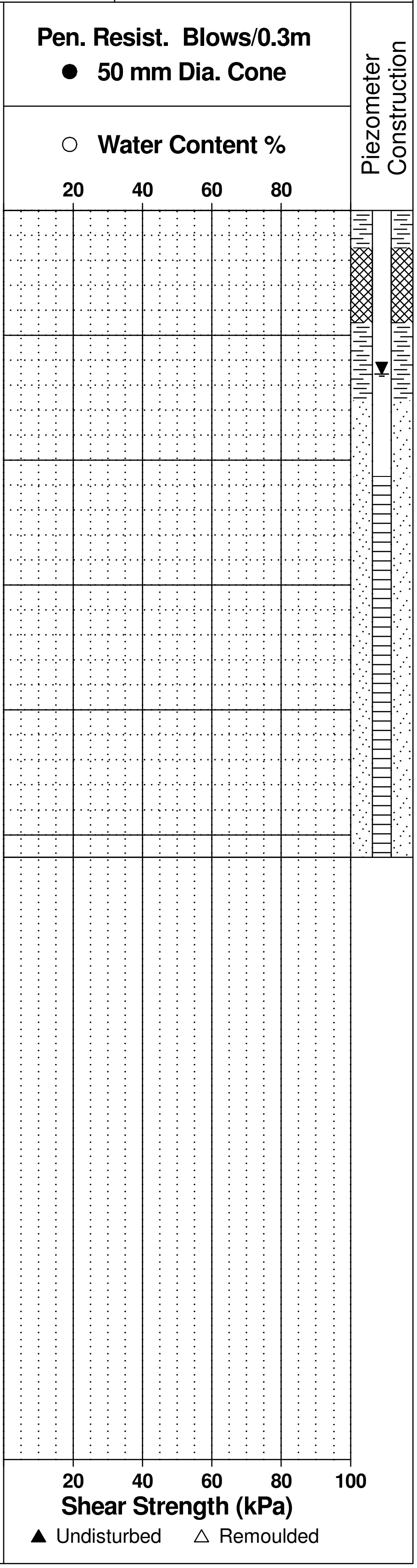
BORINGS BY CME 45 Power Auger

DATE October 18, 2010

FILE NO. PG2103

HOLE NO. BH 4

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.52
Asphaltic concrete	0.04	AU	1				
FILL: Brown silty clay, some sand and gravel	1.45	SS	2	67	3	1	91.52
FILL: Grey-brown silty clay, some sand, trace gravel		SS	3	71	2	2	90.52
	3.30	SS	4	33	3	3	89.52
GLACIAL TILL: Compact to loose, grey silty sand with gravel		SS	5	8	12		
		SS	6	33	5	4	88.52
	5.18	SS	7	33	4	5	87.52
End of Borehole							
(GWL @ 1.31m-Oct. 25/10)							



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SOIL PROFILE AND TEST DATA

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Proposed Commercial Development, 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

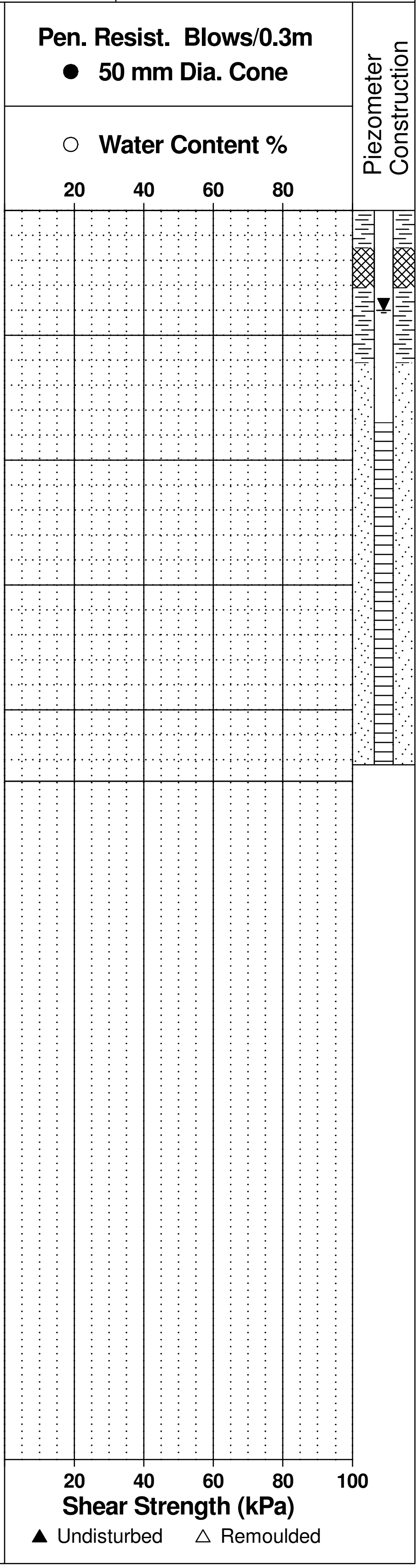
BORINGS BY CME 45 Power Auger

DATE October 18, 2010

FILE NO. PG2103

HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	92.83
FILL: Brown silty sand with gravel	AU	1				
0.60						
FILL: Grey-brown silty clay, trace sand and organics	SS	2	67	7	1	91.83
	SS	3	62	4	2	90.83
2.21						
Dense to compact, grey **SILTY SAND** with gravel and cobbles	SS	4	42	7		
					3	89.83
	SS	5	50	40		
	SS	6	50	10	4	88.83
4.57						
End of Borehole						
(GWL @ 0.80m-Oct. 25/10)						



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REMARKS

BORINGS BY CME 45 Power Auger

DATE October 18, 2010

FILE NO. PG2103

HOLE NO. BH 6

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	92.67		
Concrete 0.18		AU	1						
FILL: Brown silty sand with gravel		SS	2	70		1	91.67		
1.68									
Grey-brown **SILTY CLAY,** trace sand and organics		SS	3	46	11	2	90.67		
- grey by 2.2m depth		SS	4	71	3	3	89.67		
		SS	5	75	3				
		SS	6	67	2	4	88.67		▽
		SS	7		2	5	87.67		
5.18									
End of Borehole									
(Open hole GWL @ 4.3m depth)									

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

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SOIL PROFILE AND TEST DATA

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Proposed Commercial Development, 3725 Carp Road
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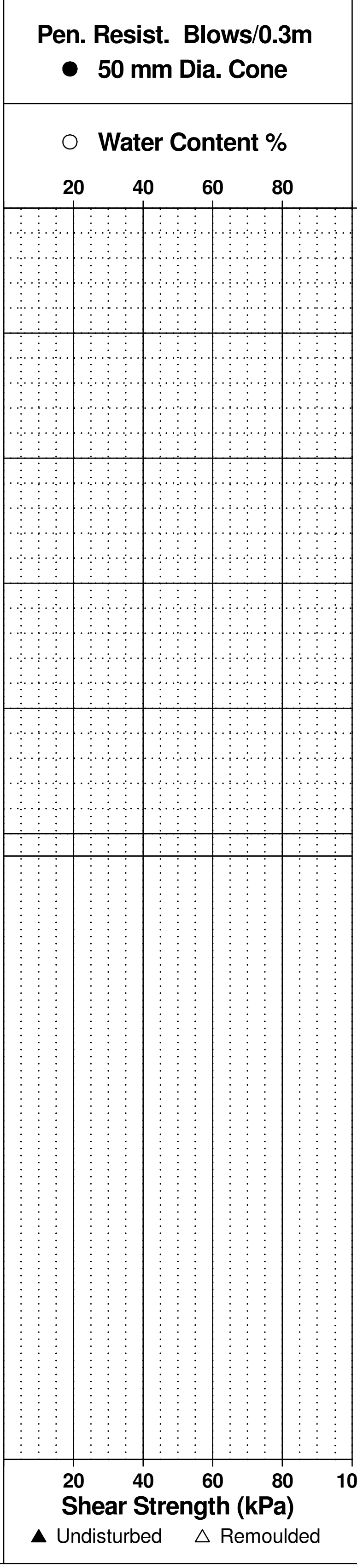
REMARKS
BORINGS BY CME 45 Power Auger
DATE October 19, 2010
FILE NO. PG2103
HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	92.58	
Asphaltic concrete 0.05		AU	1					
FILL: Brown silty sand with gravel		SS	2	50	38	1	91.58	
		SS	3	38	14			
2.21						2	90.58	
FILL: Grey silty clay with sand and gravel 2.49		SS	4	67	4			
FILL: Grey-brown silty clay, trace sand and organics						3	89.58	
		SS	5	75	2			
3.95		SS	6	92	2	4	88.58	
PEAT with clay		SS	7	92	1			
5.18						5	87.58	
End of Borehole (BH dry upon completion)								



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REMARKS

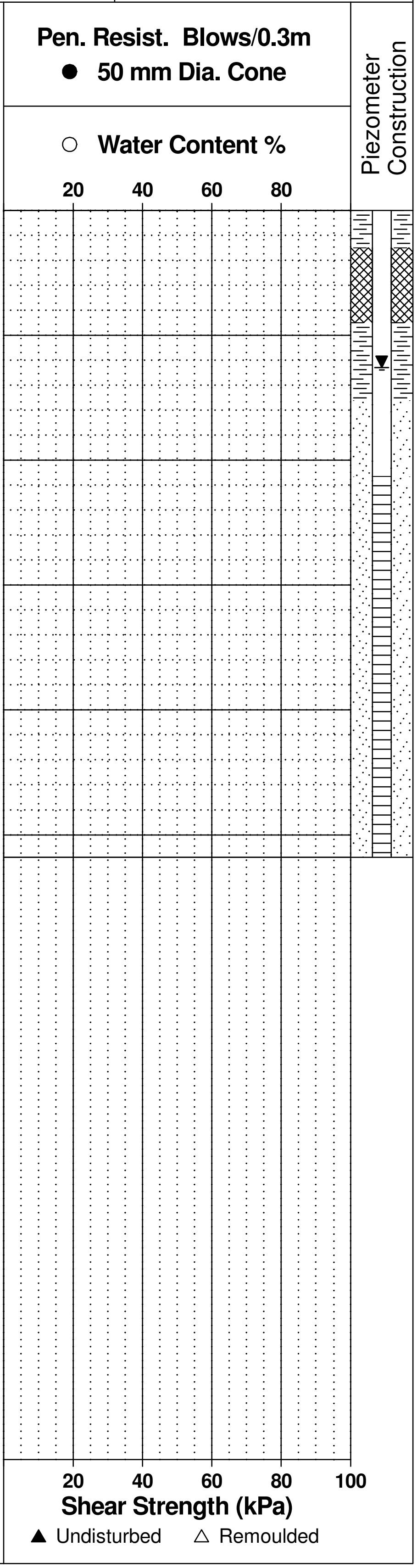
BORINGS BY CME 45 Power Auger

DATE October 19, 2010

FILE NO. PG2103

HOLE NO. BH 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.53
Concrete 0.06		AU	1				
FILL: Brown silty sand, trace clay and gravel 1.68		SS	2	42	17	1	91.53
FILL: Grey to black silty sand 2.21		SS	3	54	1	2	90.53
FILL: Grey-brown silty clay, trace gravel and organics		SS	4	8	4	3	89.53
3.73		SS	5	75	4		
PEAT with clay		SS	6	0	2	4	88.53
5.18		SS	7	75	1	5	87.53
End of Borehole							
(GWL @ 1.26m-Oct. 25/10)							



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REMARKS

BORINGS BY CME 45 Power Auger

DATE October 19, 2010

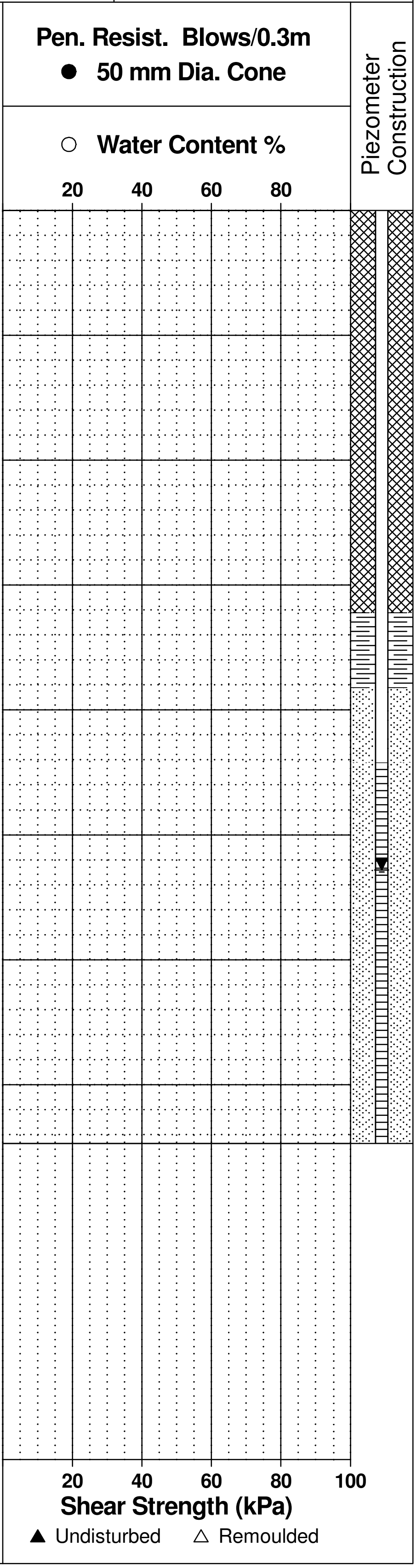
FILE NO. PG2103

HOLE NO. BH 9

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand with gravel	0.60
FILL: Brown silty sand	1.07
FILL: Grey silty clay with sand, some gravel	2.29
FILL: Grey silty clay, trace organics	3.89
PEAT	5.35
Grey-brown **SILTY CLAY,** trace gravel	5.79
Very loose, grey **SAND** with gravel	7.47
End of Borehole (GWL @ 5.28m-Oct. 25/10)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	17
SS	3	25	4
SS	4	67	5
SS	5	75	1
SS	6	92	1
SS	7	83	2
SS	8	25	2
SS	9	83	1

DEPTH (m)	ELEV. (m)
0	92.67
1	91.67
2	90.67
3	89.67
4	88.67
5	87.67
6	86.67
7	85.67



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development, 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 45 Power Auger

DATE October 19, 2010

FILE NO. PG2103

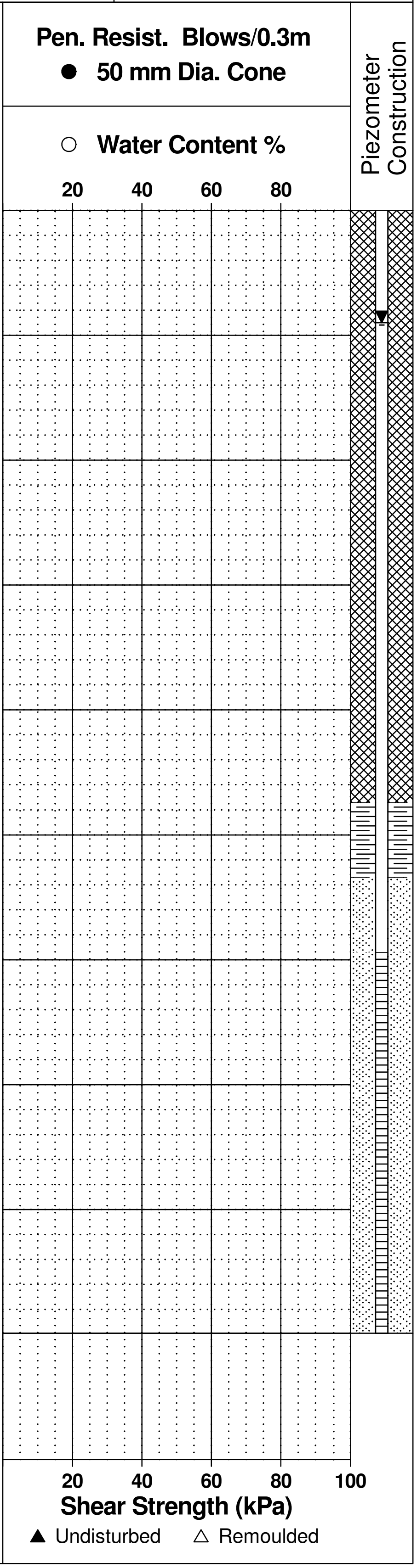
HOLE NO. BH10

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with organics, trace gravel — 0.60	AU	1		
FILL: Brown silty clay with sand, gravel, cobbles, trace organics and brick — 1.45	SS	2	67	5
PEAT — 2.13	SS	3	79	4
Loose, grey-brown **SILTY SAND** — 6.01	SS	4	50	2
	SS	5	79	8
GLACIAL TILL: Compact, brown silty sand with gravel and cobbles — 8.99	SS	6	38	14
	SS	7	25	24
	SS	8	17	11
	SS	9	17	16

DEPTH (m)	ELEV. (m)
0	92.95
1	91.95
2	90.95
3	89.95
4	88.95
5	87.95
6	86.95
7	85.95
8	84.95

End of Borehole
Running sand encountered @ 3.5m depth

(GWL @ 0.90m-Oct. 25/10)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Commercial Development, 3725 Carp Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 45 Power Auger

DATE October 19, 2010

FILE NO. PG2103

HOLE NO. BH11

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	93.85		
FILL: Brown silty sand with gravel, cobbles and boulders 0.60	▨	AU	1						
Grey-brown **SILTY CLAY,** trace organics	▨	SS	2	21	9	1	92.85		
	▨	SS	3	67	2	2	91.85		
2.74	▨	SS	4	62	4				
End of Borehole (BH dry upon completion)\									

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

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

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

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

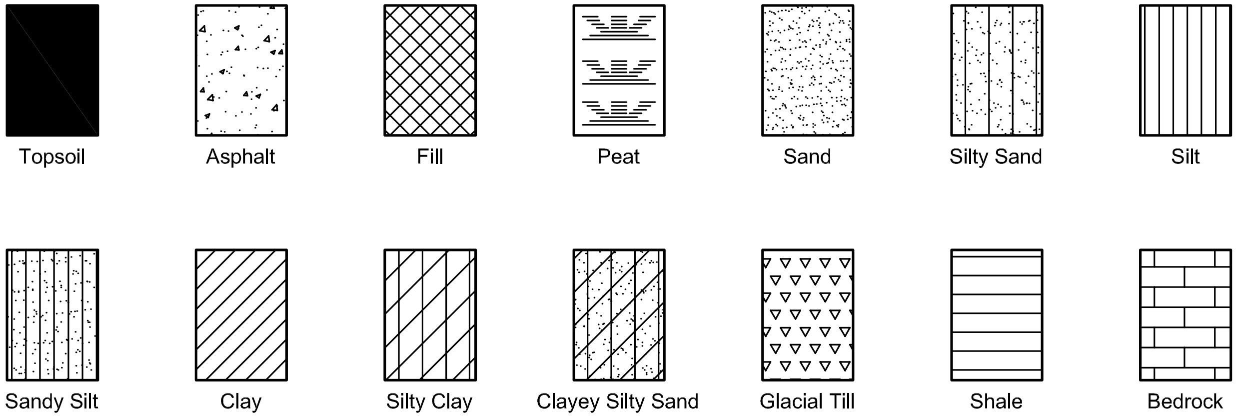
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

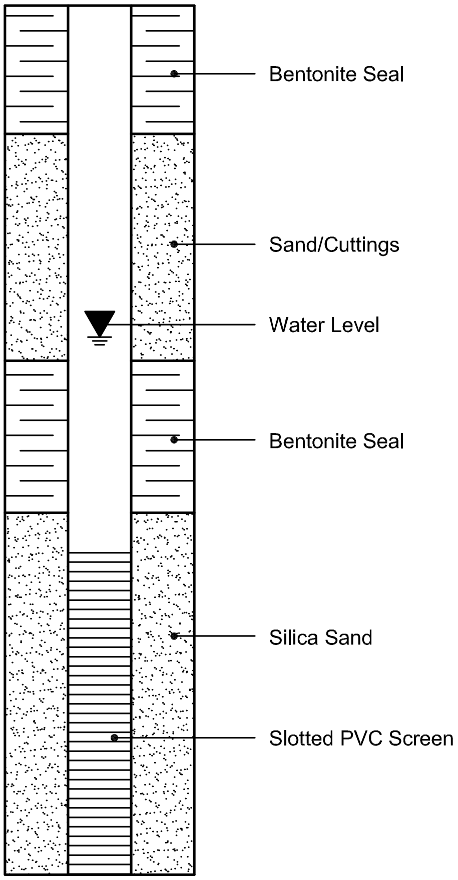
SYMBOLS AND TERMS (continued)

STRATA PLOT

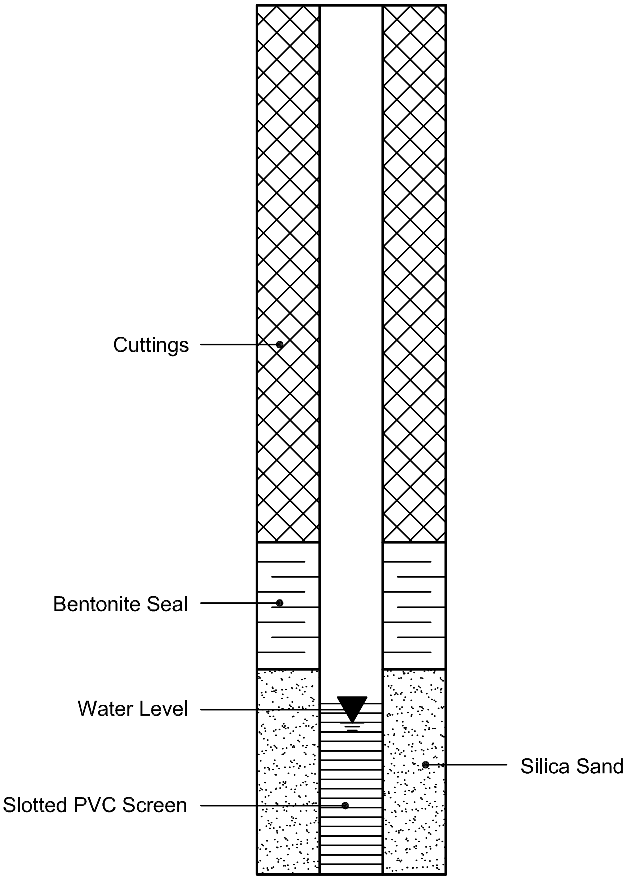


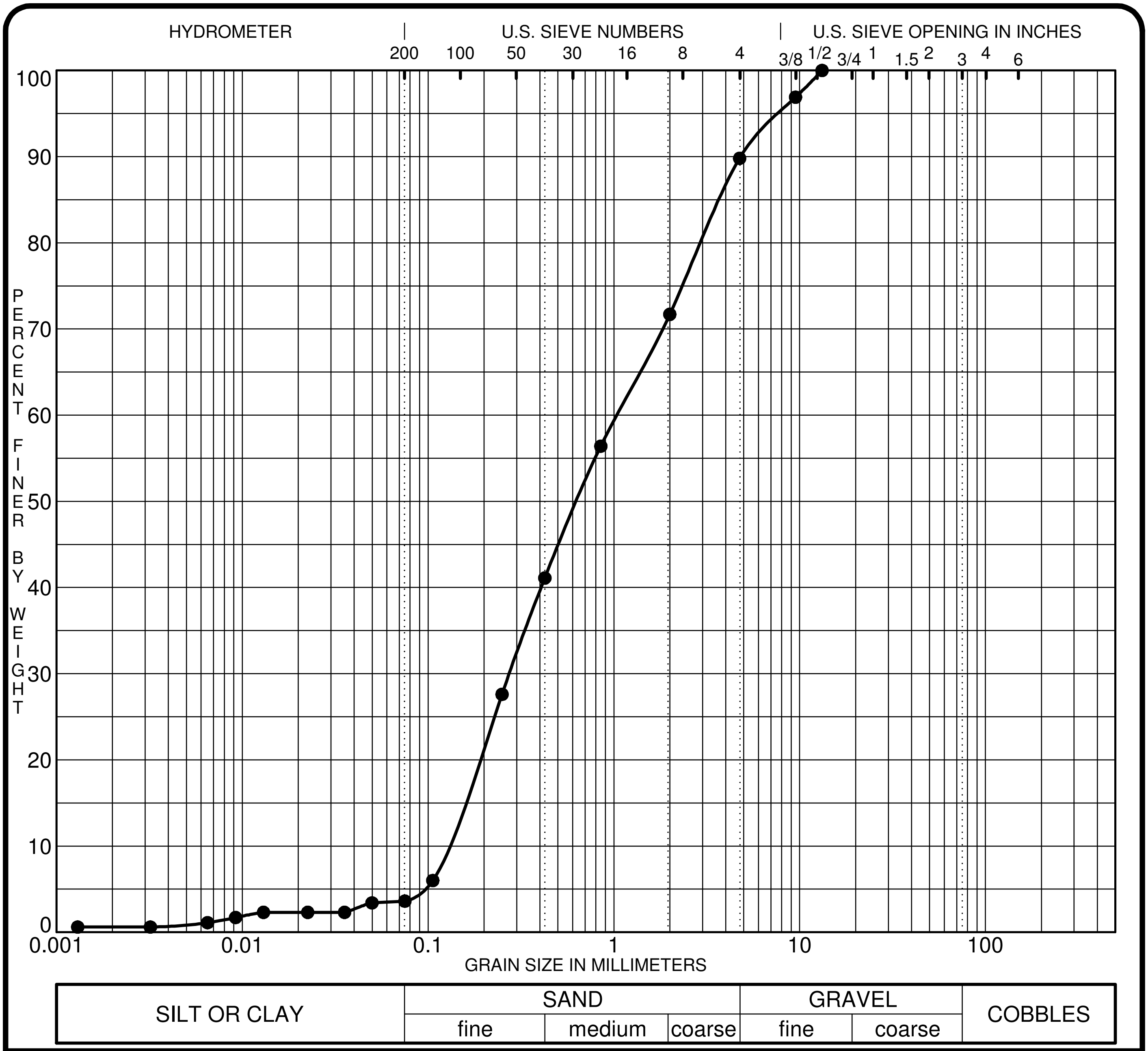
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 3-23**	**SS7**						**0.58**	**8.4**
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 3-23**	**SS7**	**13.20**	**1.04**	**0.275**	**0.1243**	**10.2**	**86.2**	**3.6**	
⊠										
▲										
★										

CLIENT **Karson Kartage Konstruction Ltd.**

FILE NO. **PG2103**

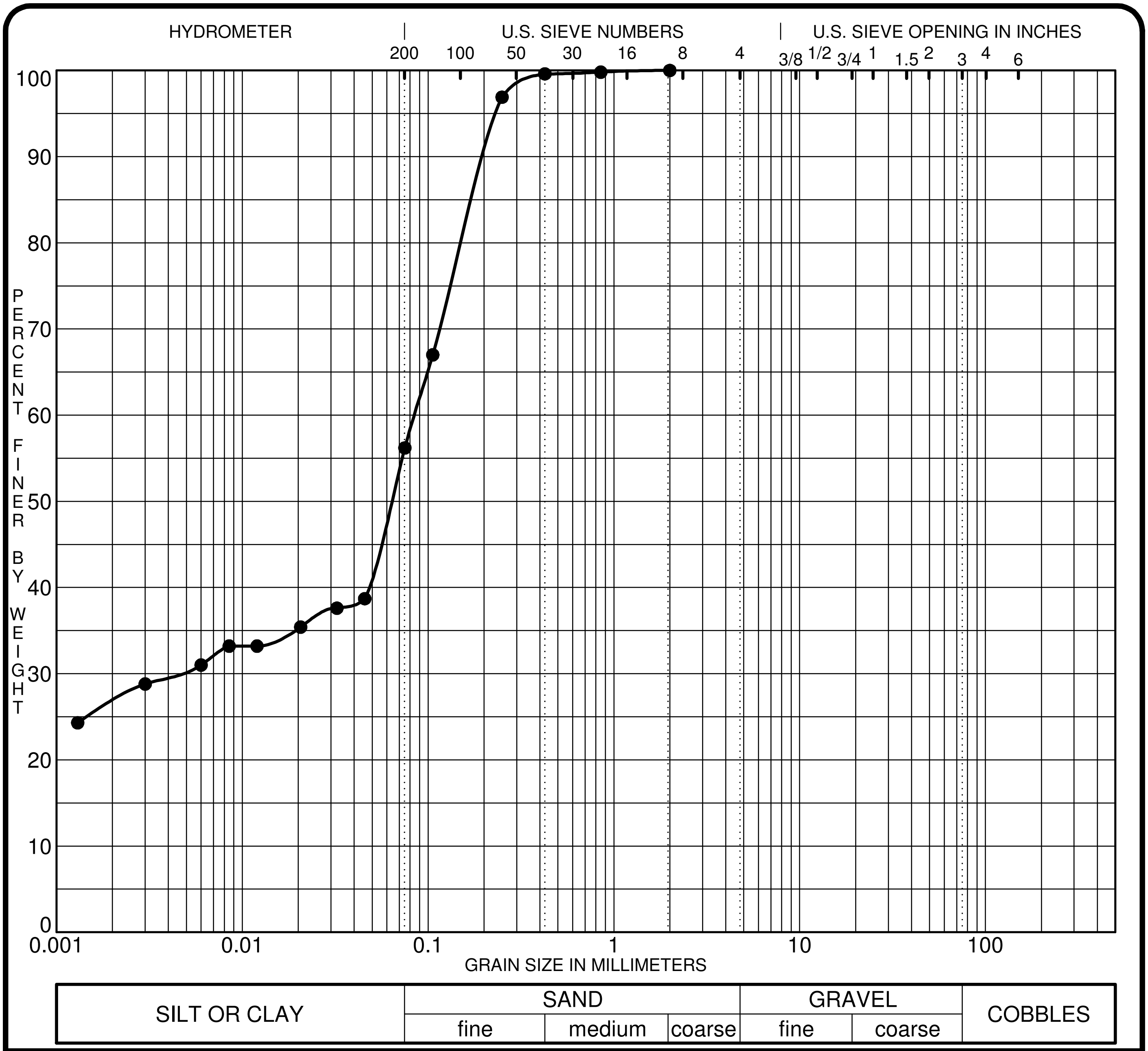
PROJECT **Geotechnical Investigation - Prop. Mixed-Use Development - 3725 Carp Road**

DATE **6 Jan 23**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 9-23 SS8**							
⊠								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 9-23 SS8**	**2.00**	**0.08**	**0.004**		**0.0**	**43.8**	**56.2**	
⊠									
▲									
★									

CLIENT **Karson Kartage Konstruction Ltd.**

FILE NO. **PG2103**

PROJECT **Geotechnical Investigation - Prop. Mixed-Use Development - 3725 Carp Road**

DATE **10 Jan 23**

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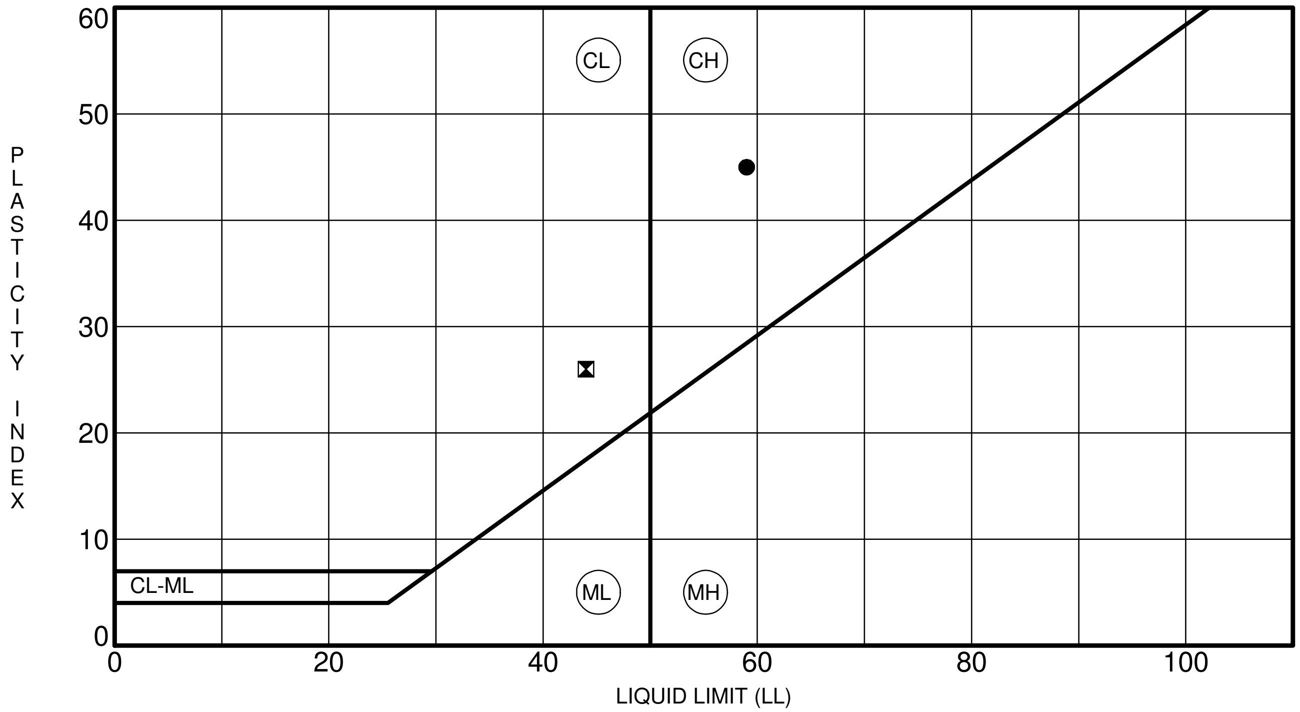
GRAIN SIZE DISTRIBUTION



	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 4-23 SS6**	**59**	**14**	**45**		**CH - Inorganic clay of high plasticity**
☒	**BH 5-23 SS5**	**44**	**18**	**26**		**CL - Inorganic clay of low plasticity**

CLIENT **Karson Kartage Konstruction Ltd.**

FILE NO. **PG2103**

PROJECT **Geotechnical Investigation - Prop. Mixed-Use Development - 3725 Carp Road**

DATE **6 Jan 23**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

ATTERBERG LIMITS' RESULTS

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 56571

Report Date: 17-Jan-2023

Order Date: 12-Jan-2023

Project Description: PG2103

	Client ID:	BH3-23 SS3	-	-	-
	Sample Date:	06-Jan-23 09:00	-	-	-
	Sample ID:	2302478-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	80.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.00	-	-	-
Resistivity	0.10 Ohm.m	98.4	-	-	-
Anions					
Chloride	10 ug/g dry	<10	-	-	-
Sulphate	10 ug/g dry	22	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 TO 7 – AERIAL PHOTOGRAPHS

FIGURES 8 TO 15 – SLOPE STABILITY CROSS SECTIONS

PHOTOGRAPHS FROM SITE VISIT

DRAWING PG2103-1 – TEST HOLE LOCATION PLAN

DRAWING PG2103-2 – PERMISSIBLE GRADE RAISE PLAN

DRAWING PG2103-3 – TREE PLANTING SETBACK RESTRICTIONS

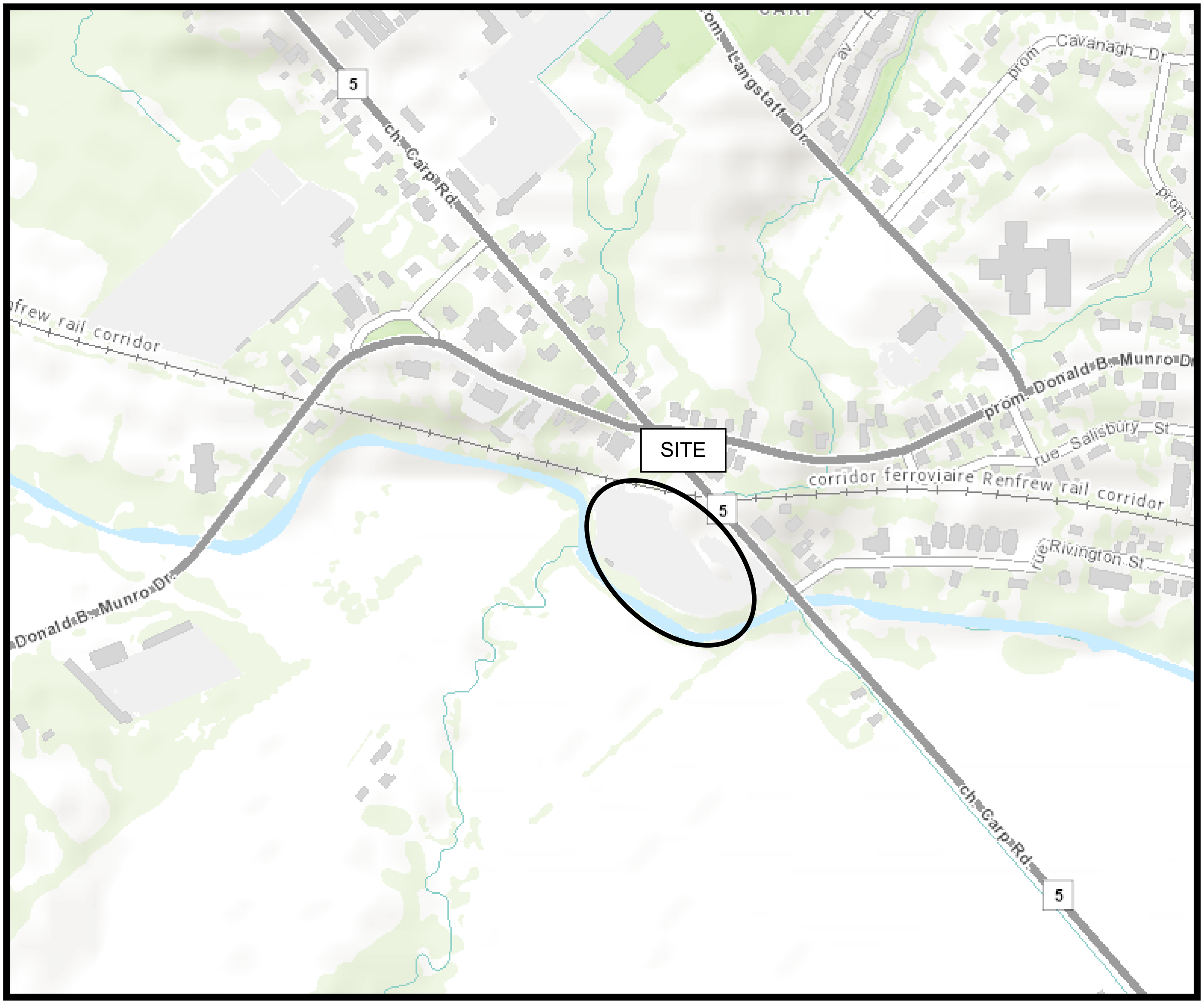


FIGURE 1

KEY PLAN

FIGURE 2

Aerial Photograph – 1976

FIGURE 3

Aerial Photograph – 1991

FIGURE 4

Aerial Photograph – 2011

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FIGURE 5

Aerial Photograph – 2014

PATERSON GROUP

FIGURE 6

Aerial Photograph – 2015

PATERSON GROUP

FIGURE 7

Aerial Photograph – 2021

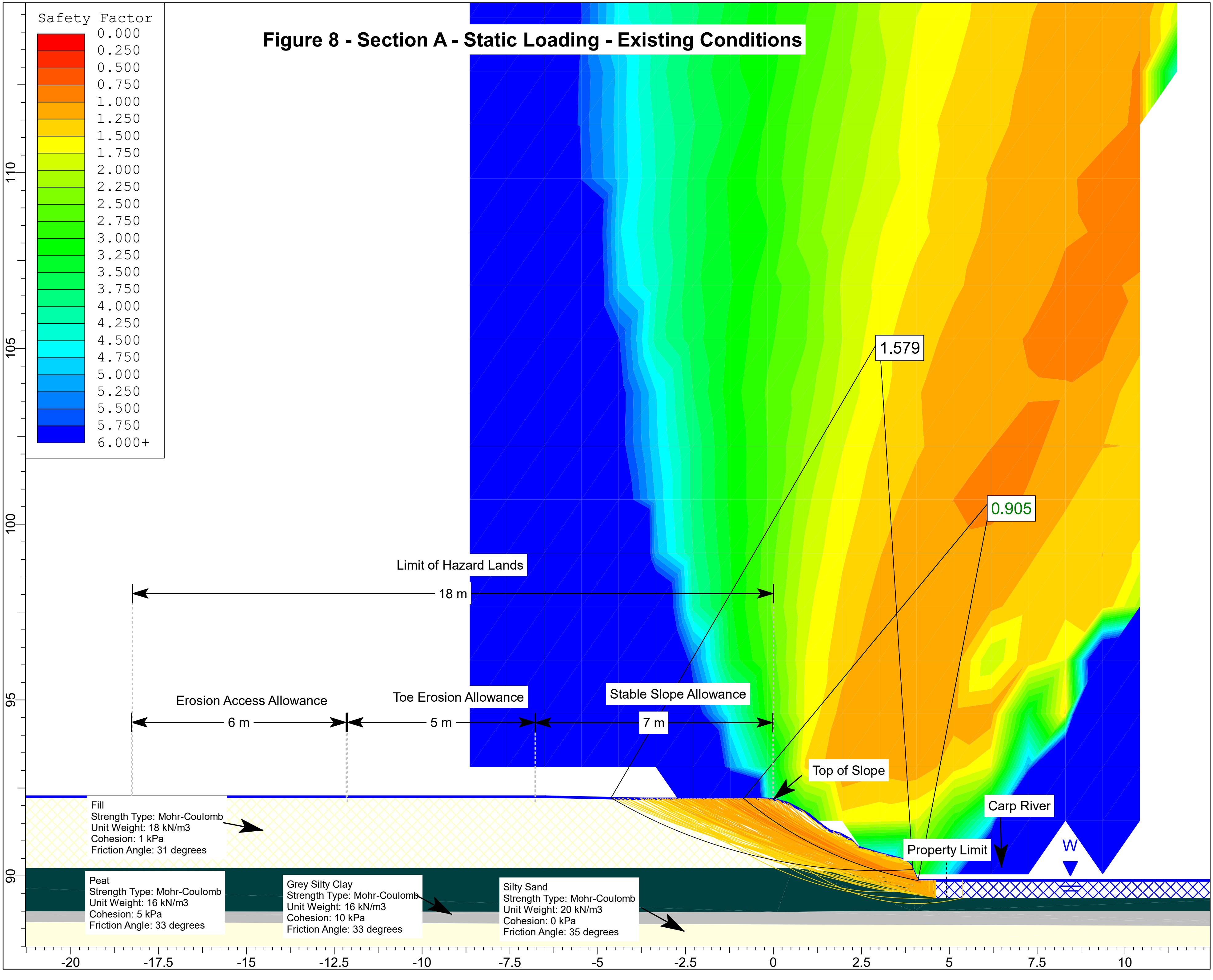


Figure 8 - Section A - Static Loading - Existing Conditions

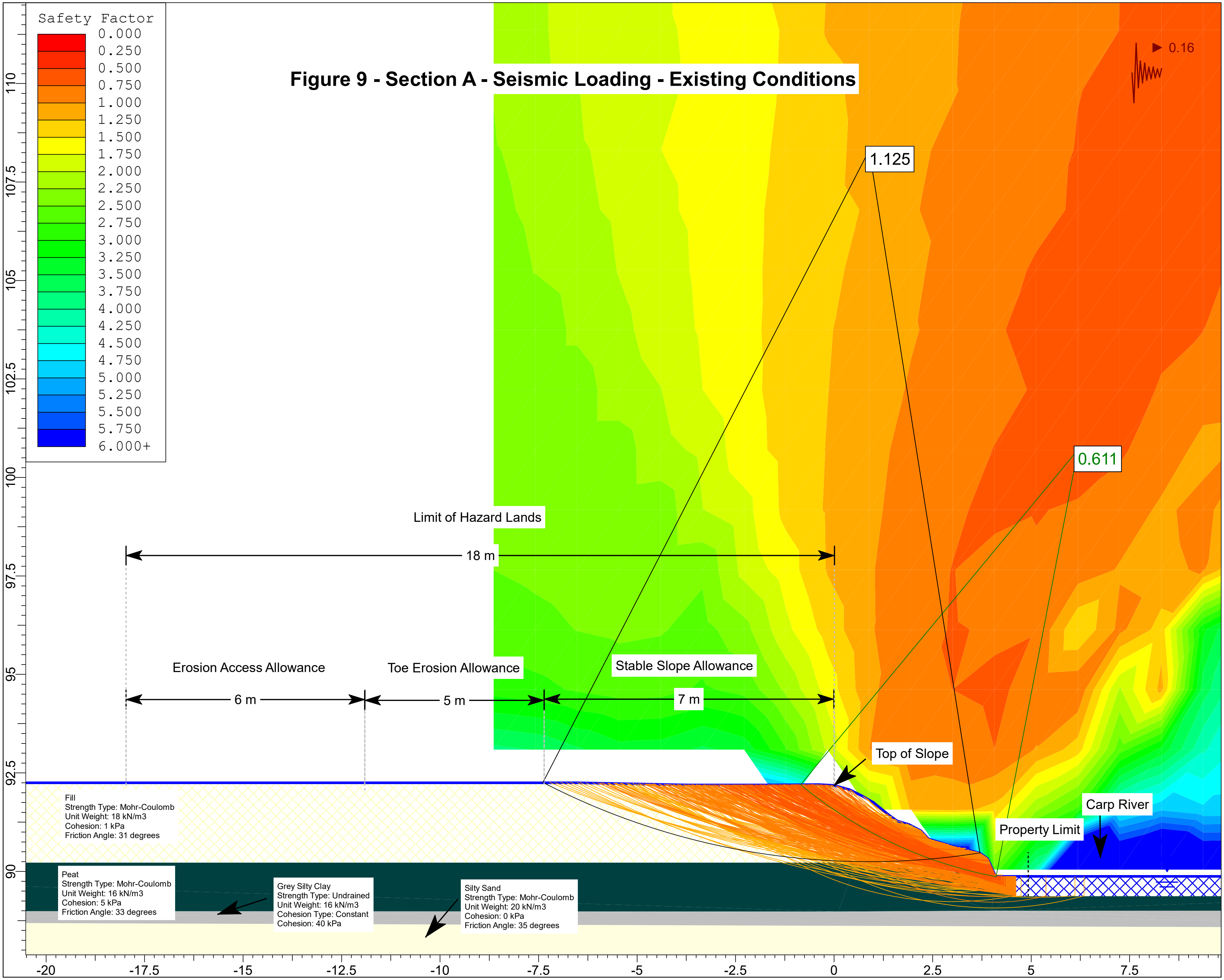


Figure 9 - Section A - Seismic Loading - Existing Conditions

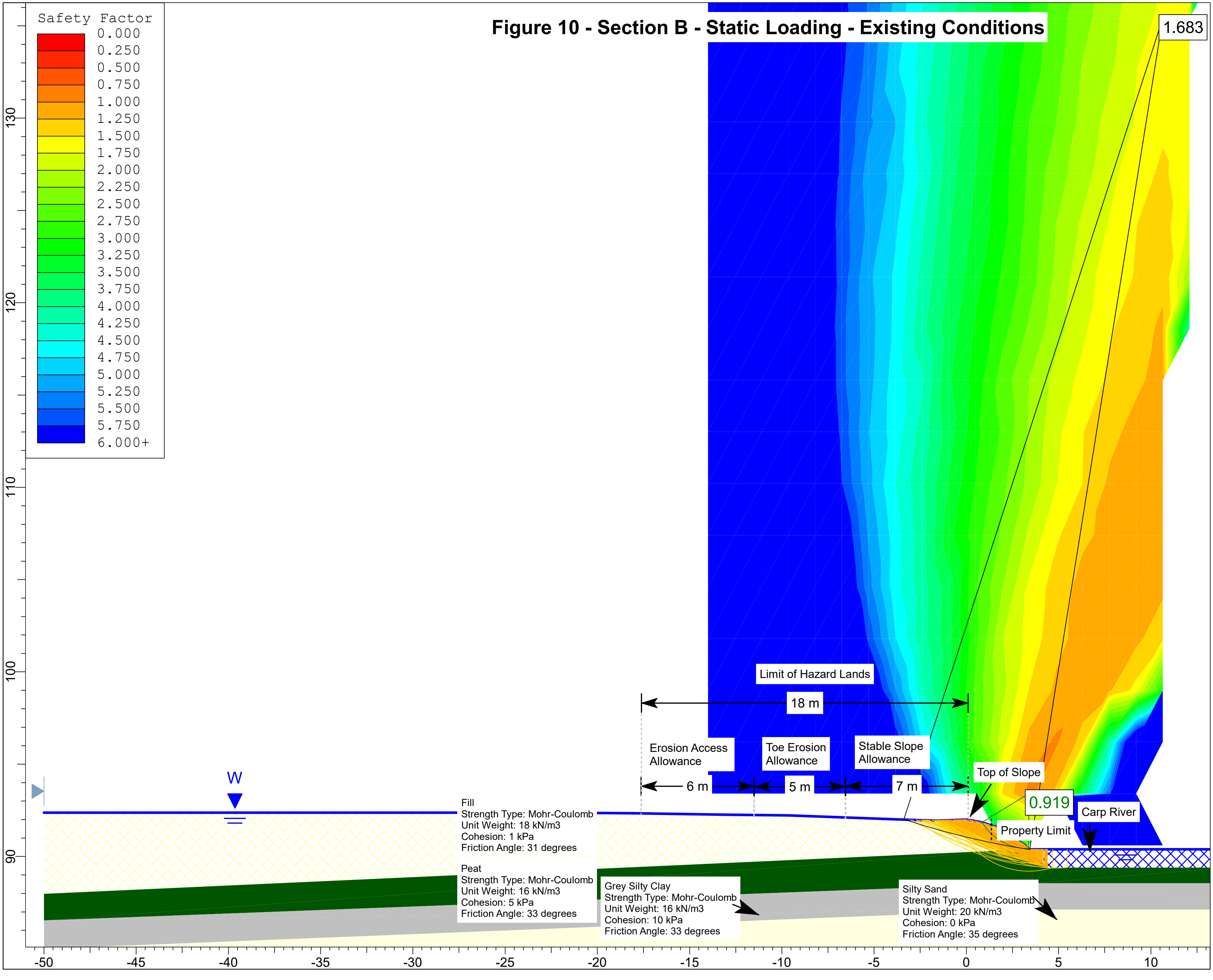


Figure 10 - Section B - Static Loading - Existing Conditions

Fill
Strength Type: Mohr-Coulomb
Unit Weight: 18 kN/m3
Cohesion: 1 kPa
Friction Angle: 31 degrees

Peat
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees

Grey Silty Clay
Strength Type: Mohr-Coulomb
Unit Weight: 16 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees

Silty Sand
Strength Type: Mohr-Coulomb
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 degrees

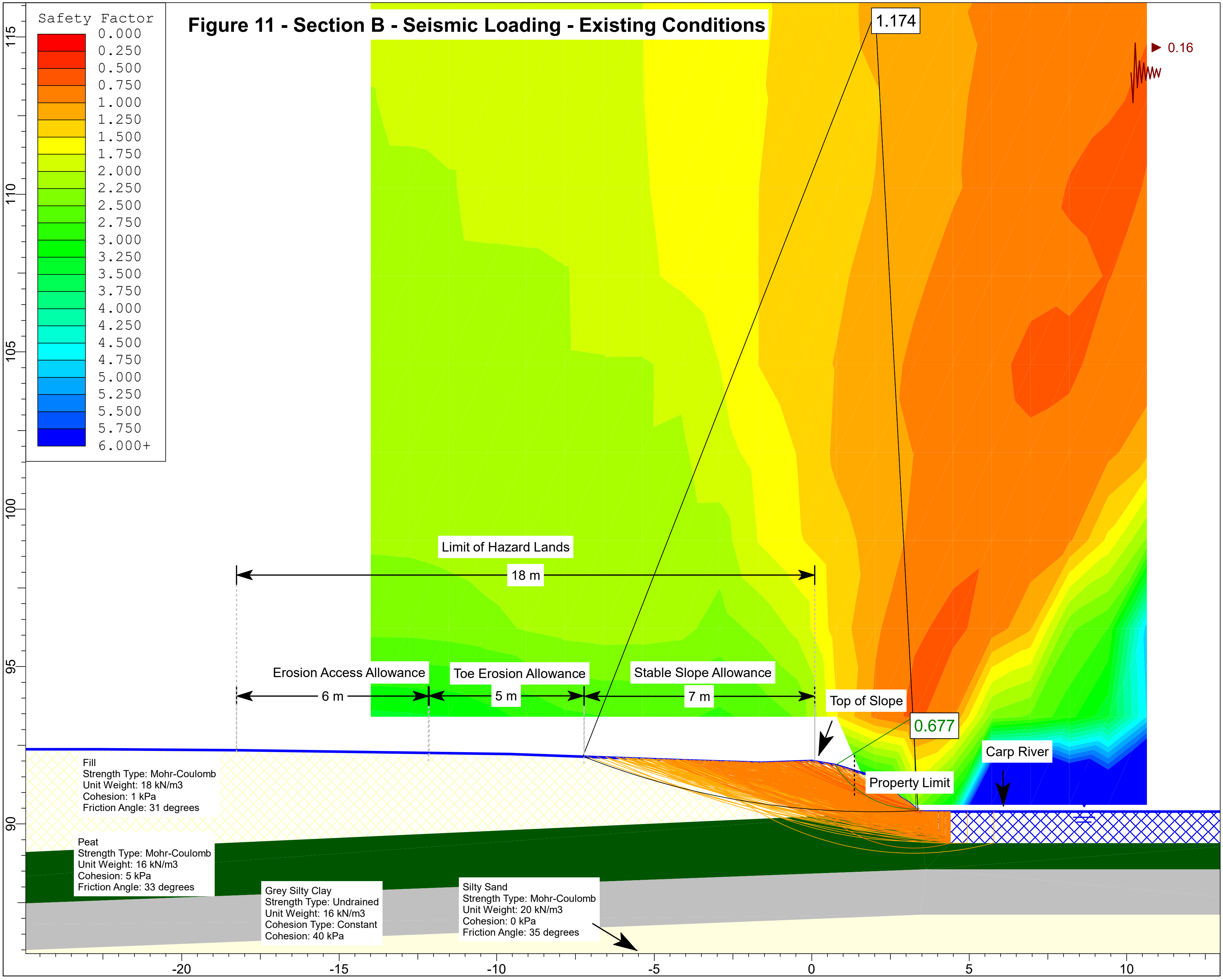
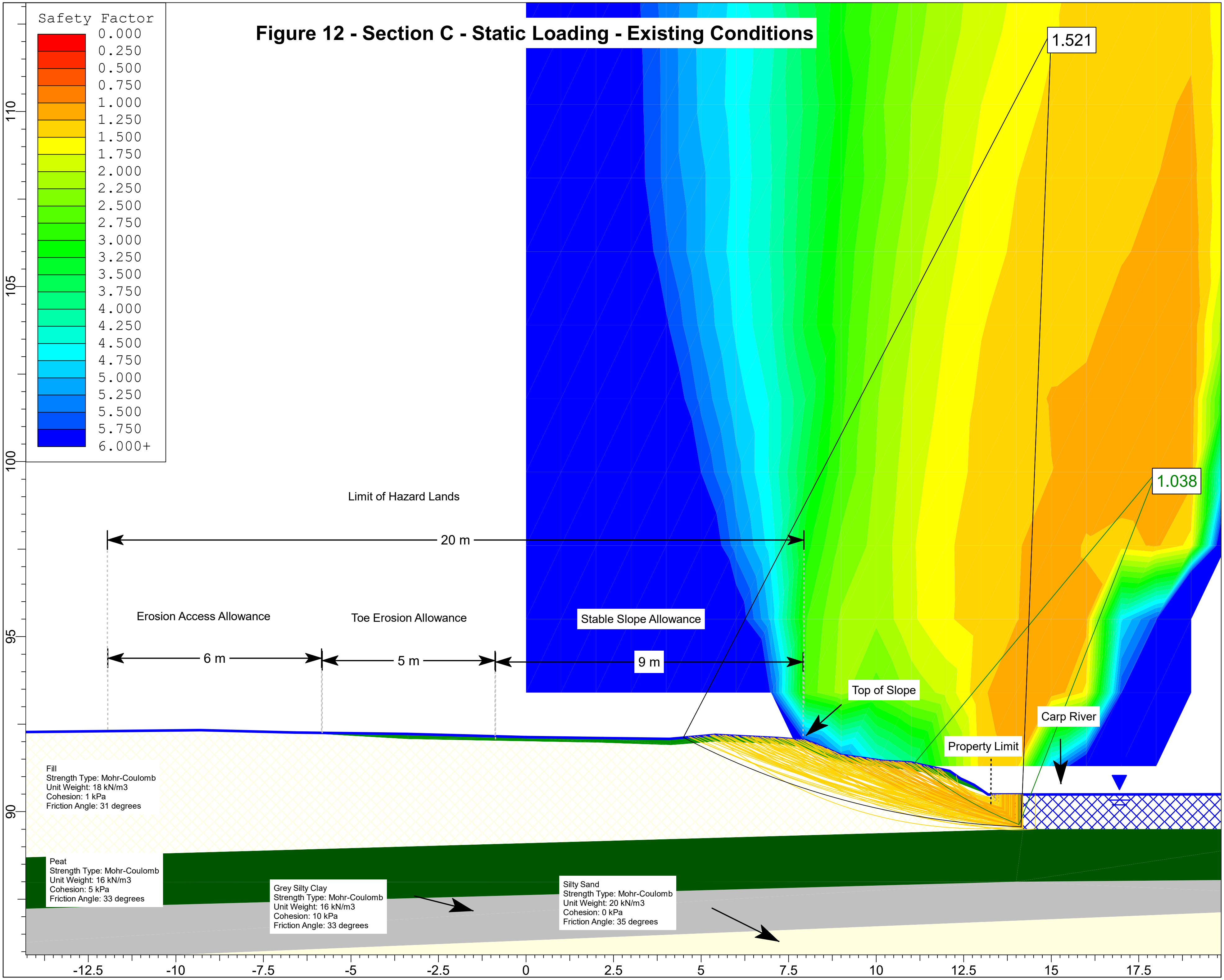


Figure 11 - Section B - Seismic Loading - Existing Conditions

Figure 12 - Section C - Static Loading - Existing Conditions



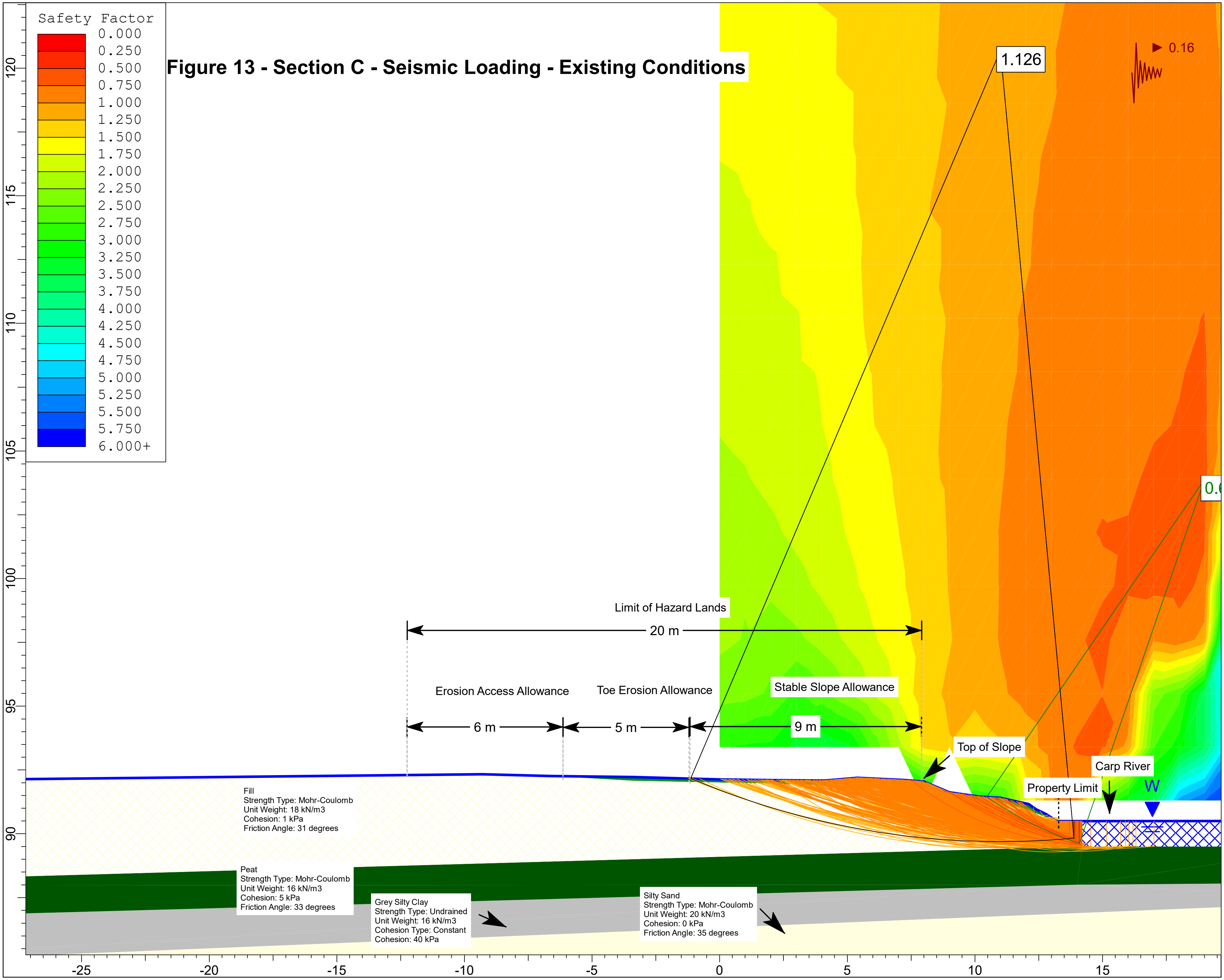


Figure 13 - Section C - Seismic Loading - Existing Conditions

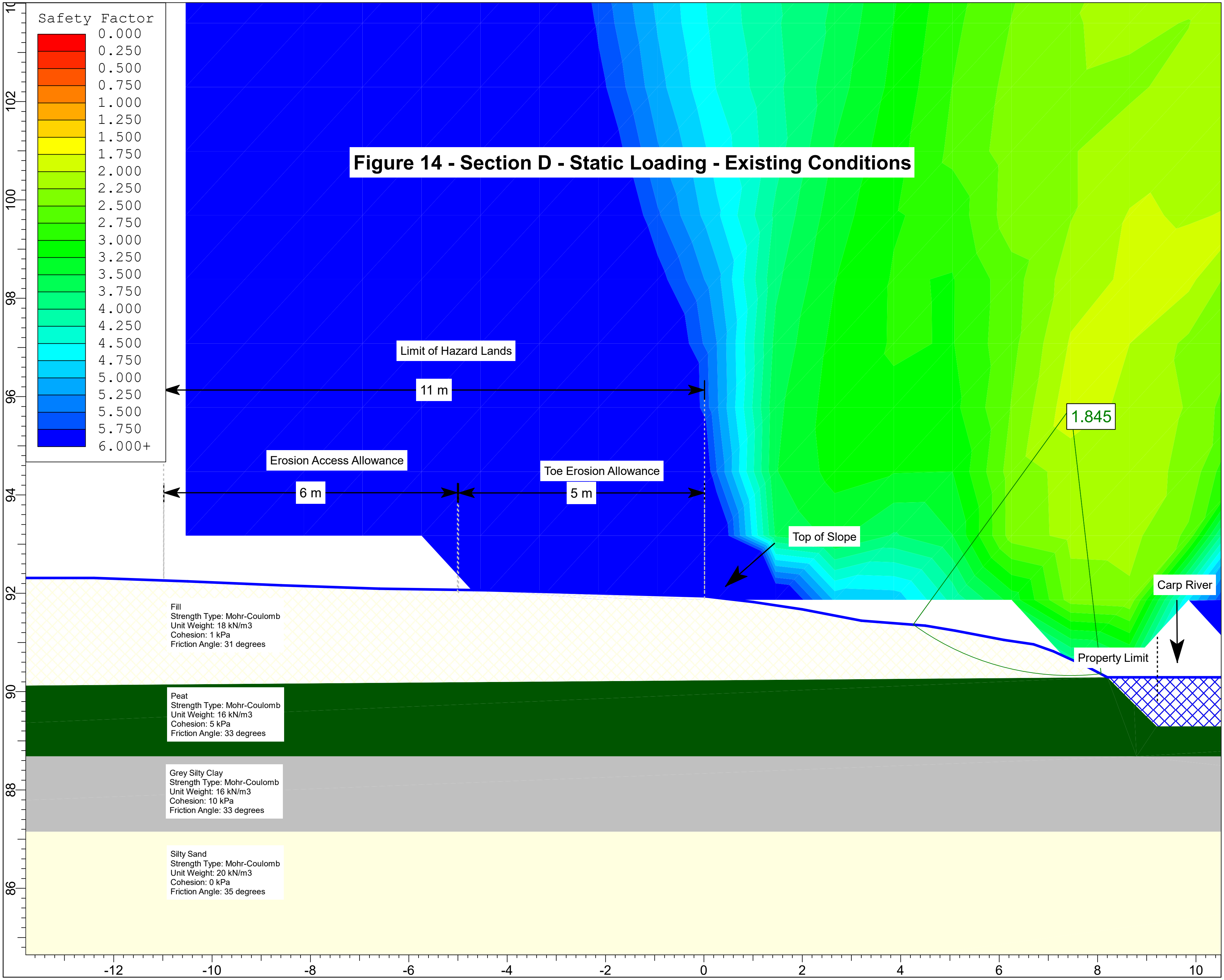


Figure 14 - Section D - Static Loading - Existing Conditions

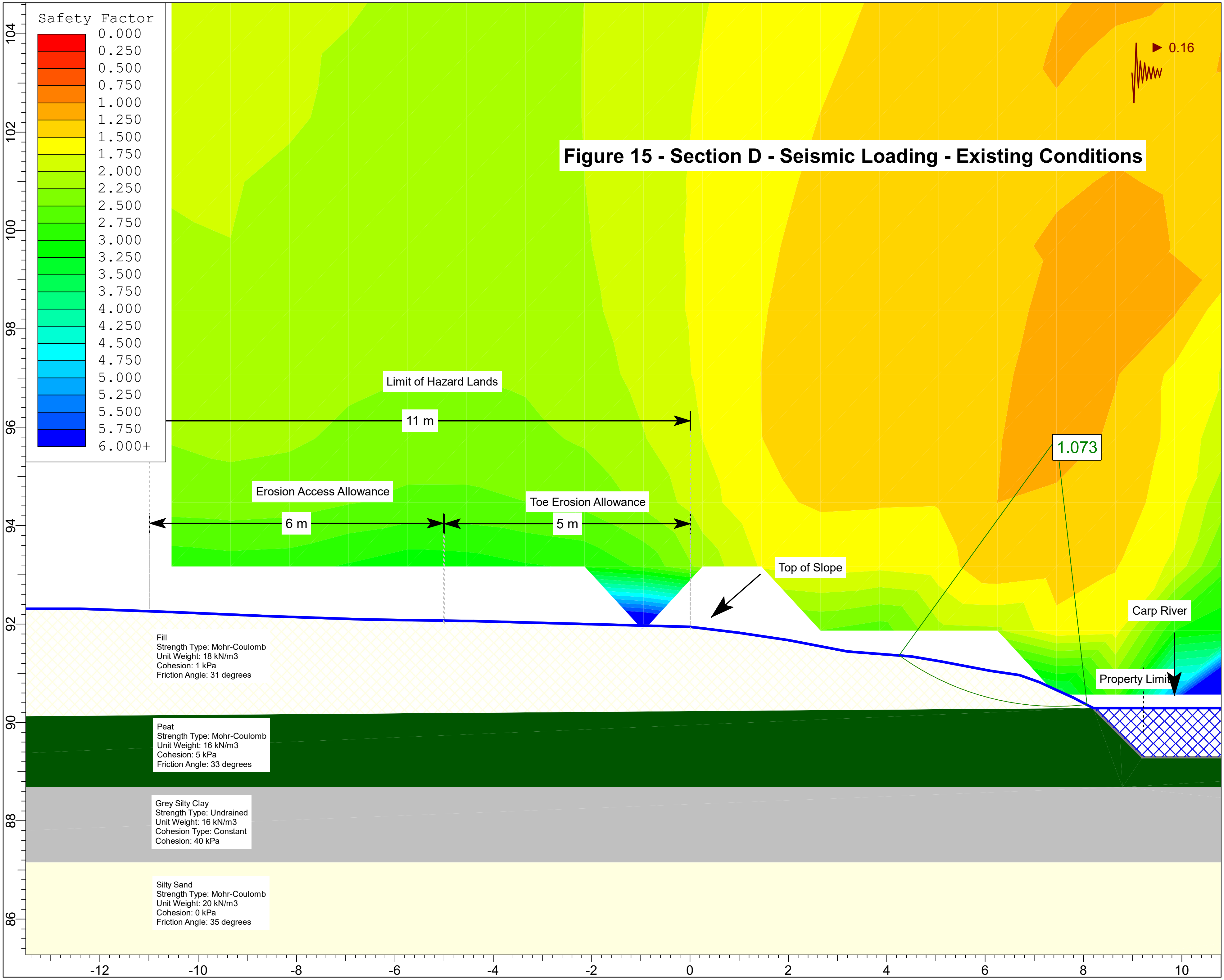


Figure 15 - Section D - Seismic Loading - Existing Conditions

Photo 1: Area of Cross-Section A. Photograph taken at north portion of site, facing southeast.

Photo 2: Area of Cross-Section A. Sidewall of watercourse and slope face observed to be covered with vegetation. Photograph taken facing southwest.

Photo 3: Area of Cross-Section B. The gentle slope face is observed to be covered with vegetation. Photograph taken facing northwest.

Photo 4: Watercourse observed to be overflooding the slope face. General area observed to be covered with vegetation. Photograph taken facing west.

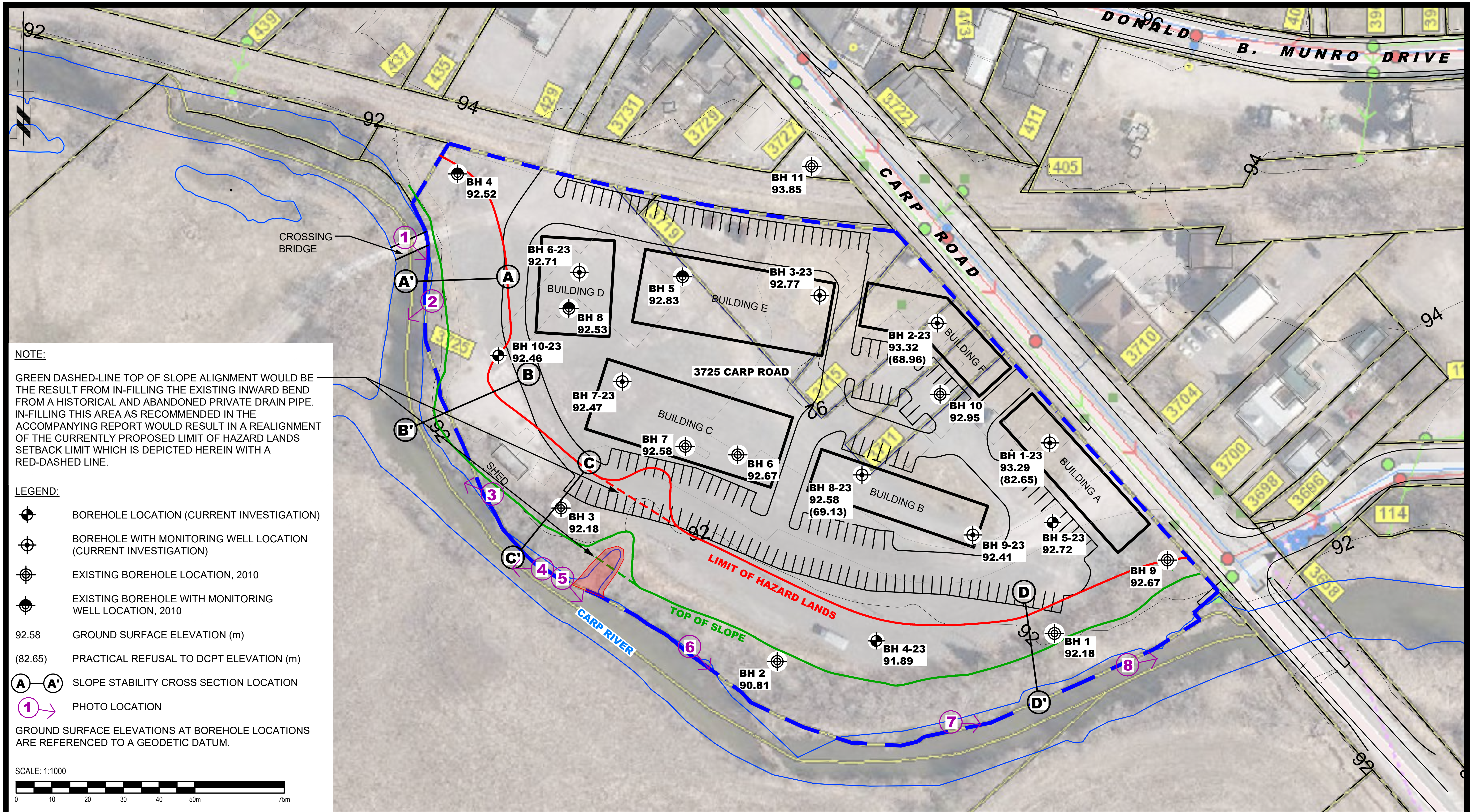
Photographs from Site Visit – January 9, 2023

Photo 5: Photograph of area south of Cross-Section C. The slope face is observed to be smooth and covered with vegetation. Photograph taken facing southeast.

Photo 6: Area of watercourse covered with vegetation and did not appear to have been affected by erosion. Photograph taken facing southeast.

Photo 7: Watercourse sidewall is observed to be covered with vegetation. Photograph taken facing east.

Photo 8: Slope face and watercourse sidewall observed to be covered with vegetation. Signs of erosion were not observed during the site visit. Photograph taken facing northeast.



NOTE:

GREEN DASHED-LINE TOP OF SLOPE ALIGNMENT WOULD BE THE RESULT FROM IN-FILLING THE EXISTING INWARD BEND FROM A HISTORICAL AND ABANDONED PRIVATE DRAIN PIPE. IN-FILLING THIS AREA AS RECOMMENDED IN THE ACCOMPANYING REPORT WOULD RESULT IN A REALIGNMENT OF THE CURRENTLY PROPOSED LIMIT OF HAZARD LANDS SETBACK LIMIT WHICH IS DEPICTED HEREIN WITH A RED-DASHED LINE.

LEGEND:

⊕	BOREHOLE LOCATION (CURRENT INVESTIGATION)
⌖	BOREHOLE WITH MONITORING WELL LOCATION (CURRENT INVESTIGATION)
⊕	EXISTING BOREHOLE LOCATION, 2010
⊕	EXISTING BOREHOLE WITH MONITORING WELL LOCATION, 2010
92.58	GROUND SURFACE ELEVATION (m)
(82.65)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)
Ⓐ—Ⓐ'	SLOPE STABILITY CROSS SECTION LOCATION
①→	PHOTO LOCATION

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

SCALE: 1:1000

0 10 20 30 40 50m 75m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

KARSON HOLDINGS INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
3725 CARP ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000
Date: 04/2023
Drawn by: YA
Report No.: PG2103-1
Checked by: DP
Dwg. No.: PG2103-1
Approved by: FA
Revision No.:

11x17

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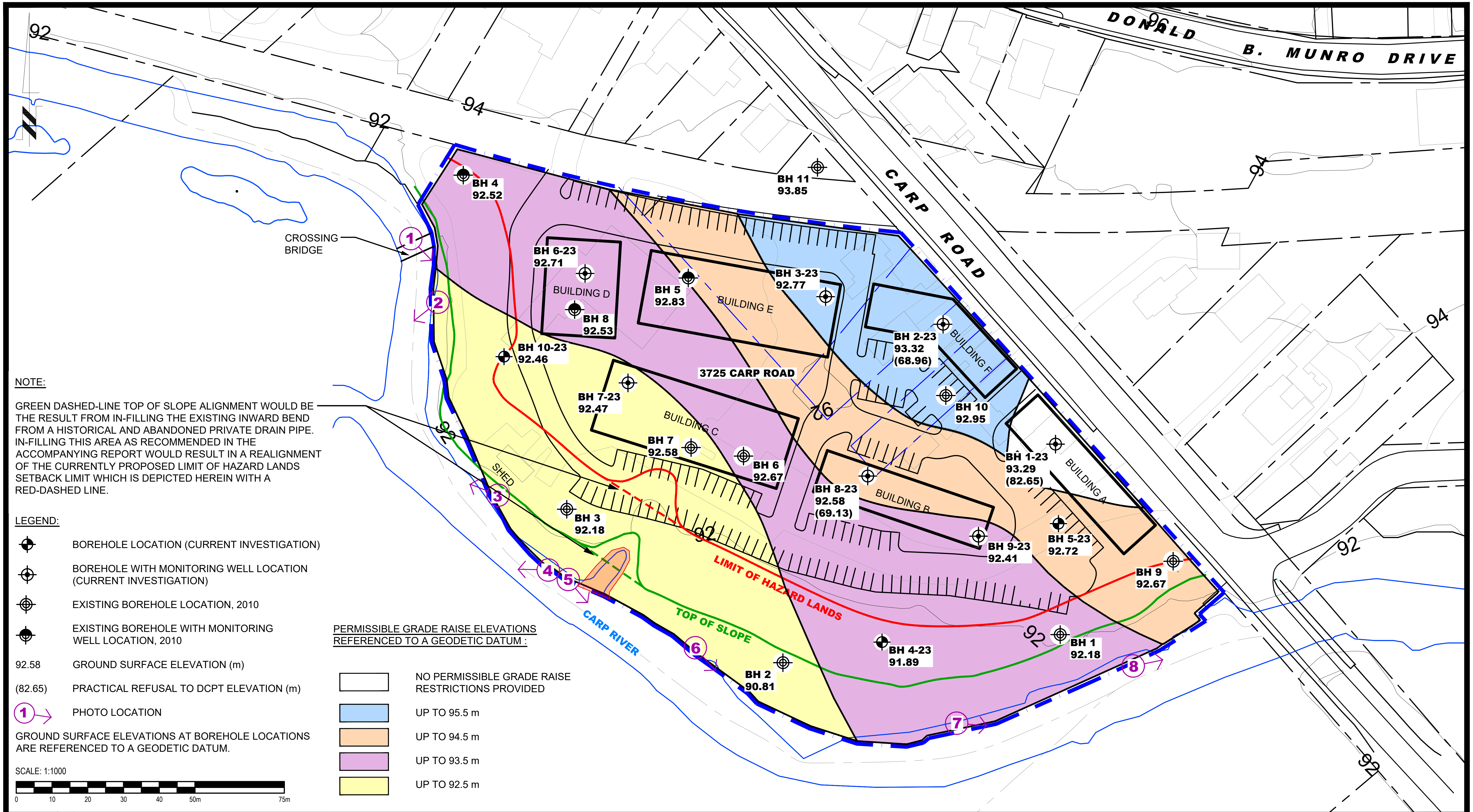

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(82.65)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)
① →	PHOTO LOCATION

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

SCALE: 1:1000

0 10 20 30 40 50m 75m

PERMISSIBLE GRADE RAISE ELEVATIONS REFERENCED TO A GEODETIC DATUM :

☐	NO PERMISSIBLE GRADE RAISE RESTRICTIONS PROVIDED
🟦	UP TO 95.5 m
🟧	UP TO 94.5 m
🟪	UP TO 93.5 m
🟨	UP TO 92.5 m

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KARSON HOLDINGS INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
3725 CARP ROAD
OTTAWA, ONTARIO

Title:
PERMISSIBLE GRADE RAISE PLAN

Scale: 1:1000
Date: 04/2023
Drawn by: YA
Report No.: PG2103-1
Checked by: DP
Dwg. No.: PG2103-2
Approved by: FA
Revision No.:

11x17

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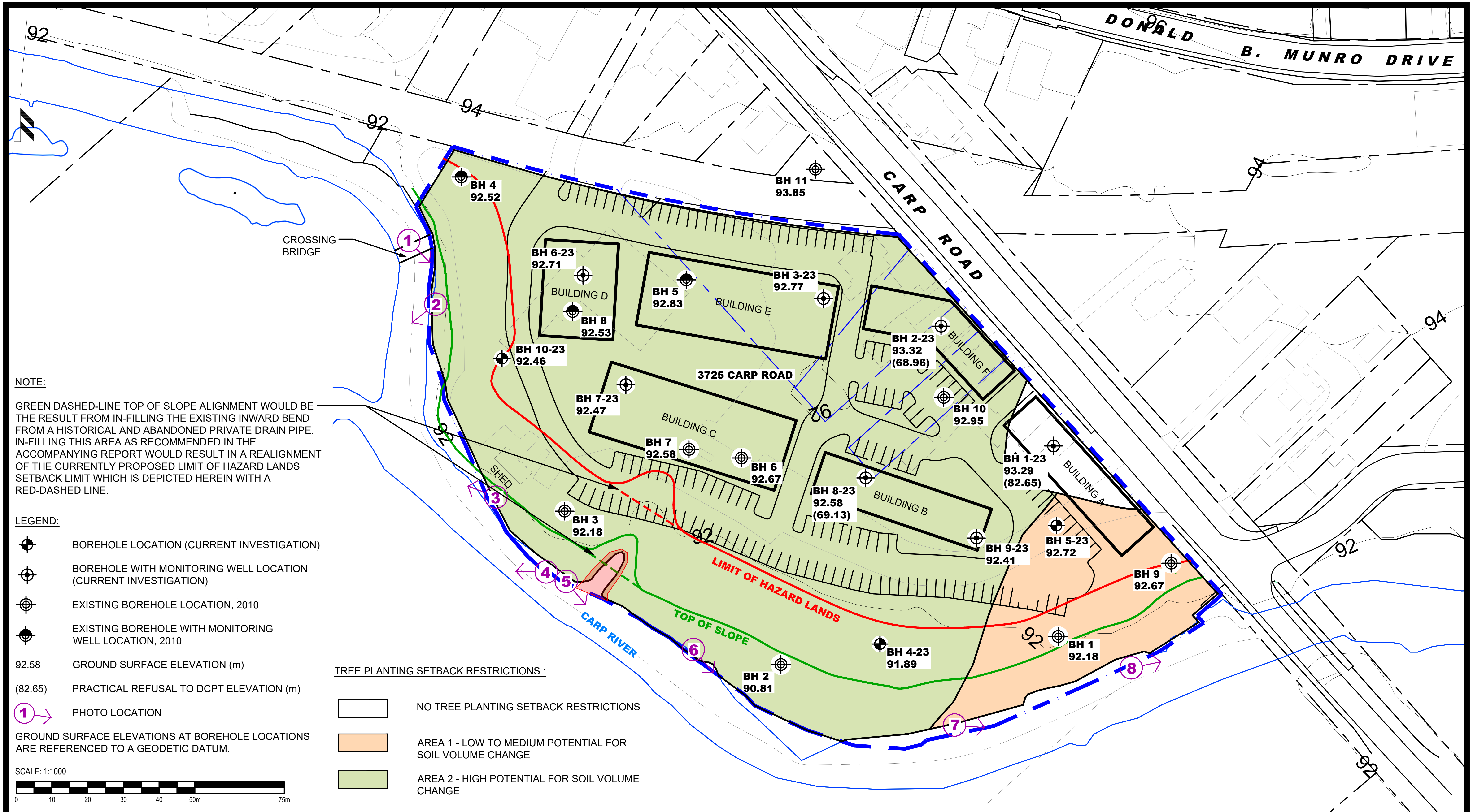

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GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:1000

0 10 20 30 40 50m 75m

TREE PLANTING SETBACK RESTRICTIONS :

☐	NO TREE PLANTING SETBACK RESTRICTIONS
🟧	AREA 1 - LOW TO MEDIUM POTENTIAL FOR SOIL VOLUME CHANGE
🟩	AREA 2 - HIGH POTENTIAL FOR SOIL VOLUME CHANGE

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KARSON HOLDINGS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
3725 CARP ROAD

OTTAWA, ONTARIO

Title:

TREE PLANTING SETBACK RESTRICTIONS

Scale: 1:1000	Date: 04/2023
Drawn by: YA	Report No.: PG2103-1
Checked by: DP	Dwg. No.: PG2103-3
Approved by: FA	Revision No.:

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