

Geotechnical Investigation Proposed Industrial Building

3113 Carp Road
Carp, Ontario

Prepared for Megha Holdings Inc.

Report PG7598-1 dated August 4, 2026



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1.0 Introduction

Paterson Group (Paterson) was commissioned by Meghan Holdings Inc. to conduct a geotechnical investigation for the proposed industrial Building, located at 3113 Carp Road, Carp, Ontario (refer to Figure 1 – Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of test holes.
- ☐ Provide geotechnical recommendations pertaining to the design of the proposed development, including construction considerations which may affect the design.

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

2.0 Proposed Development

Based on the available most recent site plan, it is understood that the proposed industrial development will consist of a single-storey industrial building of slab-on-grade construction along with associated access lanes, loading decks, parking areas, retaining walls and landscaped areas.

It is also understood that the proposed building will be serviced by a private sewage system and well.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was conducted between June 23 and June 28, 2022. At that time, a total of **seven (7) boreholes** were advanced to a maximum depth of 7.3 m below the existing ground surface.

The test hole locations were distributed across the site in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on the Drawing PG7598-1 – Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered from the auger flights or using a 50 mm diameter split-spoon sampler. The split-spoon samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the auger flight and split-spoon samples were recovered from the boreholes are depicted as AU and SS, respectively, on the Soil Profile and Test Datasheets in Appendix 1.

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

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at borehole BH 5-22. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

Boreholes BH 3-22 and BH 6-22 were fitted with 51 mm diameter monitoring wells. The other boreholes were fitted with flexible piezometers to allow groundwater level monitoring. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ 1.5 m of slotted 51 mm diameter PVC screen at the base of the boreholes.
- ☐ 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.
- ☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed warehouse addition, taking into consideration the existing site features and underground utilities.

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson personnel using a GPS unit and referenced to a geodetic datum and presented in Drawing PG7598-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site consists of agricultural land. The ground surface across the subject site is generally flat and slightly lower than the neighbouring roads and lots.

The site is bordered by Carp Road followed by single-family dwellings to the east, agricultural land to the south, a vacant lot to the west, and a church with associated landscaped areas to the north. A residential dwelling abuts the northeastern portion of the property, fronting Carp Road.

4.2 Subsurface Profile

Generally, the subsurface soil profile at the test hole locations consists of topsoil underlain by a layer of loose brown silty sand, followed by a silty clay deposit. The upper portion of the silty clay deposit was observed to be hard to stiff and brown. Furthermore, the lower portion of the silty clay deposit below was observed to be stiff and grey by approximately 4.4m depth below the existing ground surface.

Within the eastern portion of the site, a layer of loose to compact silty sand to sandy silty was encountered below the topsoil and/or silty clay deposit, at the locations of BH 1-22 through BH 4-22. Running sand condition was observed within this layer at depths of 4.6m and 5.3m below the existing ground surface.

Practical refusal to DCPT was encountered at an approximate depth of 13.7 m at BH 5-22.

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

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of Precambrian Paragneiss with an overburden drift thickness ranging between 15 and 25 m depth.

Grain Size Distribution and Hydrometer Testing

Two (2) grain size distribution and hydrometer testing were completed by Paterson to classify selected soil samples according to the Unified Soil Classification System (USCS). The results are summarized in Table 1 and on the Grain Size Distribution sheets in Appendix 1.

Table 1 – Grain Size Distribution and Hydrometer Testing Results							
Borehole	Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Classification
BH 3-22	SS2	1.37	0.0	6.5	53.0	40.5	Silty Clay
BH 5-22	SS2	1.37	0.0	19.4	41.1	39.5	Silty Clay

Atterberg Limit Testing

Two (2) selected silty clay samples were submitted for Atterberg limits testing by Paterson. The results are summarized in Table 2 and on the Atterberg Limits Results sheet in Appendix 1.

Table 2 – Atterberg Limits Testing Results						
Borehole	Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 3-22	SS2	1.37	54	24	30	CH
BH 5-22	SS2	1.37	59	25	34	CH
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plastic Index. CH: Inorganic Clay of High Plasticity						

Shrinkage Testing

The shrinkage limit and shrinkage ratio of the tested silty clay sample (BH 4-22, SS2) were found to be 21.84 and 1.718, respectively.

The results of the linear shrinkage testing are presented in Appendix 1.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells and piezometers installed at the borehole locations on July 8, 2022. The measured groundwater levels noted at that time are presented in Table 3. The groundwater level readings are also presented in the Soil Profile and Test Data sheets in Appendix 1.

Table 3 – Summary of Groundwater Levels				
Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level / Groundwater Infiltration for Test Pits		Dated Recorded
		Depth (m)	Elevation (m)	
BH1-22	117.21	1.67	115.54	July 8, 2022
BH2-22	116.97	1.00	115.97	
BH3-22	116.78	1.22	115.56	
BH4-22	116.44	0.71	115.73	
BH5-22	116.37	0.71	115.66	
BH6-22	116.33	1.00	115.33	
BH7-22	116.34	1.01	115.33	
Note: The ground surface elevation at each borehole location was surveyed using a high precision GPS unit and are referenced to a geodetic datum.				

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Long-term groundwater levels can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 3 to 4 m below the ground surface.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed industrial development. It is expected that the proposed building can be constructed over conventional shallow footings placed on an undisturbed, very stiff brown silty clay bearing surface. Where the footing subgrade is found to consist of loose silty sand bearing surface, the material should be proof-rolled using a suitable vibratory equipment making several passes under dry conditions and above freezing temperatures, and reviewed by Paterson at the time of construction.

Due to the presence of a silty clay deposit, the proposed development will be subjected to grade raise restrictions. The permissible grade raise recommendations are discussed in subsection 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures. Precautions should be taken to ensure that all bearing surfaces and subgrade soils remain undisturbed during site preparation activities. Under paved areas, existing construction remnants, such as foundation walls, pipe ducts, etc., should be excavated to a minimum depth of 1 m below final grade.

Fill Placement

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

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. Site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Proof Rolling

It is expected that site grading and preparation will consist of stripping of the soils containing significant amounts of organic materials. The contractor should take appropriate precautions to avoid disturbing the subgrade and bearing surfaces from construction and worker traffic. Any loose or disturbed areas within the subgrade level, below the proposed footings is recommended to be proof-rolled under dry conditions and above freezing temperatures by an adequately sized roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by the geotechnical consultant. In poor performing areas, consideration may be given to removing the poor performing soil and replace with an approved engineered fill such as OPSS Granular A or Granular B Type II compacted to a minimum 98% of the material's SPMDD.

5.3 Foundation Design

Bearing Resistance Values (Conventional Spread Footings)

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, very stiff to stiff 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**.

Footings placed directly on an undisturbed, compact silty sand 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**. It is recommended that the bearing surface be proof-rolled using a suitably sized vibratory roller, making several passes under dry and above freezing conditions in areas where the silty sand is found to be in a loose state of compactness. Paterson personnel should complete periodic inspections during the proof-rolling operations.

A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Settlement

Footings placed on a soil bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

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 existing soil bearing surface above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher bearing capacity as the bearing medium soil.

Permissible Grade Raise Restriction

Based on the current borehole information, a **permissible grade raise restriction of 2.5 m** is recommended for the proposed building and settlement sensitive structures. A post-development groundwater lowering of 0.5 m was assumed for our calculations.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to determine the applicable seismic site designation for the proposed building in accordance with the Ontario Building Code 2024. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 3 and 4 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as presented in Drawing PG7598-1 – Test Hole Location Plan, attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 3 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio.

The shot locations are also completed in forward and reverse directions (i.e.-striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 25, 4.5 and 3 m away from the first geophone and last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30} , of the upper 30 m profile, immediately below the foundation of the buildings. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on our testing results, the average overburden shear wave velocity is **258 m/s**, while the bedrock shear wave velocity is **2,159 m/s**. Through interpretation the bedrock surface was interpreted at approximately 17 m below existing ground surface. Considering the proposed development is founded on conventional footings.

The V_{s30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2024.

$$V_{s30} = \frac{\text{Depth}_{of\ interest}(m)}{\left(\frac{\text{Depth}_{Layer1}(m)}{V_{sLayer1}(m/s)} + \frac{\text{Depth}_{Layer2}(m)}{V_{sLayer2}(m/s)} \right)}$$

$$V_{s30} = \frac{30\ m}{\left(\frac{17\ m}{258\ m/s} + \frac{13\ m}{2,159\ m/s} \right)}$$

$$V_{s30} = 417\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30} , is **417 m/s** for conventional footings placed on a silty clay bearing surface. Therefore, a **Site Designation X₄₁₇** is applicable for the design of the proposed building according to the Ontario Building Code 2024.

Although some lower “N” values were recorded in some of the boreholes in the silty sand, this is considered to be the result of the running sand encountered in these boreholes, which is noted on these Soil Profile and Test Data sheets. Running sands typically occur when augering in cohesionless soils below the groundwater table, which creates a differential hydrostatic pressure head between the outside and inside of the auger, causing sand to flow into the augers. This loosens the column of sand tested during the SPT, resulting in “N” values which do not reflect the true in-situ density of such cohesionless soils.

Accordingly, the soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2024 for a full discussion of the earthquake design requirements.

5.5 Floor Slab Construction

With the removal of all topsoil and deleterious fill within the footprint of the proposed warehouse, the existing soil subgrade, reviewed and approved by Paterson personnel at the time of construction, will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction. Where soils are noted to be in a loose state of compaction, a proof-rolling program witnessed by Paterson personnel is recommended to be completed.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone for slab on grade construction. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Pavement Design

Car only parking areas, access lanes and loading areas are anticipated at this site. The proposed pavement structures are shown in Tables 4 and 5.

Table 4 – Recommended Flexible Pavement Structure – Car Only Parking Areas	
Thickness (mm)	Material Description
50	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE – OPSS Granular A Crushed Stone
300	SUBBASE – OPSS Granular B Type II
SUBGRADE – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

Table 5 – Recommended Flexible Pavement Structure – Access Lanes and Heavy Truck Parking and Loading Areas	
Thickness (mm)	Material Description
40	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
50	Wear Course – HL-8 or Superpave 19 Asphaltic Concrete
150	BASE – OPSS Granular A Crushed Stone
400	SUBBASE – OPSS Granular B Type II
SUBGRADE – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

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 material's SPMDD using suitable compaction equipment.

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

It is further understood that a roller compacted concrete (RCC) or heavy duty concrete pavement structure is planned for the proposed development for loading areas. Our recommended pavement structure for roller compacted concrete is presented in Table 6.

Table 6 – Recommended Roller Compacted Concrete Pavement Structure Heavy Truck Access Lanes, Parking and Loading Areas	
Thickness (mm)	Material Description
200	Zero slump, no air entrainment -32 MPa at 28-day strength - 12 to 13% cement content
300	SUBBASE – OPSS Granular A Crushed Stone
SUBGRADE – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

The compacted concrete must be compacted to a minimum 100% of its SPMDD and frequently tested by Paterson personnel. A well graded aggregate blend with a maximum particle size of 19 mm is further recommended, as well as, a water-cement ratio of 0.25 to 0.35. Moisture control during curing is critical for the RCC pavement structure. To control cracking, it is recommended that strategically located saw cuts be used to create control joints at a 8 to 10 m spacing. The joints should be cut between 25 and 30% of the thickness of the concrete slab.

If shallow footings are proposed along the loading docks, it is recommended that Paterson review the finalized drawings to provide frost protection, if applicable.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. The subdrain inverts should be approximately 300 mm below subgrade level, and the subgrade surface should be crowned to promote water flow to drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed industrial building. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe wrapped in a geosock, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The clear stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta-Terraxx or equivalent, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

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 insulation equivalent) should be provided in this regard.

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

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Where space restrictions exist, or to reduce the trench width, the excavation can be carried out within the confines of a fully braced steel trench box.

Unsupported Excavations

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

The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

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

6.4 Pipe Bedding and Backfill

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

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

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

The trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

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

Groundwater Control

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

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

6.6 Winter Construction

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

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

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

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

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Setbacks

In general accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing and grain size distribution analysis was completed for recovered silty clay samples at selected locations throughout the subject site. The above noted test results were completed between design underside of footing elevation and a 3.5 m depth below anticipated finished grade. The results of our testing are presented in Table 1 and Table 2 in Subsection 4.2 and in Appendix 1.

A low to medium sensitivity clay soil was encountered between anticipated underside of footing elevations and 3.5 m below anticipated finished grades at the subject site. Based on our Atterberg Limits test results, the plasticity index limit generally does not exceed 40% across the subject site. Therefore, the following tree planting setbacks are recommended for the low to medium sensitivity areas.

Large trees (mature height over 14 m) can be planted within this area 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).

A tree planting setback limit of **4.5 m** is applicable for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m 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 centre 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 of 25 m³ of available soil volume while a medium tree must be provided with a minimum of 30 m³ of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.
- ❑ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.
- ❑ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).
- ❑ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision Grading Plan.

6.9 Global Stability Analysis

Paterson Group (Paterson) completed a global stability analysis of the proposed concrete retaining wall system to be located at the aforementioned development. The retaining wall system has been checked for global stability, considering the site constraints and grading to date. The analysis was completed for the worst-case scenario (with the highest wall section on the most recent civil drawing) as per the *Canadian Highway and Bridge Design Code (CHBDC) 2019*.

The global stability analysis was modeled using SLIDE2, a computer program which permits a two-dimensional slope stability analysis using several limit equilibrium analysis methods, which are widely used and accepted analysis methods. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain the risks of failure are acceptable.

A minimum factor of safety of 1.5 is generally recommended for conditions where the slope failure would comprise permanent structures. An analysis considering seismic loading was also completed. A horizontal acceleration of 0.15 g (which is taken as 0.5 of the area's peak ground acceleration (PGA), was considered for the section for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The proposed retaining wall has been verified for global stability and has an adequate factor of safety in excess of the required 1.5 for static conditions and 1.1 for seismic loading conditions. The applicable seismic design incorporates a PGA of 0.300, as per NBCC 2020.

The retaining wall section was reviewed using the design loading according to CHBDC 2019.

A representative section was evaluated as a worst-case scenario for soil condition and loading possible to encounter on site at the location of the proposed retaining wall.

The highest section of the proposed retaining wall at the subject site was analyzed in Figures 4A and 4B, presented in Appendix 2. The subsurface soil conditions at this location were interpreted based on the nearest boreholes completed as part of our geotechnical investigation.

The following parameters in Table 7 were used for the slope stability analysis under static and seismic conditions:

Table 7 – Effective Soil Parameters for Stability Analysis			
Soil Layer	Unit Weight (kN/m³)	Friction Angle (degrees)	Cohesion (kPa)
Engineered Fill	19	33	0
OPSS Granular A or Granular B Type II	22	36	0
Silty Sand to Sandy Silt	18	33	2
Brown Silty Clay	17	33	5

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strengths recovered within the open boreholes completed at the time of our geotechnical investigation and based on our general knowledge of the geology in the area. The following parameters presented in Table 8 below were used for the slope stability analysis under seismic conditions.

Table 8 – Total Strength Soil Parameters for Seismic Analysis			
Soil Layer	Unit Weight (kN/m³)	Friction Angle (degrees)	Cohesion (kPa)
Engineered Fill	19	33	0
OPSS Granular A or Granular B Type II	22	36	0
Silty Sand to Sandy Silt	18	33	2
Brown Silty Clay	17	0	100

Analysis Results

The factor of safety for the retaining wall section was found to be greater than 1.5 for static conditions. Similarly, the results under seismic loading yielded a factor of safety for this section greater than 1.1. Based on these results, the retaining wall design is considered suitable from a geotechnical perspective. Reference should be made to Figures 4A and 4B in Appendix 2.

Additional Geotechnical Recommendations – Proposed Retaining Wall

Backfill Material

The retaining wall should be backfilled with free-draining granular backfill materials and incorporate longitudinal drains and weep holes to provide positive drainage for the accumulated surface water within the backfill material of the retaining wall. For the purpose of this report, it is recommended that the wall be backfilled with either OPSS Granular B Type II or Granular A materials.

The backfill should be placed within a wedge-shaped zone defined by a line drawn up and back from the back edge of the base of the wall at an inclination of 1H:1V or a minimum of 1 m behind the back of the wall, unless otherwise noted in the design drawing. All material should be compacted to a minimum of 98% of the material's SPMDD.

A non-woven geotextile liner such as Terrafix 270R or equivalent should be placed between the backfill material and the retained soil to ensure that the backfill material can retain its free draining characteristics.

Drainage

A 100 mm diameter perforated drainage pipe wrapped in geotextile such as Terrafix 270R or equivalent approved other, surrounded on all sides by 150 mm of clear crushed stone, should be installed at the heel of the bottom block. The drainage system should have outlets through the wall to drain at a nearby catch basin, drainage ditch, or ground surface. Drainage outlets should be spaced at minimum intervals of 10 m.

Retaining Wall Construction Recommendations and Considerations

It is recommended that the following be completed once the retaining wall design and course of action are determined:

- ☐ Observation of all bearing surfaces prior to backfill placement.
- ☐ Observation of all subgrades prior to placing backfilling materials.
- ☐ Observation of the drainage system prior to backfilling.
- ☐ Field density tests to ensure the specified level of compaction was achieved.
- ☐ Periodic observation of the retaining wall installation.

It is further recommended that all bedding and backfill materials be placed under dry conditions and above freezing temperatures and approved by the geotechnical consultant at the time of construction. Precautions should be taken to ensure that the bedding material does not freeze before placement of the retaining wall, which could lead to detrimental movement within the retaining wall once the frost leaves the bedding material.

7.0 Recommendations

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

- ☐ Review detailed grading, servicing, landscaping, and structural plan(s) from a geotechnical perspective.
- ☐ Review of the geotechnical aspects of the drainage and/or waterproofing design system, if not designed by Paterson, prior to construction.

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

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

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

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

8.0 Statement of Limitations

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

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

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

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

Paterson Group Inc.



Yashar Ziaeimehr, M.A.Sc., P.Eng.



Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ☐ Megha Holdings Inc. (email copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS TESTING RESULTS

GRAIN SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

SHRINKAGE TESTING RESULTS

ANALYTICAL TESTING RESULTS



SOIL PROFILE AND TEST DATA

DATUM: Geodetic

FILE NO. **PG7598**

REMARKS:

HOLE NO. **BH 1-22**

BORINGS BY: Track-Mount Power Auger

DATE: June 23, 2022

[illegible]



SOIL PROFILE AND TEST DATA

DATUM: Geodetic

FILE NO. **PG7598**

REMARKS:

HOLE NO. **BH 2-22**

BORINGS BY: Track-Mount Power Auger

DATE: June 23, 2022

[illegible]



**PATERSON
GROUP**

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Industrial Building - 3113 Carp Road
Ottawa, Ontario

DATUM: Geodetic

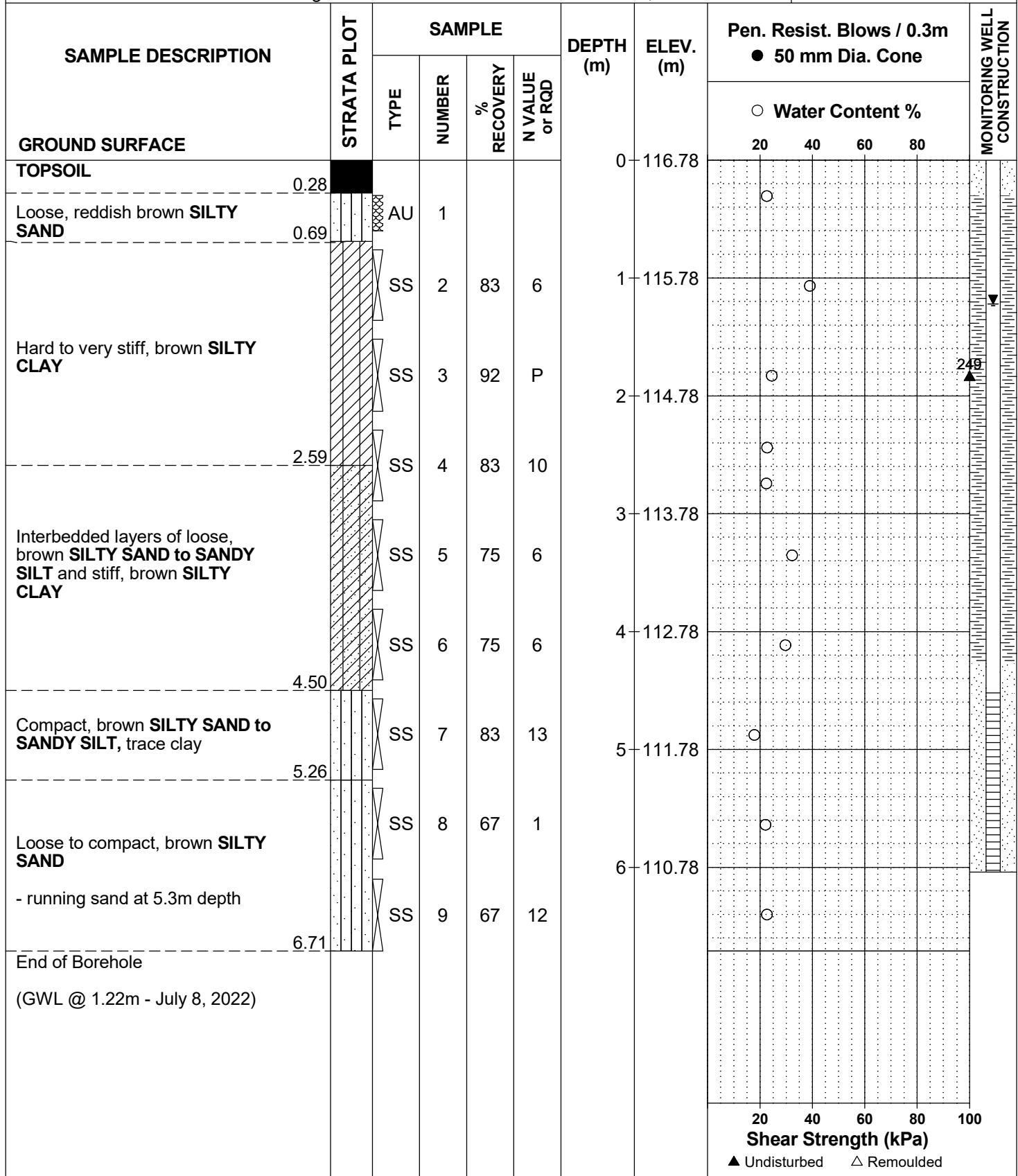
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REMARKS:

HOLE NO. **BH 3-22**

BORINGS BY: Track-Mount Power Auger

DATE: June 23, 2022



SOIL PROFILE AND TEST DATA

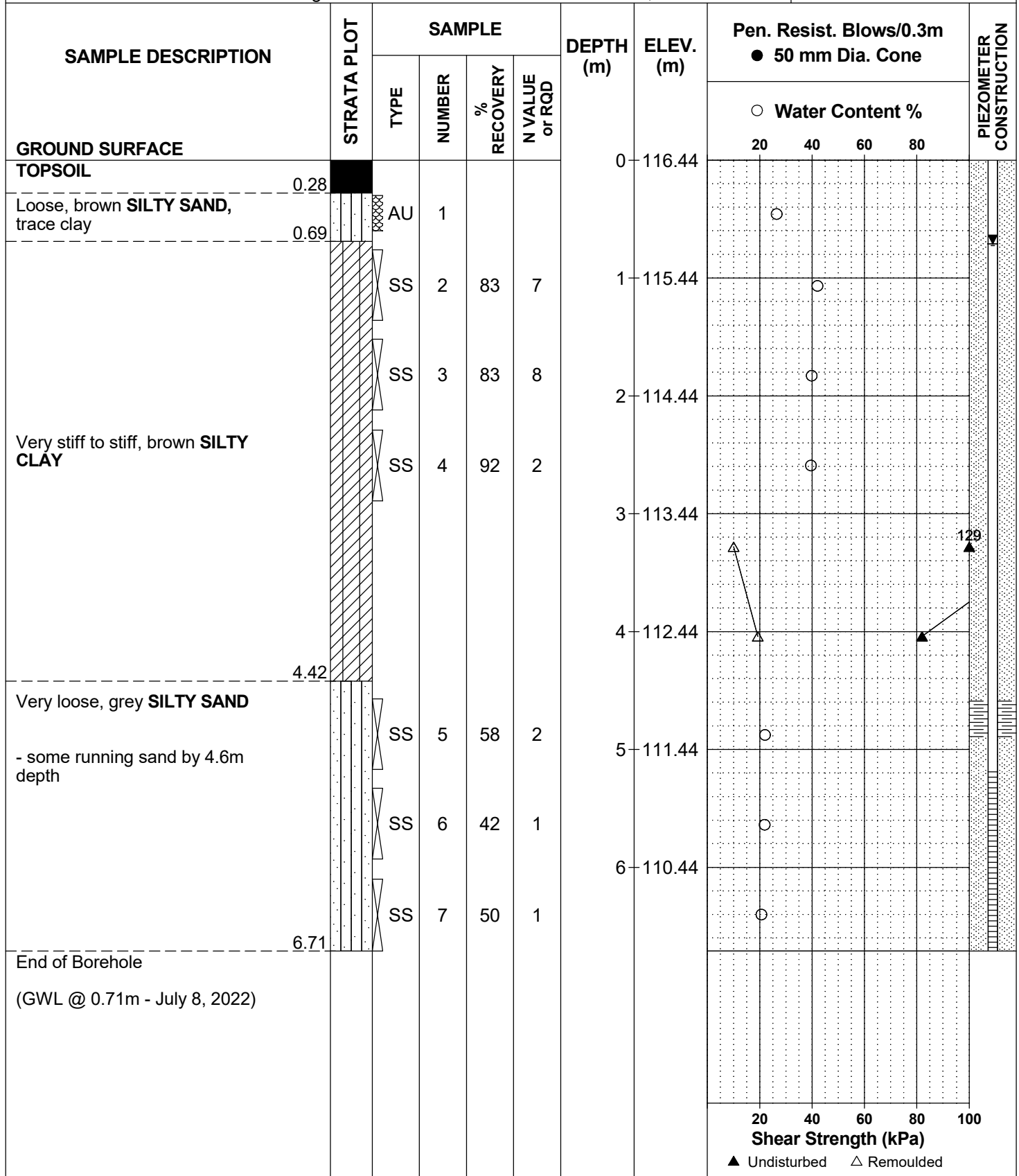
Geotechnical Investigation
Proposed Industrial Building - 3113 Carp Road
Ottawa, Ontario

DATUM: Geodetic

FILE NO. PG7598

REMARKS:
HOLE NO. BH 4-22

BORINGS BY: Track-Mount Power Auger

DATE: June 23, 2022




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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Industrial Building - 3113 Carp Road
Ottawa, Ontario

DATUM: Geodetic

FILE NO. **PG7598**

REMARKS:

HOLE NO. **BH 4A-22**

BORINGS BY: Track-Mount Power Auger

DATE: June 28, 2022

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	116.44					
OVERBURDEN						1	115.44					
						2	114.44					
						3	113.44					
						4	112.44					
	4.57											
Very loose, grey SILTY SAND - some running at 5.3m depth		SS	1	25	3	5	111.44	○				
		SS	2	33	1			○				
	6.10					6	110.44					
End of Borehole												
								20	40	60	80	100
								Shear Strength (kPa)				
								▲ Undisturbed △ Remoulded				

SOIL PROFILE AND TEST DATA

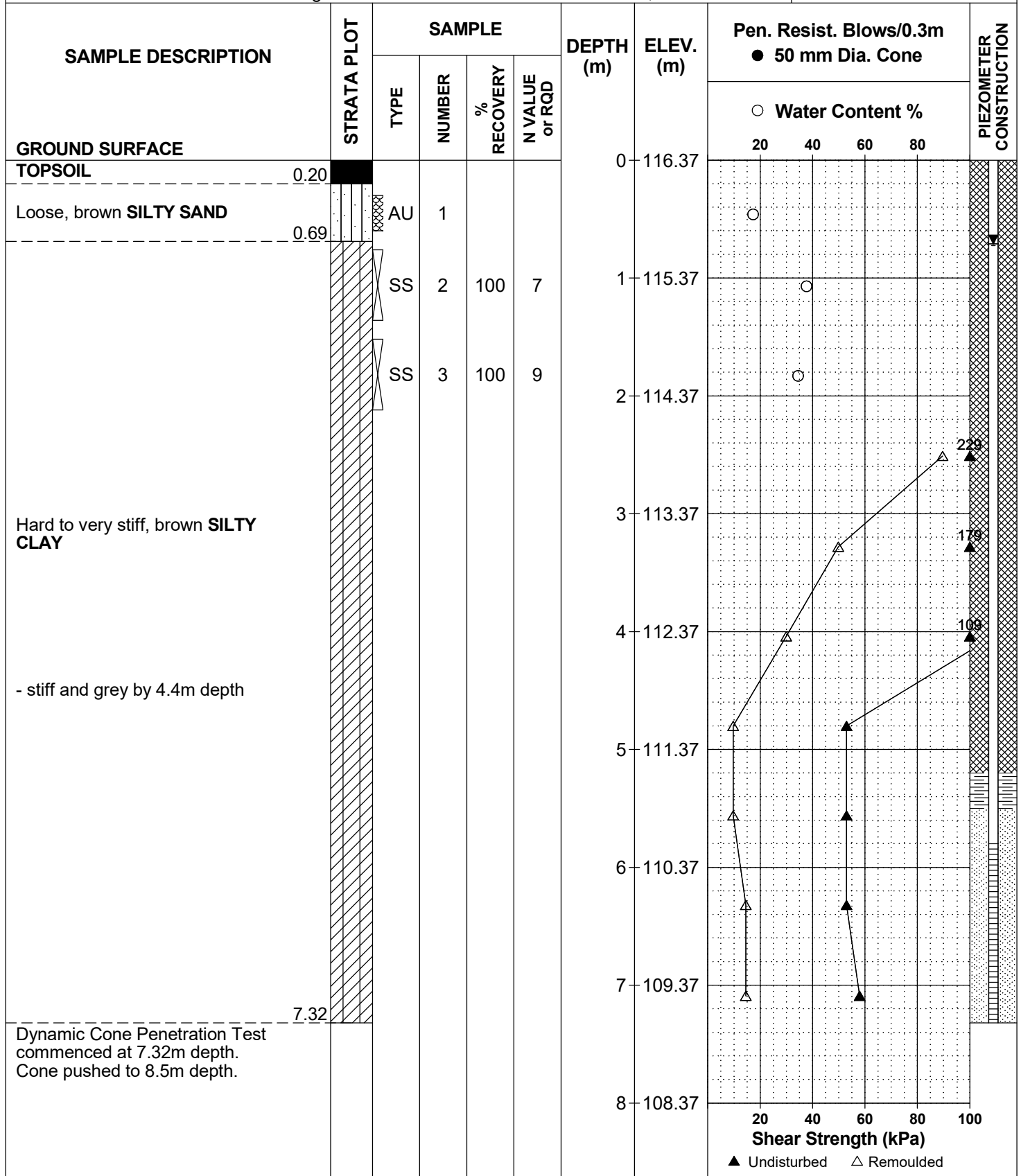
Geotechnical Investigation
Proposed Industrial Building - 3113 Carp Road
Ottawa, Ontario

DATUM: Geodetic

FILE NO. PG7598

REMARKS:
HOLE NO. BH 5-22

BORINGS BY: Track-Mount Power Auger

DATE: June 28, 2022


SOIL PROFILE AND TEST DATA

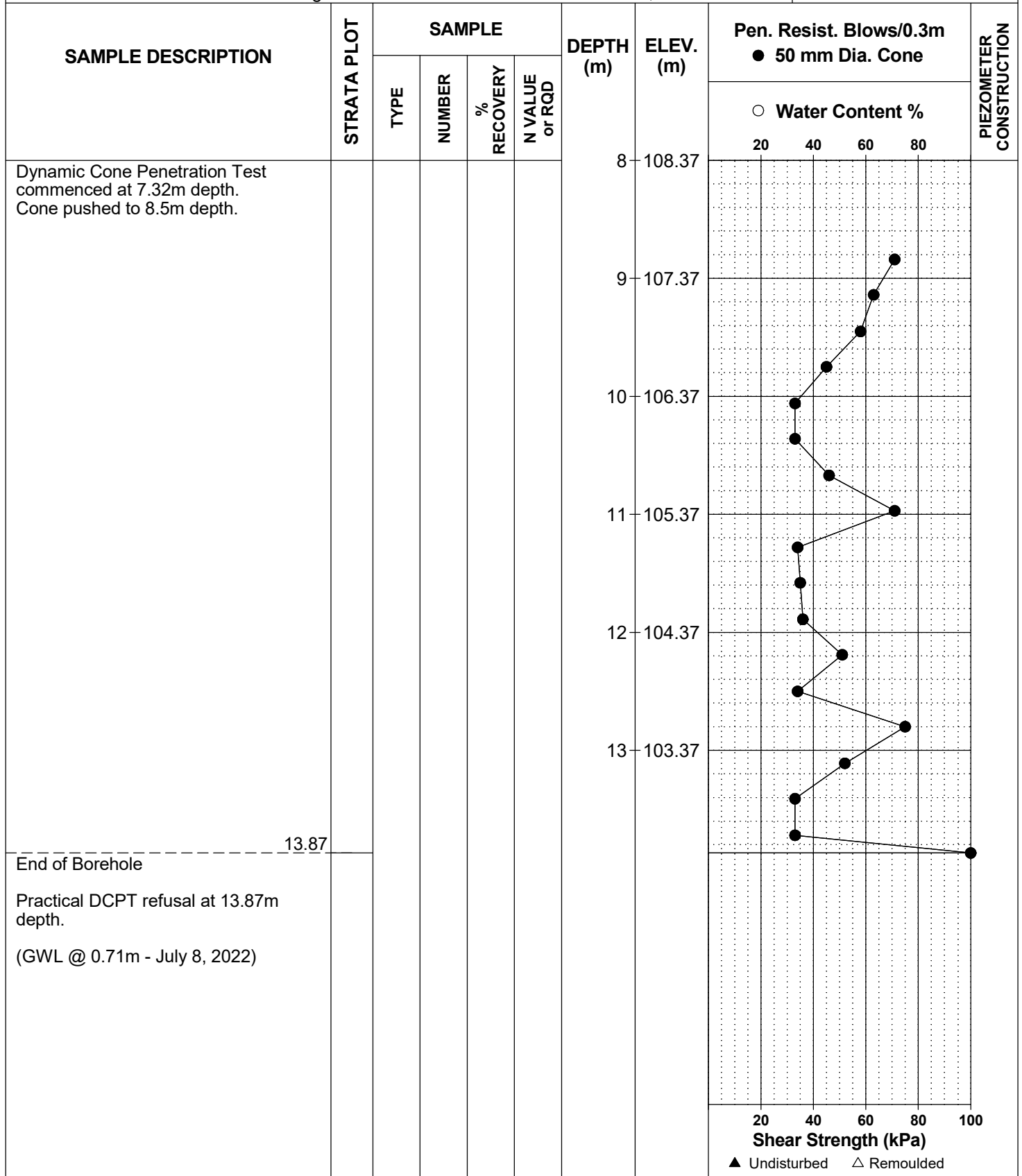
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Proposed Industrial Building - 3113 Carp Road
Ottawa, Ontario

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FILE NO. PG7598

REMARKS:
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BORINGS BY: Track-Mount Power Auger

DATE: June 28, 2022


SOIL PROFILE AND TEST DATA

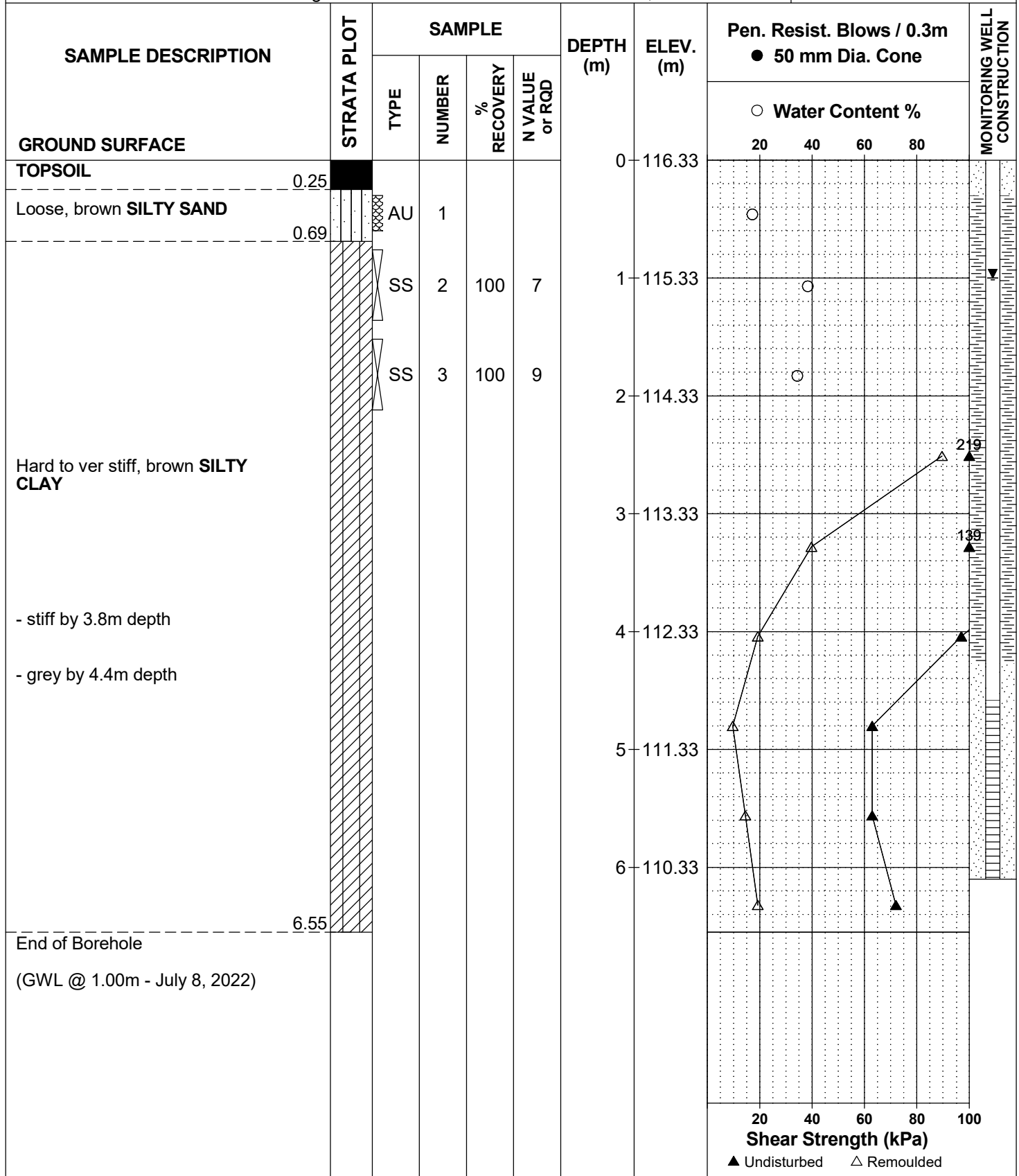
Geotechnical Investigation
Proposed Industrial Building - 3113 Carp Road
Ottawa, Ontario

DATUM: Geodetic

FILE NO. PG7598

REMARKS:
HOLE NO. BH 6-22

BORINGS BY: Track-Mount Power Auger

DATE: June 28, 2022


SOIL PROFILE AND TEST DATA

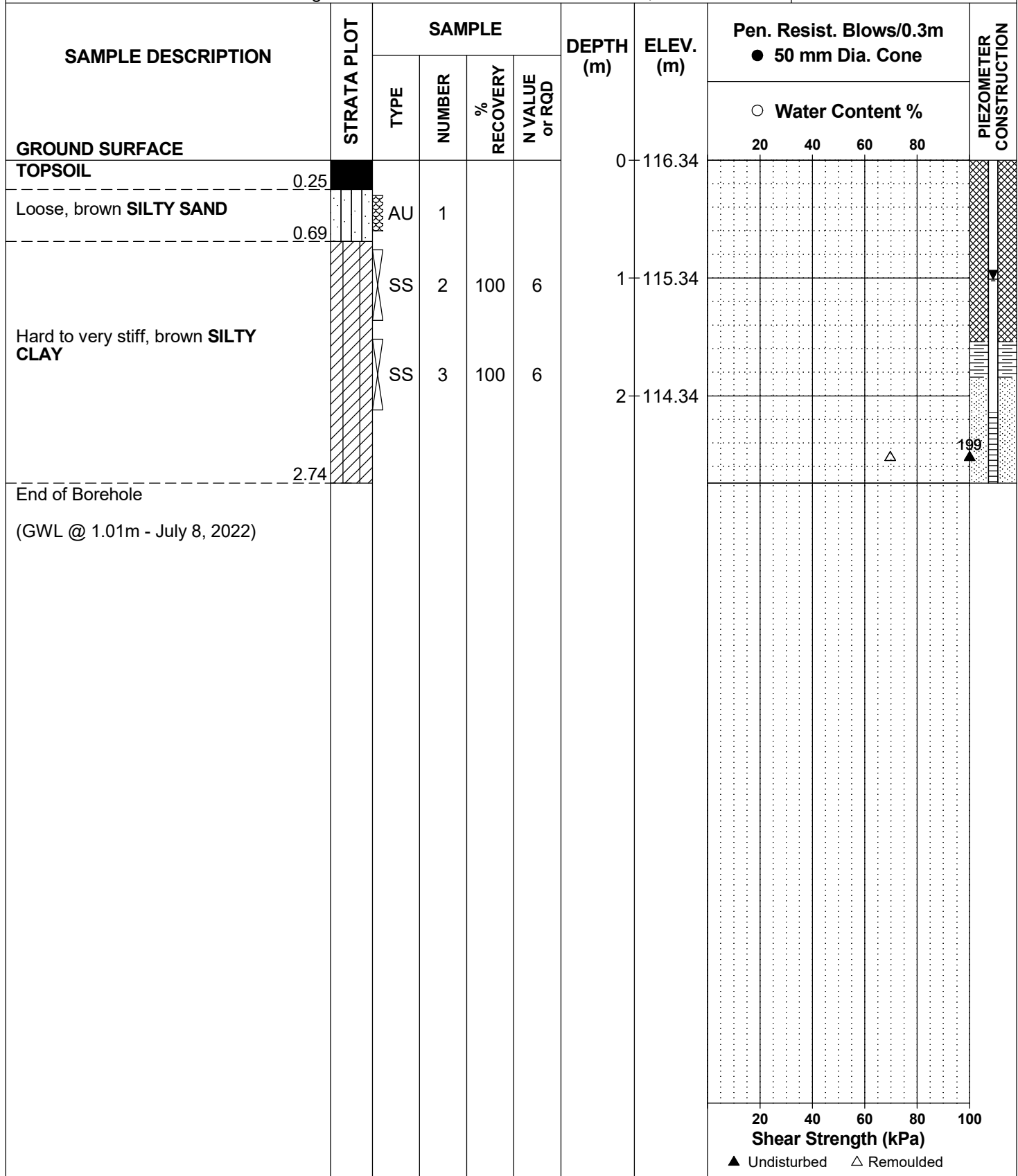
Geotechnical Investigation
Proposed Industrial Building - 3113 Carp Road
Ottawa, Ontario

DATUM: Geodetic

FILE NO. PG7598

REMARKS:
HOLE NO. BH 7-22

BORINGS BY: Track-Mount Power Auger

DATE: June 28, 2022


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 = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = D_{60} / D_{10}

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

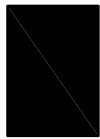
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

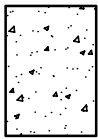
k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during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

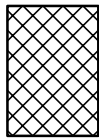
STRATA PLOT



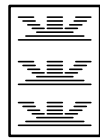
Topsoil



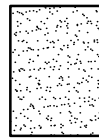
Asphalt



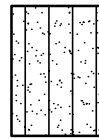
Fill



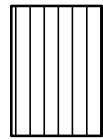
Peat



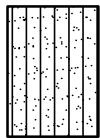
Sand



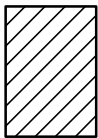
Silty Sand



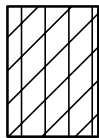
Silt



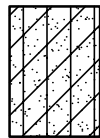
Sandy Silt



Clay



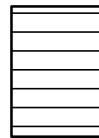
Silty Clay



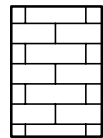
Clayey Silty Sand



Glacial Till



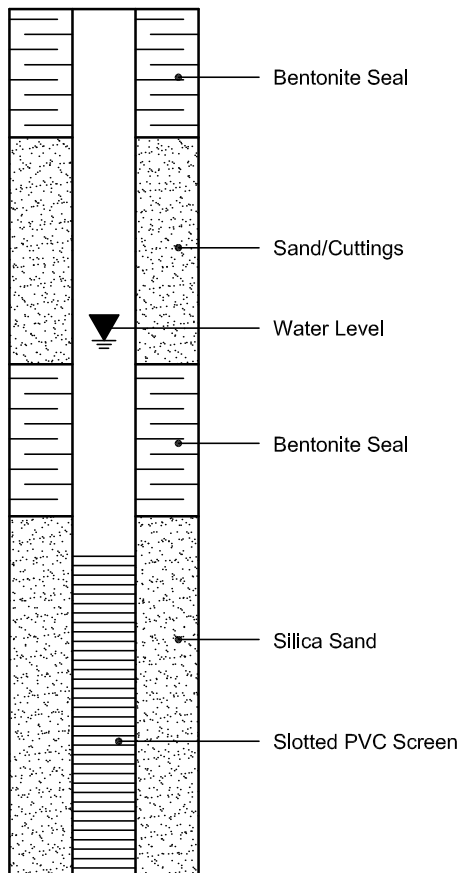
Shale



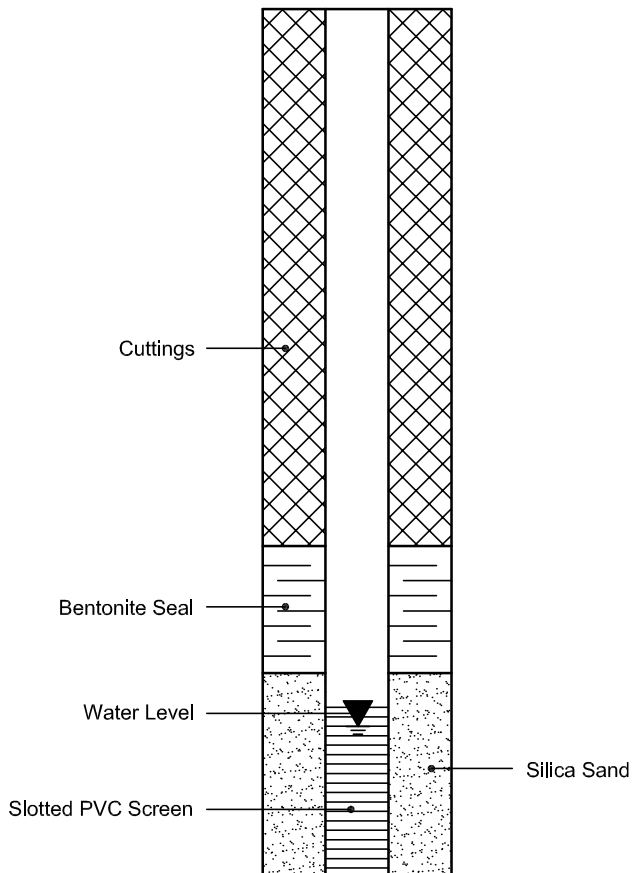
Bedrock

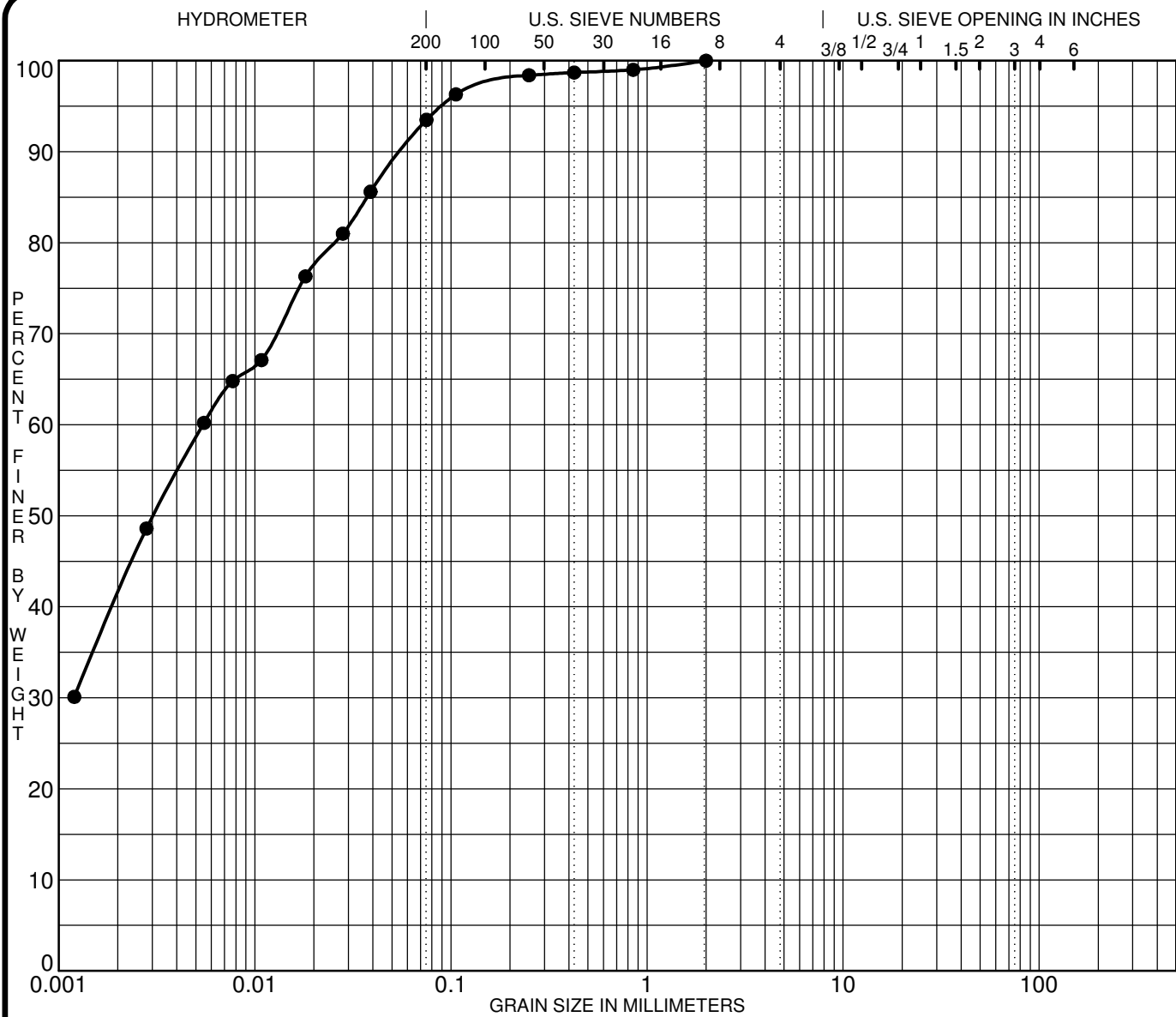
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





Specimen Identification			Classification				MC%	LL	PL	PI	Cc	Cu
●	BH 3-22	SS2	CH - Inorganic clays of high plasticity					54	24	30		
☒												
▲												
★												
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
●	BH 3-22	SS2	2.00	0.01			0.0	6.5	93.5			
☒												
▲												
★												

CLIENT Canadian Rental Development Services

PROJECT Geotechnical Investigation - Proposed Industrial Building - 3113 Carp Road

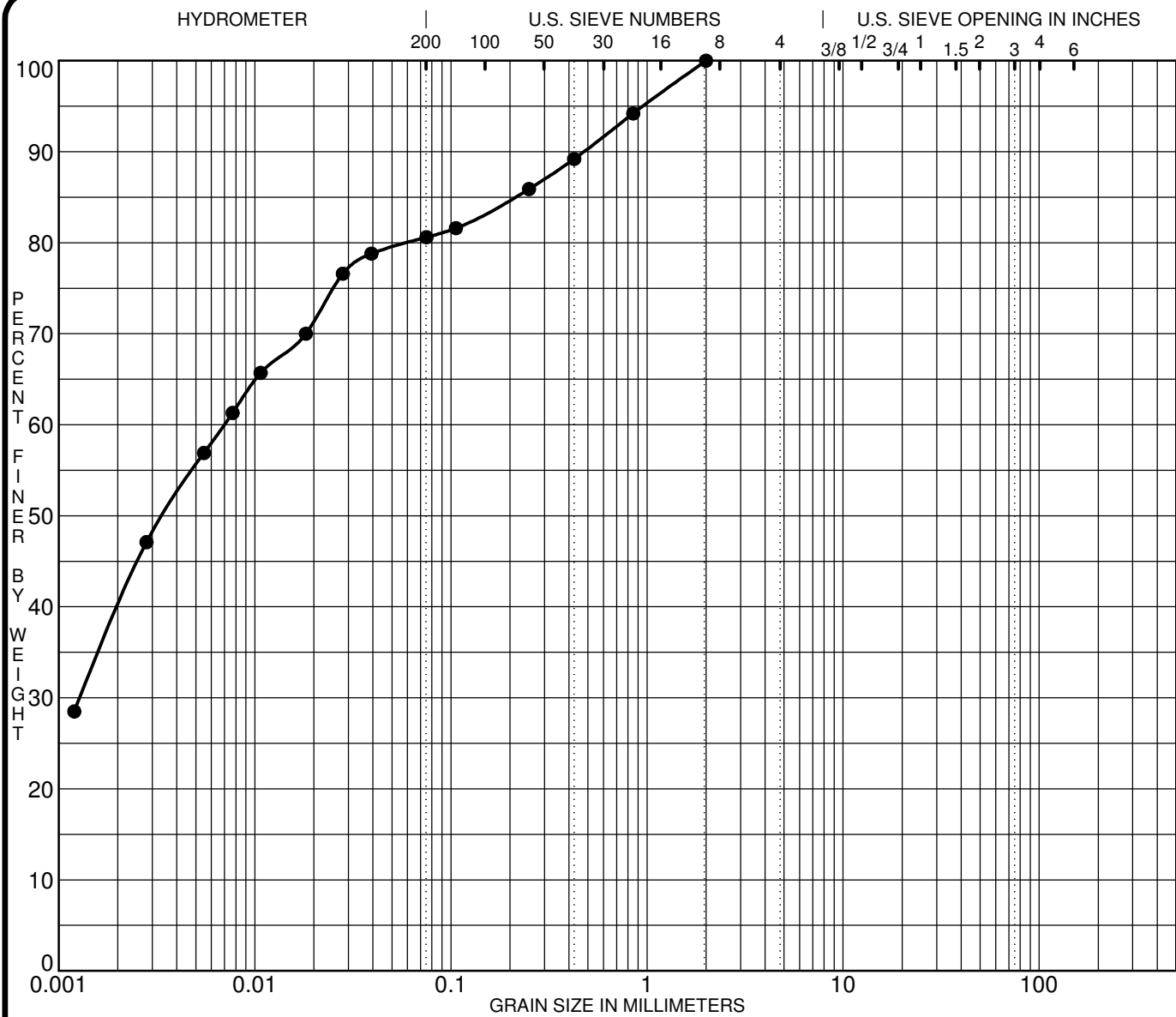
FILE NO. PG6308

DATE 23 Jun 22

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

Specimen Identification			Classification				MC%	LL	PL	PI	Cc	Cu
●	BH 5-22	SS2	CH - Inorganic clays of high plasticity					59	25	34		
☒												
▲												
★												
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
●	BH 5-22	SS2	2.00	0.01	0.001		0.0	19.4	80.6			
☒												
▲												
★												

CLIENT Canadian Rental Development Services

PROJECT Geotechnical Investigation - Proposed Industrial Building - 3113 Carp Road

FILE NO. PG6308

DATE 28 Jun 22

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION

patersongroup consulting engineers				Linear Shrinkage ASTM D4943-02									
CLIENT:	Canadian Rental Dev	DEPTH	2'-6" to 4'-6"	FILE NO.:	PG6308								
PROJECT:	3113 Carp Rd	BH OR TP No:	BH4-22 SS2	DATE SAMPLED	23-Jun								
LAB No:	37024	TESTED BY:	CP/ CS	DATE RECEIVED	18-Aug								
SAMPLED BY:	AE	DATE REPORTED:	29-Aug-22	DATE TESTED	22-Aug								
LABORATORY INFORMATION & TEST RESULTS													
Moisture		No. of Blows(4)	Calibration (Two Trials)		Tin NO.(x15)								
Tare		4.92	Tin	4.78	4.8								
Soil Pat Wet + Tare		63.87	Tin + Grease	4.94	4.92								
Soil Pat Wet		58.95	Glass	48.97	48.97								
Soil Pat Dry + Tare		38.96	Tin + Glass + Water	91.25	91.12								
Soil Pat Dry		34.04	Volume	37.34	37.23								
Moisture		73.18	Average Volume	37.29									
<table border="1"> <tr> <td>Soil Pat + String</td> <td>34.21</td> </tr> <tr> <td>Soil Pat + Wax + String in Air</td> <td>34.95</td> </tr> <tr> <td>Soil Pat + Wax + String in Water</td> <td>14.31</td> </tr> <tr> <td>Volume Of Pat (Vdx)</td> <td>20.64</td> </tr> </table>						Soil Pat + String	34.21	Soil Pat + Wax + String in Air	34.95	Soil Pat + Wax + String in Water	14.31	Volume Of Pat (Vdx)	20.64
Soil Pat + String	34.21												
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Volume Of Pat (Vdx)	20.64												
RESULTS:													
<table border="1"> <tr> <td>Shrinkage Limit</td> <td>21.84</td> </tr> <tr> <td>Shrinkage Ratio</td> <td>1.718</td> </tr> <tr> <td>Volumetric Shrinkage</td> <td>88.227</td> </tr> <tr> <td>Linear Shrinkage</td> <td>19.007</td> </tr> </table>						Shrinkage Limit	21.84	Shrinkage Ratio	1.718	Volumetric Shrinkage	88.227	Linear Shrinkage	19.007
Shrinkage Limit	21.84												
Shrinkage Ratio	1.718												
Volumetric Shrinkage	88.227												
Linear Shrinkage	19.007												
REVIEWED BY:	Curtis Beadow		Joe Forsyth, P. Eng.										
													

Certificate of Analysis

Report Date: 30-Jun-2022

Client: Paterson Group Consulting Engineers

Order Date: 24-Jun-2022

Client PO: 55077

Project Description: PG6308

Client ID:	BH3-22 SS3	-	-	-	-
Sample Date:	23-Jun-22 09:00	-	-	-	-
Sample ID:	2226628-01	-	-	-	-
Matrix:	Soil	-	-	-	-
MDL/Units					

Physical Characteristics

% Solids	0.1 % by Wt.	80.6	-	-	-	-
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General Inorganics

pH	0.05 pH Units	7.85	-	-	-	-
Resistivity	0.1 Ohm.m	68.2	-	-	-	-

Anions

Chloride	5 ug/g	8	-	-	-	-
Sulphate	5 ug/g	<5	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 AND 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURES 4A AND 4B – GLOBAL STABILITY OF PROPOSED RETAINING WALL

DRAWING PG7598-1 – TEST HOLE LOCATION PLAN

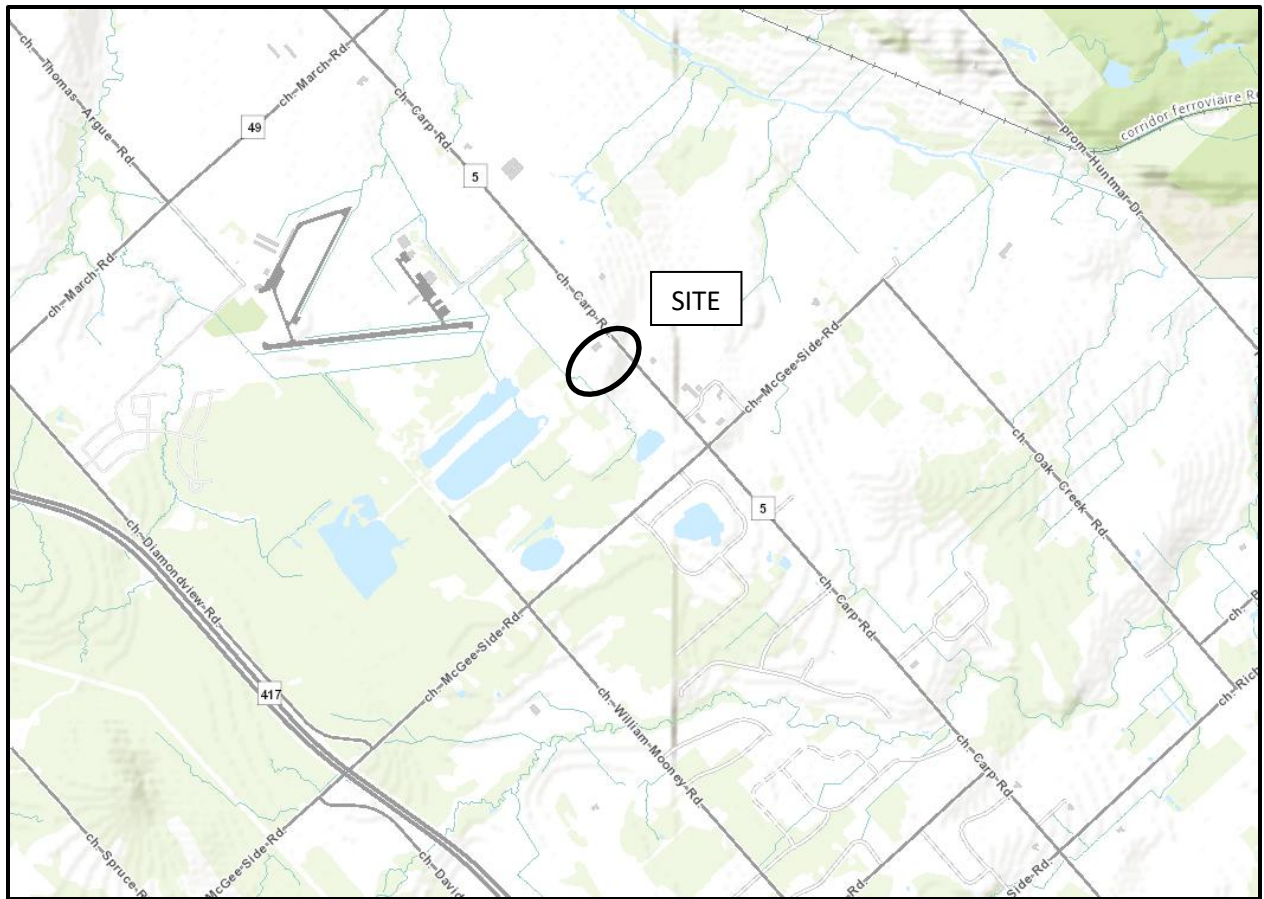


FIGURE 1

KEY PLAN

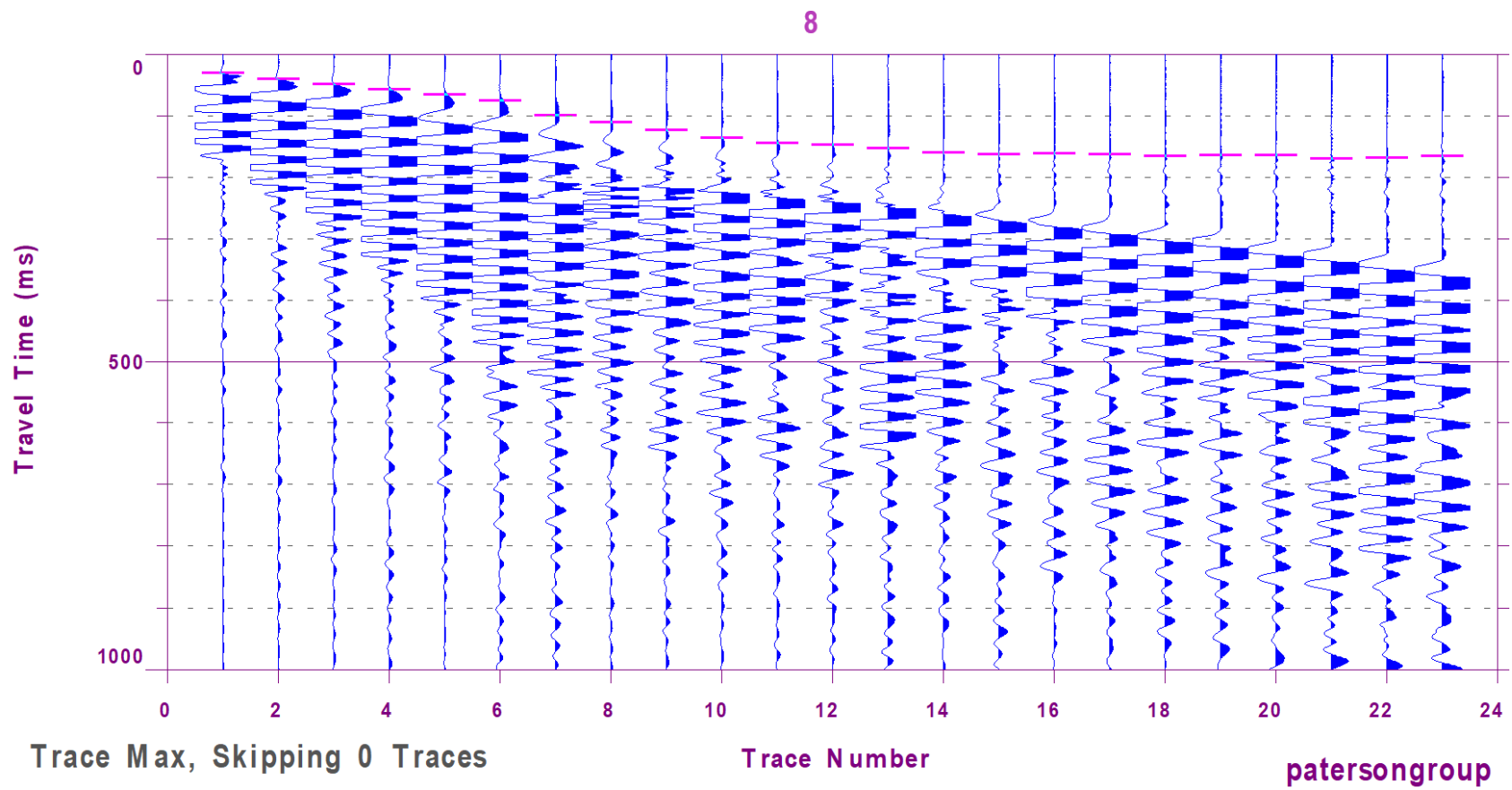


Figure 2 – Shear Wave Velocity Profile at Shot Location -4.5 m

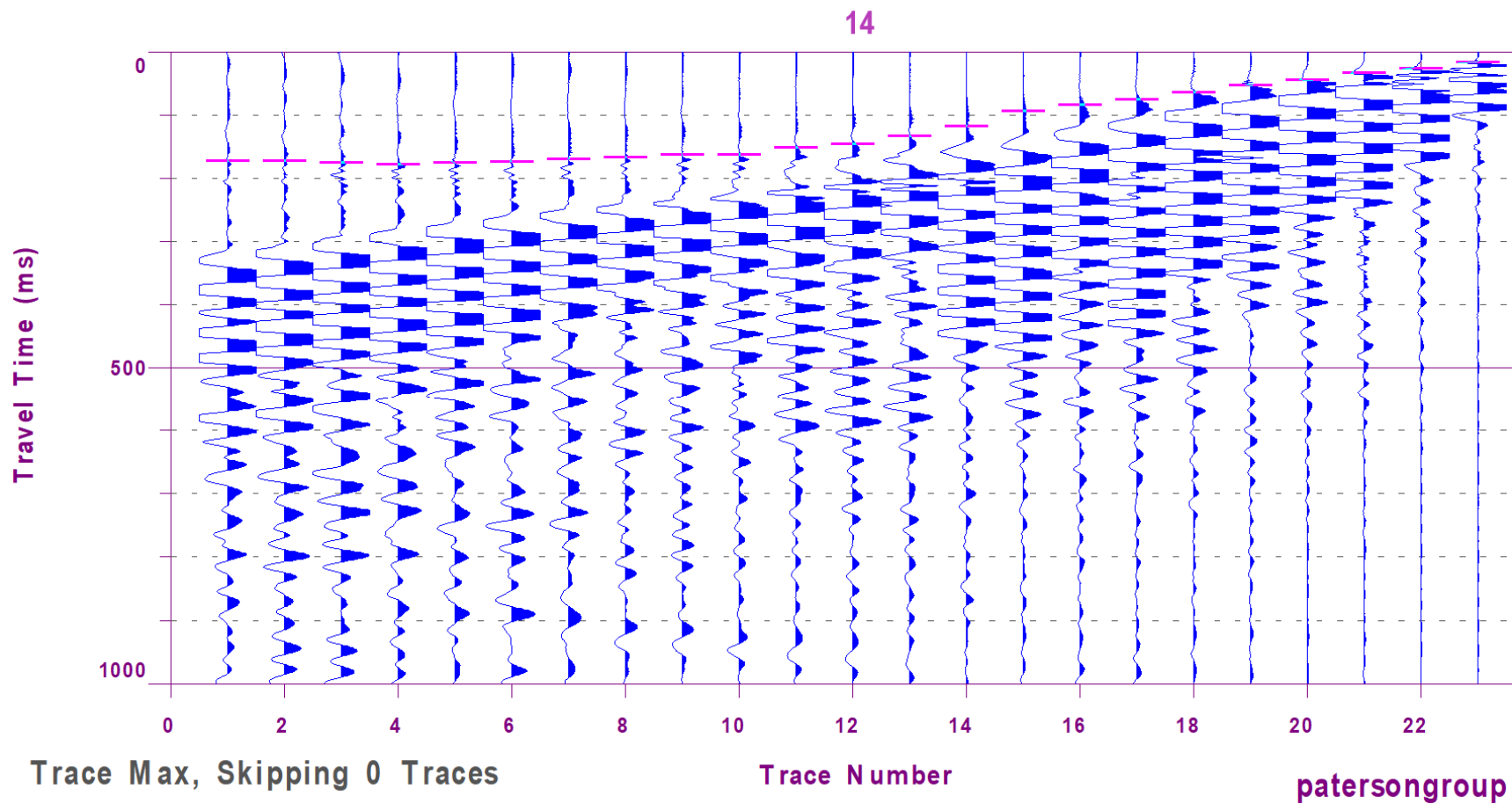
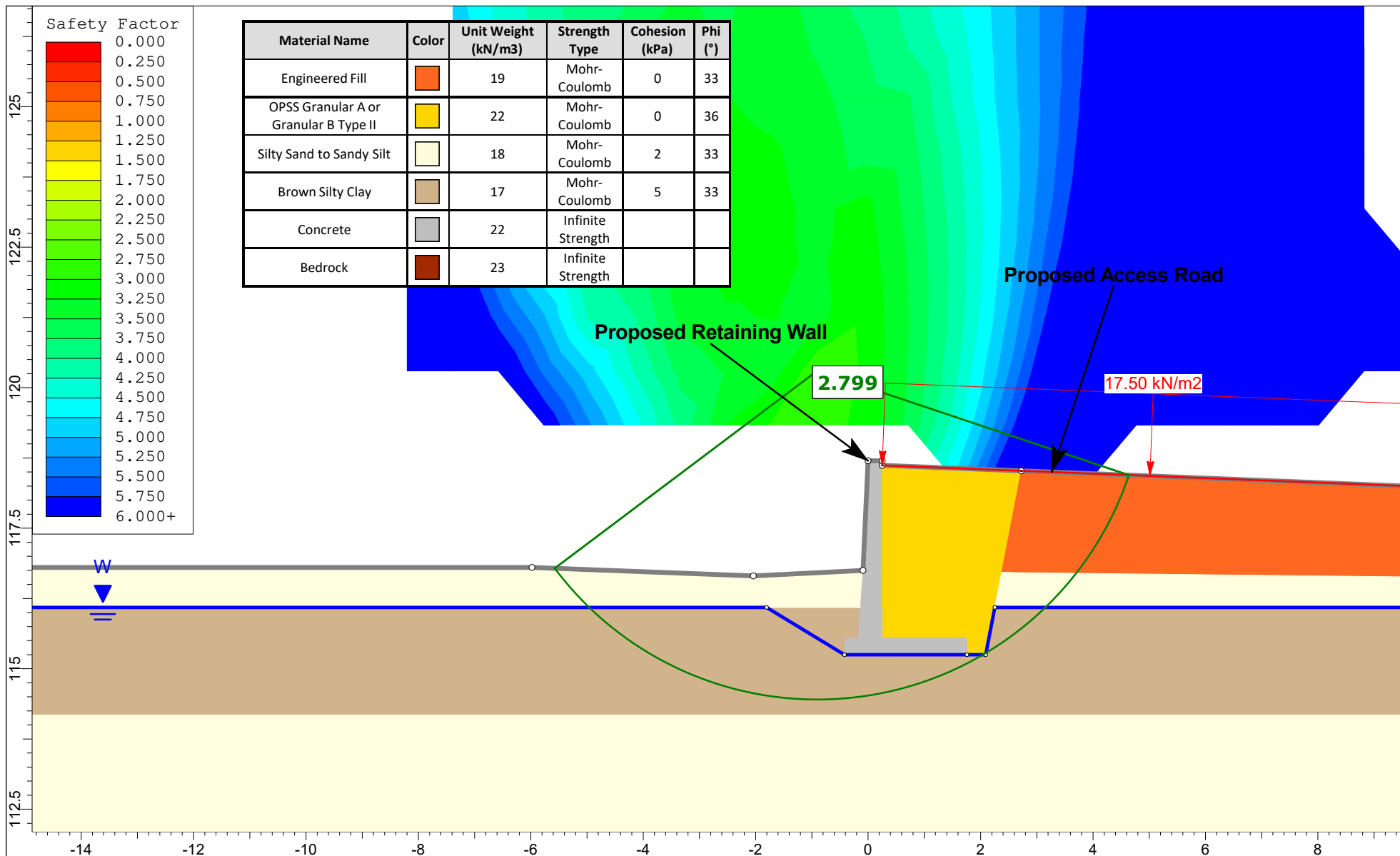
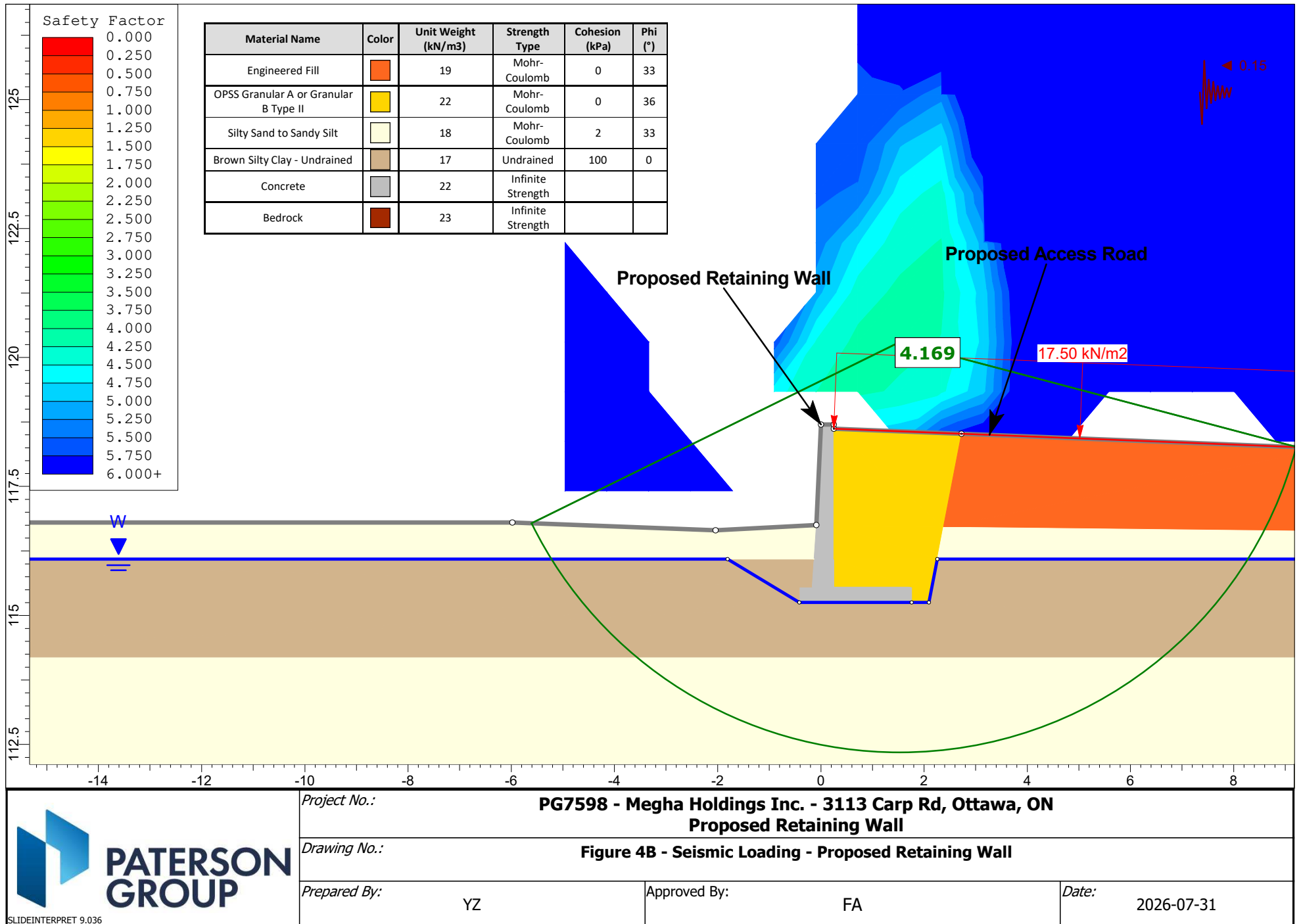
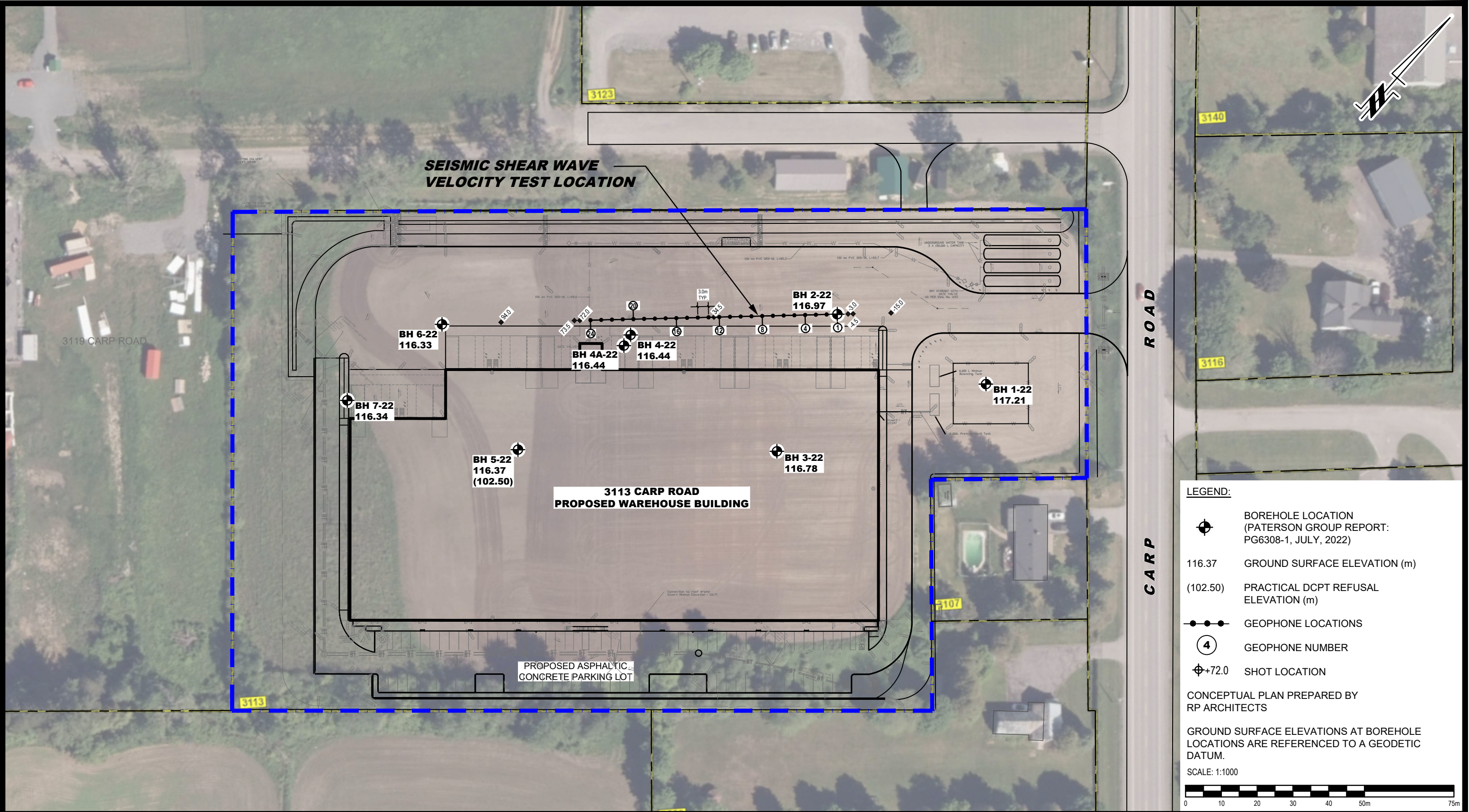


Figure 3 – Shear Wave Velocity Profile at Shot Location 72 m



	Project No.: PG7598 - Megha Holdings Inc. - 3113 Carp Rd, Ottawa, ON		
	Drawing No.: Figure 4A - Static Loading - Proposed Retaining Wall		
	Prepared By: YZ	Approved By: FA	Date: 2026-07-31





LEGEND:

- BOREHOLE LOCATION
(PATERSON GROUP REPORT: PG6308-1, JULY, 2022)
- 116.37 GROUND SURFACE ELEVATION (m)
- (102.50) PRACTICAL DCPT REFUSAL ELEVATION (m)
- GEOPHONE LOCATIONS
- GEOPHONE NUMBER
- SHOT LOCATION

CONCEPTUAL PLAN PREPARED BY
RP ARCHITECTS

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

SCALE: 1:1000

 9 AURIGA DRIVE OTTAWA, ON K2E 7T9 TEL: (613) 226-7381				MEGHA HOLDINGS INC. GEOTECHNICAL INVESTIGATION PROPOSED INDUSTRIAL BUILDING 3113 CARP ROAD ONTARIO				Scale: 1:1000	Date: 08/2026
				OTTAWA, Title:				Drawn by: YA	Report No.: PG7598-1
				TEST HOLE LOCATION PLAN				Checked by: YZ	Dwg. No.: PG7598-1
								Approved by: FA	Revision No.:
NO.	REVISIONS	DATE	INITIAL						