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Geotechnical Investigation

Proposed Residential Development
Arcadia - Stage 5
Campeau Drive - Ottawa

Prepared For

Minto Communities

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Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Minto Communities to conduct a geotechnical investigation for Stage 5 of the Arcadia residential development to be located at Campeau Drive, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

☐ determine the subsurface soil and groundwater conditions at this site by means of test holes.

☐ provide geotechnical recommendations for 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 conceptual drawings, it is understood that Stage 5 of the proposed development will consist of a series of single-family and townhouse style residential dwellings with basements or slab-on-grade construction. It is also understood that the proposed development will include attached garages, associated driveways, local roadways and landscaped areas. It is further anticipated that the proposed development will be serviced by future municipal water, sanitary and storm services.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current geotechnical investigation was conducted on January 13, 2021 and consisted of 5 boreholes advanced to a maximum depth of 6.7 m below the existing ground surface. Previous investigations were carried out at, or within the vicinity of the current stage of the development by this firm between 2005 and 2012. During that time, a total of 8 boreholes were advanced to a maximum depth of 19.1 m below existing grade within the immediate area of Stage 5 of the proposed development. The test holes were determined in the field by Paterson personnel and distributed in a manner to provide general coverage of the current phase of the residential development, taking into consideration site features and underground utilities. The test hole locations are presented on Drawing PG4933-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler, 73 mm diameter thin walled (TW) Shelby tubes in conjunction with a piston sampler or from the auger flights. All soil samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags and the Shelby tubes were sealed at both ends on site. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split-spoon and Shelby tube samples were recovered from the test holes are shown as AU, SS and TW, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Undrained shear strength testing was conducted at regular intervals in cohesive soils and completed using a field vane apparatus.

The overburden thickness was evaluated by dynamic cone penetration testing (DCPT) at boreholes BH 8, BH 9, BH 16 and BH 41. 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 at the test hole locations were recorded in detail in the field. Our findings are presented in the Soil Profile and Test Data sheets in Appendix 1.

Groundwater Monitoring

Flexible standpipes were installed in select boreholes during the historical investigations to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the current stage of the residential development taking into consideration existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson. The ground surface elevations at the borehole locations were referenced to a geodetic datum. It should be noted that all historical borehole locations and ground surface elevations were also referenced to a geodetic datum. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG4933-2 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were visually examined in our laboratory to review the results of the field logging. Moisture Content, Gradation and Atterberg Limits testing were also completed on select samples obtained from the geotechnical investigations. The results of this testing are provided in section 4.2.

A total of 6 Shelby tube samples collected from the boreholes during previous investigations were submitted for unidimensional consolidation testing. The results of the consolidation are presented in Table 2 - Summary of Consolidation Results and are further discussed in Section 5.

Soil samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

Four (4) soil samples from adjacent stages were submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples were submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.8.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and the original ground surface slopes downwards to the east. However, the majority of the original ground surface of Stage 5 has been covered with site excavated fill material collected from the previous stages of the same development between 2008 and present. In the summer of 2019, the fill piles were shaped within Stage 5, to the regulated flood line of the carp river

Arcadia Stage 5 is bordered to the north by the Carp River Municipal Drain, to the west by a proposed park block and the proposed Paine Pond stormwater management facility, to the east by future Campeau Drive and to the south by future Winterset Road and Arcadia Stage 3.

4.2 Subsurface Profile

Overburden

At the time of the original field investigation, the soil conditions encountered at the test hole locations consist of a topsoil followed a layer of stiff brown silty clay, which is underlain by a stiff to firm grey silty clay. The clay was inferred to be underlain by a glacial till or bedrock. Approximately 3 to 4 m of fill has been placed across Stage 5 of the subject site between 2005 and present. The fill material generally consists of brown silty clay mixed with sand, gravel, cobbles and boulders.

Practical refusal to augering/DCPT was encountered between 11.2 to 19.3 m below existing ground surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, local bedrock consists of interbedded limestone and shale of the Verulam formation as well sandstone of the Nepean Formation with an anticipated overburden thickness of 5 to 25 m.

Grain Size Distribution and Hydrometer Testing Results

The results of the 4 soil samples which were submitted for grain size analysis and hydrometer testing from our geotechnical investigations are summarized in Table 1.

Table 1 - Grain Size Distribution

Test Hole	Sample	Gravel (%)	Sand (%)	Silt and Clay (%)
BH 2-21	SS3	0.0	4.2	95.8
BH 3-21	SS8	0.0	1.5	98.5
BH 5-21	SS3	0.0	7.6	92.4
BH 16	SS7	0.0	0.3	99.7

Atterberg Limit Testing Results

A total of 11 silty clay samples were submitted for Atterberg Limits testing during the course of the investigations. The results are summarized in Table 2 below and on the Atterberg Limits results sheets in Appendix 1.

Table 2 - Summary of Atterberg Limits Tests

Borehole No.	Sample	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Classification
BH 1-21	SS3	28	45	22	23	CL
BH 2-21	SS3	27	36	21	15	CL
BH 3-21	SS8	36	44	23	21	CL
BH 4-21	SS6	32	46	21	25	CL
BH 5-21	SS3	34	49	23	26	CL
BH 16	SS5	-	45	24	21	CL
BH 16	SS6	-	41	20	22	CL
BH 16	SS7	-	34	20	15	CL
BH 17	TW3	67	50	23	27	CL
BH 17	SS5	-	33	19	13	CL
BH 17	SS7	-	35	20	15	CL

Note: CL: Inorganic Clay of Low Plasticity

Shrinkage Testing Results

The results of the shrinkage testing of BH 5-21 - SS3 resulted in a shrinkage limit of **17%** with a shrinkage ratio of **1.84**.

4.3 Groundwater

Groundwater level readings were recorded from February 21, 2005 to April 25, 2012 at the boreholes within Stage 5 during the previous geotechnical investigations. The groundwater level readings are presented in Table 3 below and on the Soil Profile and Test Data sheets in Appendix 1.

Table 3 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 5-12	99.15	1.43	97.72	April 25, 2012
BH 8	93.62	3.10	90.52	February 21, 2005
BH 9	92.63	1.30	93.93	February 21, 2005
BH 16	93.01	Flooded GS	93.01	April 11, 2006
BH 17	92.67	0.52	92.15	February 16, 2006
BH 24	92.64	-1.20	93.84	June 18, 2006
BH 25	92.55	-0.35	92.90	June 18, 2006
BH 41	93.43	-0.21	93.64	July 4, 2007

Note: Groundwater levels are referenced to the borehole ground surface elevations as provided by Webster and Simmonds Surveying Limited.

It is important to note that groundwater level readings within piezometers could be influenced by surface water infiltrating the backfilled borehole, which can lead to higher water levels than noted during the investigation. The long-term groundwater level can also be estimated based on moisture levels, consistency and colouring of the recovered soil samples. Therefore, based on these observations, the long-term groundwater table can be estimated between an elevation of 90 and 91 m within the subject site. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore could vary during time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

It is anticipated that the proposed buildings will be supported by shallow footings placed over an undisturbed stiff to firm brown silty clay bearing surface or an engineered fill placed over an undisturbed stiff to firm silty clay bearing surface.

Due to the presence of the sensitive silty clay layer, the proposed development will be subjected to grade raise restrictions.

For areas where proposed grades exceed our permissible grade raise recommendations, a settlement surcharge program will be a valid option to induce settlement within the subject site until adequate settlement rates are observed. Alternatively, the use of Lightweight Fill with varying thicknesses and extents will be specified by Paterson based on final grading plans for the lots/block where permissible grade raise exceedances occur and a surcharge program has not been completed.

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

5.2 Site Grading and Preparation

Stripping Depth

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

It is understood that fill material has been accumulated within the subject site as a result of the site excavations of the previous stages. The fill was not placed in an engineered fashion and will require sub-excavation and reinstatement prior to placement of site services or the proposed dwellings. As such, if the fill material is deemed acceptable, it should be removed and reinstated from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the proposed building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be placed in lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD).

If blast rock is used to as fill to build up the bearing medium below housing areas, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm placed in maximum 600 mm loose lifts and compacted using a large smooth drum vibratory roller making several passes per lift and approved by the Paterson Group at the time of placement. Any blast rock greater than 300 mm in diameter should be segregated and hoe rammed into acceptable fragments. The blast rock fill with maximum particle size of 300 mm should be capped with a minimum of 300 mm of Granular B Type II or Granular A crushed stone material should be compacted to at least 98% of its SPMDD.

In areas where fill is required to build up the bearing medium within areas identified with permissible grade raise restrictions on Drawing PG4933-2 - Permissible Grade Raise Plan, consideration should be taken to utilizing a lighter silty clay material to reduce the overall weight on the underlying soils. The silty clay should consist of a relatively dry, unfrozen, workable brown silty clay, free of organic containing materials and approved by Paterson at the time of construction. The workable silty clay should be placed in maximum 300 mm thick loose lifts and compacted by a sheepsfoot roller making several passes under dry, unfrozen conditions and periodically inspected and approved by Paterson field personnel. It is further recommended that the engineer clay fill be capped with a minimum of 300 mm of Granular B Type II or Granular A crushed stone material should be compacted to at least 98% of its 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. The existing fill materials should be spread in maximum 300 mm thick lifts and compacted using a suitable roller making several passes to minimize voids. Non-specified existing fill and site-excavated soil are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

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 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 paves areas should be compacted to at least 95% of its SPMDD.

If blast rock is to be used as fill to build up the subgrade for roadways, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. Where the fill is open-graded, a binding layer of finer granular fill or a geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements.

Protection of Subgrade and Bearing Surfaces

It is expected that site grading and preparation will consist of stripping of the soils containing significant amounts of organic materials and previous surcharge material above design underside of footing elevation. The contractor should take appropriate precautions to avoid disturbing the subgrade and bearing surfaces from construction and worker traffic. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional fill.

5.3 Foundation Design

Bearing Resistance Values

Using continuously applied loads, footings for the proposed buildings can be designed using the bearing resistance values presented in Table 4.

Table 4 - Bearing Resistance Values

Bearing Surface	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Very Stiff to Stiff Silty Clay	150	225
Firm Grey Silty Clay	75	150
Engineered Silty Clay Fill	100	150
Engineered Fill over Silty Clay Crust	150	225

Note: Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed over a silty clay bearing surface can be designed using the above noted bearing resistance values.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

In the case of the engineered silty clay fill, preparation of a suitable bearing surface may be difficult and it may be necessary to “cap” the fill with 200 to 300 mm thick layer of Granular A crushed stone compacted to 98% of SPMDD below design underside of footing elevation. This requirement should be evaluated by Paterson on a lot-by-lot basis during the construction phase. Bearing resistance values for footing design should also be determined on a per lot basis at the time of construction.

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 the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Settlement/Grade Raise

During the course of investigations completed within the confines of Stage 5, a total of 6 silty clay samples collected at this site during our investigations were subjected to unidimensional consolidation testing. The results of the testing are presented in Table 5 and Appendix 1.

The value for p'_c is the preconsolidation pressure and p'_o is the effective overburden pressure of the test sample. The difference between these values is the available preconsolidation. The increase in stress on the soil due to the cumulative effects of the fill surcharge, the footing pressures, the slab loadings and the lowering of the groundwater should not exceed the available preconsolidation if unacceptable settlements are to be avoided.

The values for C_{cr} and C_c are the recompression and compression indices, respectively. These soil parameters are a measure of the compressibility due to stress increases below and above the preconsolidation pressures. The higher values for the C_c, as compared to the C_{cr}, illustrate the increased settlement potential above, as compared to below, the preconsolidation pressure.

Table 5 - Summary of Consolidation Test Results

Borehole No.	Sample	Depth (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c	Q (*)
BH 17	TW 3	3.40	72	29	0.046	1.23	A
BH 24	TW 3	4.39	92	35	0.017	1.73	A
BH 25	TW 2	4.90	115	38	0.018	1.24	A
BH 25	TW 4H	8.08	95	35	0.015	2.88	A
BH 25	TW 4	7.98	154	55	0.011	1.18	A
BH 25	TW 6	11.10	149	73	0.023	2.06	F - P**

* - Q - Quality assessment of sample - G: Good F: Fair A: Acceptable P: Likely disturbed

The values of p'_c, p'_o, C_{cr} and C_c are determined using standard engineering testing procedures and are estimates only. Natural variations within the soil deposit will affect the results. The p'_o parameter is directly influenced by the groundwater level. Groundwater levels were measured during the site investigation. Groundwater levels vary seasonally which has an impact on the available preconsolidation. Lowering the groundwater level increases the p'_o and therefore reduces the available preconsolidation. Unacceptable settlements could be induced by a significant lowering of the groundwater level. The p'_o values for the consolidation tests carried out for the present investigation are based on the long term groundwater level observed at each borehole location. The groundwater level is based on the colour and undrained shear strength profile of the silty clay.

The total and differential settlements will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed.

The potential post construction total and differential settlements are dependent on the position of the long term groundwater level when buildings are situated over deposits of compressible silty clay. Efforts can be made to reduce the impacts of the proposed development on the long term groundwater level by placing clay dykes in the service trenches, reducing the sizes of paved areas, leaving green spaces to allow for groundwater recharge or limiting planting of trees to areas away from the buildings. However, it is not economically possible to control the groundwater level.

To reduce potential long term liabilities, consideration should be given to accounting for a larger groundwater lowering and to provide means to reduce long term groundwater lowering (e.g. clay dykes, restriction on planting around the dwellings, etc). Buildings on silty clay deposits increases the likelihood of movements and therefore of cracking. The use of steel reinforcement in foundations placed at key structural locations will tend to reduce foundation cracking compared to unreinforced foundations.

Based on the undrained shear strength testing results, consolidation testing and experience with the local silty clay deposit, permissible grade raise areas have been determined and presented on Drawing PG4933-2 - Permissible Grade Raise Plan for Housing in Appendix 2. An additional 0.5 m should be used for the permissible grade raise restrictions for roadways within the same designated grade raise restriction areas.

Where proposed grade raises exceed our permissible grade raise recommendations, several options could be considered for the foundation support of the proposed Buildings:

Scenario A

Where the grade raise is close to, but below, the maximum permissible grade raise, consideration should be given to using more reinforcement in the design of the foundation (footings and walls) to reduce the risks of cracking in the concrete foundation. The use of control joints within the brick work between the garage and basement area should also be considered.

Scenario B

Where the grade raise cannot be accommodated with soil fill, the following options could be used alone or in combination.

Option 1 - Use of Lightweight Fill

Lightweight fill (LWF) can be used, consisting of EPS (expanded polystyrene) Type 15 or 19 blocks or other light weight materials which allow for raising the grade without adding a significant load to the underlying soils. However, these materials are expensive and, in the case of the EPS, are more difficult to use under the groundwater level, as they are buoyant, and must be protected against potential hydrocarbon spills. Use lightweight fill within the interior of the garage and porch areas to reduce the fill-related loads.

Option 2 - Preloading or Surcharging

It is possible to preload or surcharge the proposed site in localized areas provided sufficient time is available to achieve the desired settlements based on theoretical values from the settlement analysis. If this option is considered, a monitoring program using settlement plates will have to be implemented. This program will determine the amount of settlement in the preloaded or surcharged areas. Obviously, preloading to proposed finished grades will allow for consolidation of the underlying clays over a longer time period. Surcharging the site with additional fill above the proposed finished grade will add additional load to the underlying clays accelerating the consolidation process and allowing for accelerated settlements. Once the desired settlements are achieved, the site can be unloaded and the fill can be used elsewhere on site.

Once the required grade raises are established, the above options could be further discussed along with further recommendations on specific requirements.

Underground Utilities

The underground services may be subjected to unacceptable total or differential settlements. In particular, the joints at the interface building/soil may be subjected to excessive stress if the differential settlements between the building and the services are excessive. This should be considered in the design of the underground services.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Site Class D** for the shallow foundations considered at this site. The soils underlying the proposed foundations are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab/Slab on Grade Construction

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the existing stiff to very stiff brown silty clay will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction.

Where existing fill, free of deleterious material and significant organic content, is encountered below the floor slab, provisions should be made to removing the existing fill from within the building footprint and replacing the fill with OPSS Granular A or Granular B Type II compacted to a minimum 98% of the material's SPMDD.

It is also acceptable to use workable, site excavated brown silty clay material, free of deleterious materials and organics, below the floor slab and outside the lateral support zone of the proposed footings provided the material is reviewed and approved by Paterson prior to placement. If the silty clay is to be used as backfill material, it is critical that the material be placed under dry conditions and above freezing temperatures and be compacted using a sheepsfoot roller making several passes under the full supervision of Paterson field personnel.

Any soft or poor performing areas should be removed and backfilled with OPSS Granular B Type II and compacted to 98% of the material's SPMDD.

It is recommended that the upper 200 mm of sub-floor fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings (but outside the zones of influence of the footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD. Within the zones of influence of the footings, the backfill material should be compacted to a minimum of 98% of its SPMDD.

5.6 Pavement Structure

Car only parking areas, local and collector roadways are anticipated at this site. The proposed pavement structures are shown in Tables 6, 7 and 8.

Table 6 - 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 fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 7 - Recommended Pavement Structure - Local Residential Roadways

Thickness (mm)	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil	

Table 8 - Recommended Pavement Structure - Roadways with Bus Traffic

Thickness mm	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** - Superpave 19.0 Asphaltic Concrete
50	**Lower Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
600	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil or OPSS Granular B Type II material placed over in situ soil	

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. Weak subgrade conditions may be experienced over service trench fill materials, which will require the use of a woven geotextile liner, such as Terratrack 200 or equivalent, as well as, an additional 300 to 600 mm thick granular layer, consisting of a 150 mm minus, well graded granular fill or crushed concrete, to provide adequate construction access.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment. Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

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.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter foundation drainage system is recommended for proposed structures. The system should consist of a 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, 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 pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless placed in conjunction with a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system.

6.2 Protection Against Frost Action

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

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

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 expected that sufficient room will be available for 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.

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.

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the City of Ottawa.

It is expected that the invert level of the municipal services will be installed at or below the long term groundwater level within the native silty clay deposit. Due to the low permeability of the silty clay deposit, it is expected that minimal groundwater infiltration will occur during installation work. It is expected that groundwater infiltration will be handled by suitably sized submersible pumps. Groundwater infiltration is not expected provided that best construction practices are followed for the sewer pipe installation work and that the sewers are installed as per design requirements.

The pipe bedding for sewer and water pipes placed on a relatively dry, undisturbed subgrade surface should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the firm 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 95% 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 and silty sand materials will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

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.

Clay Seals

To reduce long-term lowering of the groundwater at this site, clay seals should be provided within the service trenches excavated through the silty clay deposit. The seals should be at least 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. The seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches excavated through the silty clay deposit.

6.5 Groundwater Control

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

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 (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. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

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 use of straw, propane heaters, 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.

The trench excavations should be constructed in a manner that will avoid the introduction of frozen materials 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 as the work takes place. In addition, 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. Additional information could be provided, if required.

6.7 Landscaping Considerations

Tree Planting Setbacks

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. Grain size distribution and Sieve analysis testing was also completed on selected soil samples. The above noted test results were completed between design 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 4.1 and in Appendix 1.

Based on the results of the representative soil samples, the current stage of the subject site is considered as a low/medium sensitivity area for tree planting according to the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines).

Since the modified plasticity limit (PI) does not exceed 40%, large trees (mature height over 14 m) can be planted at the subject site 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).

According to the City of Ottawa Tree Planting Guidelines, tree planting setback limits may be reduced to 4.5 m 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) extends to 2.1 m or greater below the lowest finished grade 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. **However, due to the thickness of the fill material within the subject site, this condition is not required as the native silty clay material is well below the proposed underside of footing elevations (at least 3 m below proposed USF levels).**

- ☐ A small tree must be provided with a minimum of 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, or a volume that is appropriate to the species selected, 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 surround the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree).

In-Ground Swimming Pools

The in-situ soils are considered to be acceptable for the installation of in-ground swimming pools. The soil removed to accommodate an in-ground swimming pool weighs more than the water filled in-ground pool. Therefore, no additional load is being applied to the underlying sensitive clays.

Aboveground Swimming Pools, Hot Tubs and Exterior Decks

If consideration is given to construction of an above ground swimming pool, a hot tub or an exterior deck, a geotechnical consultant should be retained by the homeowner to review the site conditions. No additional grading should be placed around the exterior structure. The swimming pool should be located at least 3 m away from the existing foundation to avoid adding localized loading to the foundation and the hot tub should be located at least 2 m away from the existing foundation. Otherwise, construction is considered routine, and can be constructed in accordance with the manufacturer's specifications.

6.8 Corrosion Potential and Sulphate

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

6.9 Slope Stability Assessment and Limit of Hazard Lands Setbacks

A slope stability assessment has been conducted to determine the geotechnical slope stability for the proposed conditions at the subject site, given that the Carp River runs in close proximity to the north boundary line of Stage 5 of the subject site.

The Carp River runs in a northwest to southeast direction near the north-east boundary of the site, there is an existing asphalt paved walking path along the south portion of the river, located between the proposed development and the river. Paterson observed the condition of the river’s side slopes in the field. The side slopes of the river are relatively flat and vegetated with grass and cattails. No active erosion was observed at the time of our site visit.

It should be noted that the current slope along the north portion of the subject site was constructed under the supervision of Paterson personnel as part of an ongoing settlement surcharge program. The slope will remain in place post completion of the settlement program, however, approximately 2 m will be removed from the top of the surcharge pile and slope to accommodate the proposed grading of the phase.

Two (2) slope cross-section (Section A and B) were analyzed as the worst case scenarios of the proposed conditions under static and seismic conditions. It should be noted that assumptions were made for finished grades based on the current proposed grading provided by the site’s civil consultant. Actual finished grades planned for the proposed development were not available at the time of preparation of this report. The cross-section location is presented on Drawing PG4933-1 - Test Hole Location Plan, which is included in Appendix 2.

Slope Stability Analysis

The slope stability analysis for the proposed conditions was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods, including the Bishop’s simplified method which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure.

Theoretically, a factor of safety of 1.0 represents a condition where the slope is marginally 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.

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 9 below.

Table 9 - Effective Strength Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Engineered Clay Fill	18	33	-
Silty Sand	19	35	-
Brown Silty Clay Crust	17	33	5
Grey Silty Clay	16	33	10
Bedrock	23	-	-

The total strength parameters for seismic analysis were chosen based on the subsurface conditions encountered within the completed at the time of our geotechnical investigation, and based on our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 10 below:

Table 10 - Total Strength Soil and Material Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Engineered Clay Fill	18	33	-
Silty Sand	19	35	-
Brown Silty Clay Crust	17	-	80
Grey Silty Clay	16	-	60
Bedrock	23	-	-

Static Loading (Effective Strength) Analysis

A minimum factor of safety of 1.5 is generally recommended for static conditions where the failure of the slope would endanger permanent structures. The slope stability analysis for static conditions was completed at the slope cross-sections under a conservative scenario by assigning cohesive soils which are fully saturated.

The results of the static analysis at Sections A and B are shown on the attached Figure 2 and 4, respectively, in Appendix 2. The results indicate that the factor of safety exceeds 1.5, and is considered acceptable from a geotechnical perspective.

Seismic Loading (Total Stress) Analysis

An analysis considering seismic loading for the proposed site conditions was also completed for Sections A and B. A horizontal seismic coefficient of 0.16 g was considered for the slope. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the seismic analysis for Section A and B are shown on Figure 3 and 5, respectively, in Appendix 2. The results indicate that the factor of safety exceeds 1.1 and is considered acceptable, from a geotechnical perspective.

Based on the above, a stable slope setback is not required from a geotechnical perspective.

Toe erosion allowance and Erosion Access Allowance

It should be noted that due to the relatively flat nature of the side slopes of the river and observed river flood plain area (greater than 50 m), a toe erosion allowance setback limit is not required from a geotechnical perspective. Further, due to the proposed layout of the development along the north property line, a sufficient space is available for future maintanance of the subject slope. Therefore, the 6 m erosion access allowance is not required from a geotechnical perspective.

Limit of Hazard Lands Setback

Based on the above analysis, the subject site will not require any limit of hazard lands setbacks and is considered acceptable from a slope stability and geotechnical perspectives.

7.0 Recommendations

It is recommended that the following be completed once the master plan and site development are determined:

☐ Review detailed grading plan(s) from a geotechnical perspective.

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ 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 placing backfilling materials.

☐ Field density tests to ensure that the specified level of compaction has been 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 Paterson's recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with Paterson's present understanding of the project. Paterson requests permission to review the grading plan once available. Paterson's recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole log are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

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 to be notified immediately in order to permit reassessment of the recommendations.

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 Minto Communities or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

NPatey

Nicole R.L. Patey, B.Eng.

LICENSED PROFESSIONAL ENGINEER
Nov. 25-2021
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❑ Minto Communities (3 copies)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS TESTING RESULTS

GRAIN SIZE DISTRIBUTION SHEETS

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Stage 5 - Campeau Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

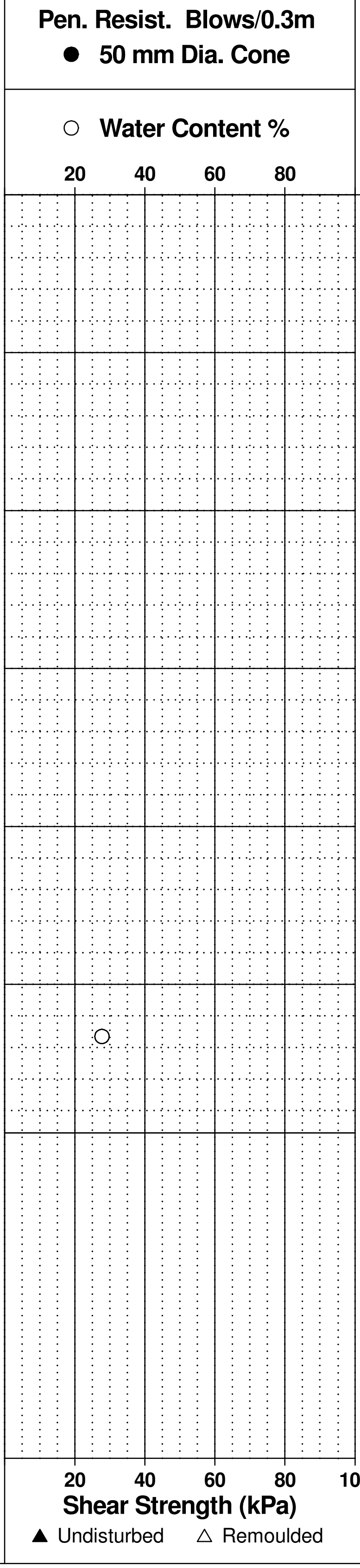
BORINGS BY CME 55 Power Auger

DATE 2021 January 13

FILE NO. PG4933

HOLE NO. BH 1-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	97.52	
FILL: Brown silty clay, trace sand, cobbles and boulders		AU	1					
						1	96.52	
						2	95.52	
						3	94.52	
						4	93.52	
4.57								
FILL: Grey silty clay, trace sand, gravel, cobbles and boulders		SS	2	8	9			
						5	92.52	
5.18								
Brown **SILTY CLAY** trace sand		SS	3	67	6			
5.94								
End of Borehole (GWL @ 5.18 m depth based on site observations)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Stage 5 - Campeau Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

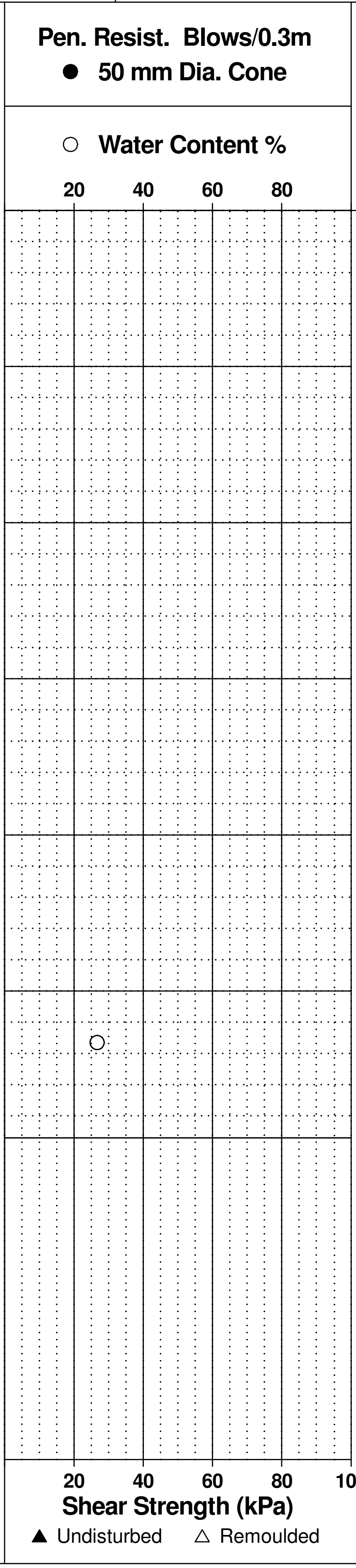
BORINGS BY CME 55 Power Auger

DATE 2021 January 13

FILE NO. PG4933

HOLE NO. BH 2-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.02
FILL: Brown silty clay trace organics, sand, gravel, cobbles and boulders		AU	1				
						1	96.02
						2	95.02
						3	94.02
						4	93.02
4.67							
Grey **SILTY CLAY** trace sand		SS	2	58	9		
						5	92.02
		SS	3	100	3		
5.94							
End of Borehole (GWL @ 5.18 m depth based on site observations)							



Piezometer Construction

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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

DATUM Geodetic

REMARKS

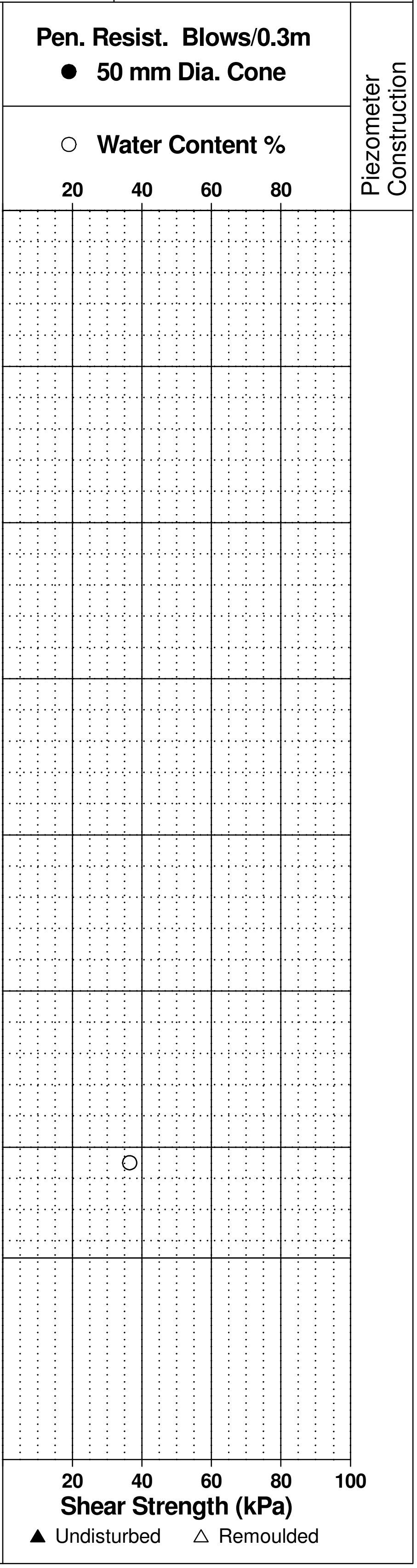
BORINGS BY CME 55 Power Auger

DATE 2021 January 13

FILE NO. PG4933

HOLE NO. BH 3-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.82
FILL: Brown silty clay trace sand, gravel, cobbles and boulders		AU	1				
						1	96.82
		SS	2	25	8		
						2	95.82
		SS	3	25	4		
						3	94.82
		SS	4	50	6		
						4	93.82
		SS	5	42	6		
		SS	6	58	10		
						5	92.82
		SS	7	33	7		
						6	91.82
6.20							
Grey **SILTY CLAY**		SS	8	100	4		
6.71							
End of Borehole (GWL @ 6.2 m depth based on site observations)							



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Ottawa, Ontario

DATUM Geodetic

REMARKS

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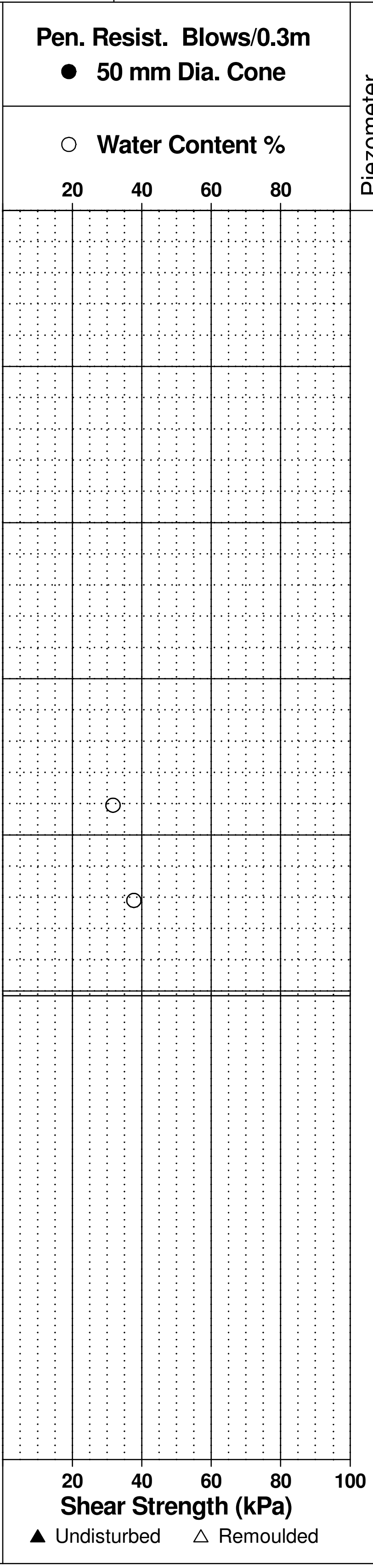
DATE 2021 January 13

FILE NO. PG4933

HOLE NO. BH 4-21

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
FILL: Brown silty clay trace sand, gravel, cobblesd and boulders		AU	1			0	100.34
		SS	2	33	6	1	99.34
		SS	3	17	4		
						2	98.34
		SS	4	25	6		
						3	97.34
		SS	5	71	9		
3.81							
Brown **SILTY CLAY**		SS	6	75	5	4	96.34
4.42							
Grey **SILTY CLAY**		SS	7	100	3		
5.03						5	95.34
End of Borehole (GWL @ 4.42 m depth based on site observations)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Stage 5 - Campeau Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

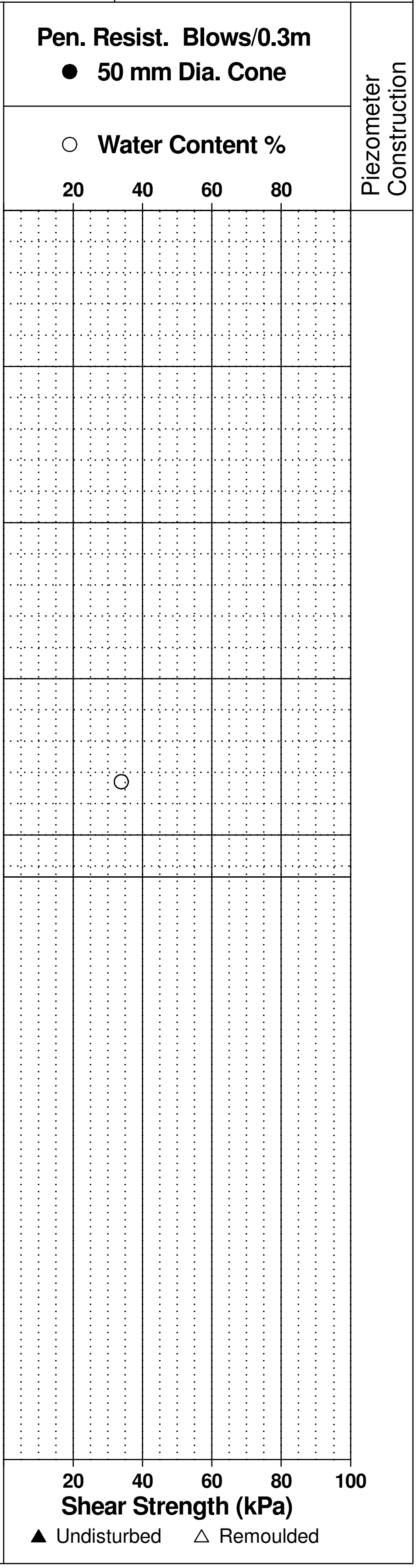
BORINGS BY CME 55 Power Auger

DATE 2021 January 13

FILE NO. PG4933

HOLE NO. BH 5-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	95.84
FILL: Brown silty clay trace sand, gravel, cobbles and boulders		AU	1				
						1	94.84
						2	93.84
						3	92.84
		SS	2	42	5		
3.66							
Grey **SILTY CLAY**		SS	3	100	5		
						4	91.84
4.27							
End of Borehole (GWL @ 3.66 m depth based on site observations)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS

BORINGS BY CME 75 Power Auger

DATE Feb 9, 05

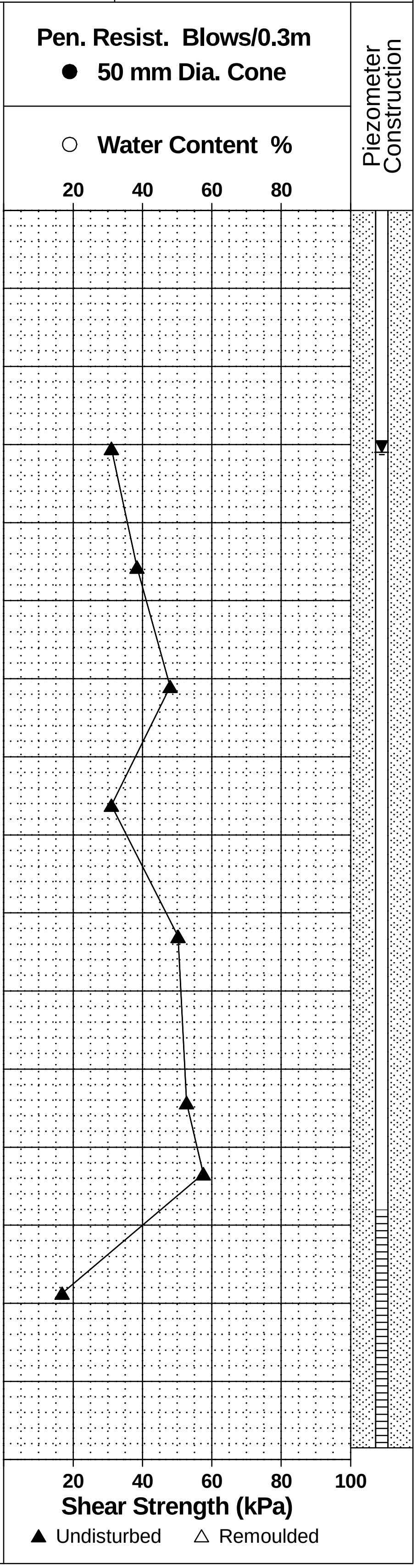
FILE NO. PG0538

HOLE NO. BH 8

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOIL	0.28
Stiff, brown **SILTY CLAY**, some sand	1.68
Stiff to firm, grey **SILTY CLAY**, trace fine sand - stiff by 8.4m depth	15.85

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	8	3
SS	2	58	1
TW	3	92	
SS	4	100	P
TW	5	100	
SS	6	92	
SS	7		

DEPTH (m)	ELEV. (m)
0	93.62
1	92.62
2	91.62
3	90.62
4	89.62
5	88.62
6	87.62
7	86.62
8	85.62
9	84.62
10	83.62
11	82.62
12	81.62
13	80.62
14	79.62
15	78.62
16	77.62



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS

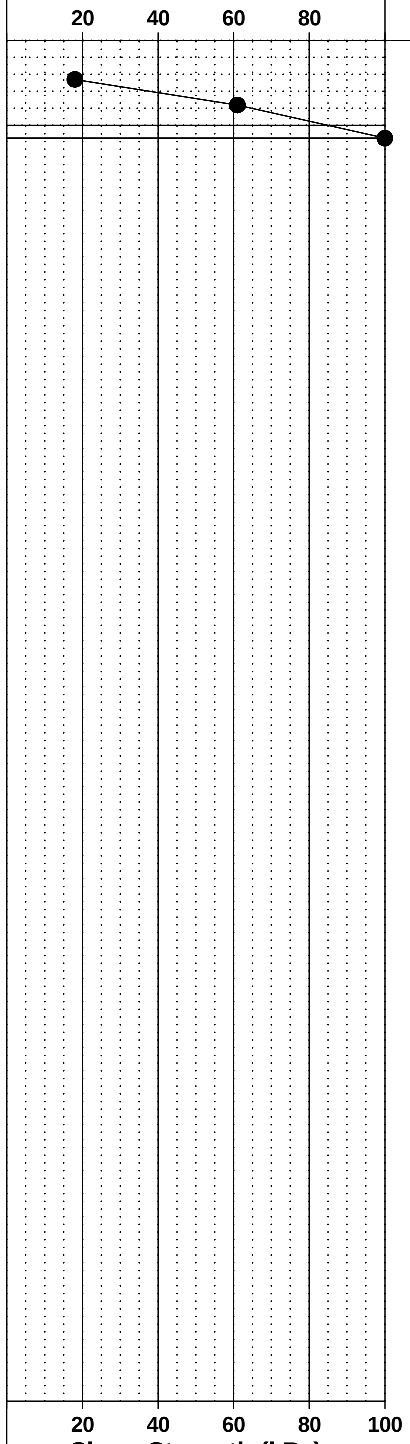
BORINGS BY CME 75 Power Auger

DATE Feb 9, 05

FILE NO. PG0538

HOLE NO. BH 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
Dynamic Cone Penetration Test commenced @ 15.85m depth. Cone pushed to 16.5m depth						16	77.62		
16.50 Inferred **SILTY CLAY**									
17.15 Inferred **GLACIAL TILL**						17	76.62		
End of Borehole DCPT refusal @ 17.15m depth (GWL @ 3.10m-Feb. 21/05)									



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS

BORINGS BY CME 75 Power Auger

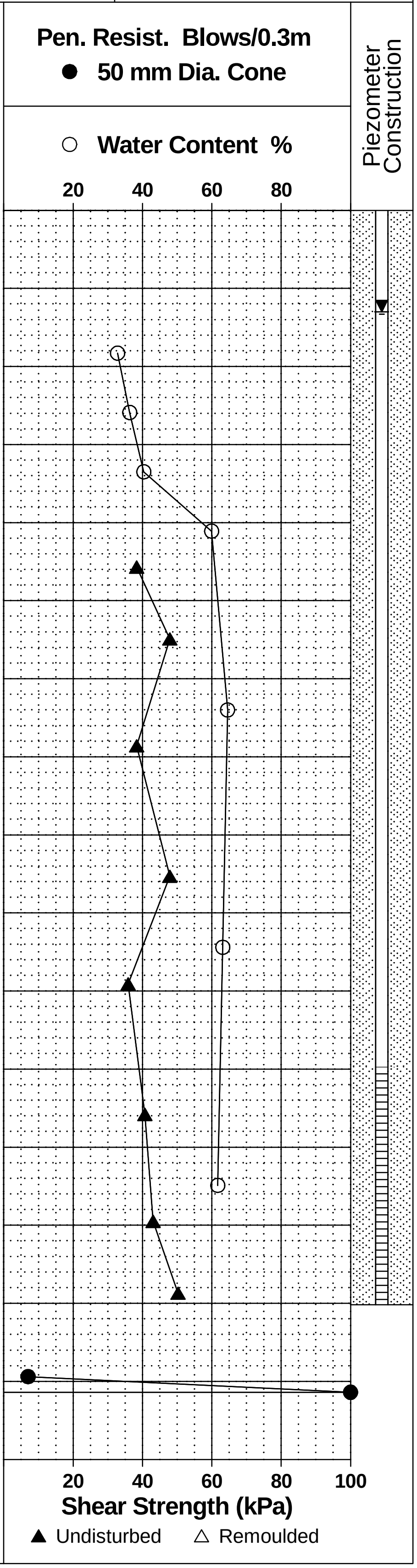
DATE Feb 11, 05

FILE NO. PG0538

HOLE NO. BH 9

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Stiff to firm, brown **SILTY CLAY**, some sand seams	SS	1	79	2
	SS	2	88	P
	SS	3	100	P
- grey by 4.0m depth	SS	4	96	P
	SS	5	100	P
	SS	6	100	P
	SS	7	100	P
14.02				
Dynamic cone Penetration Test commenced @ 14.02m depth. 15.00				
Inferred **SILTY CLAY** 15.14				
Inferred **GLACIAL TILL**				

DEPTH (m)	ELEV. (m)
0	92.63
1	91.63
2	90.63
3	89.63
4	88.63
5	87.63
6	86.63
7	85.63
8	84.63
9	83.63
10	82.63
11	81.63
12	80.63
13	79.63
14	78.63
15	77.63



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS

BORINGS BY CME 75 Power Auger

DATE Feb 11, 05

FILE NO. PG0538

HOLE NO. BH 9

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				
End of Borehole DCPT refusal @ 15.14m depth (GWL @ 1.30m-Feb. 21/05)									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS

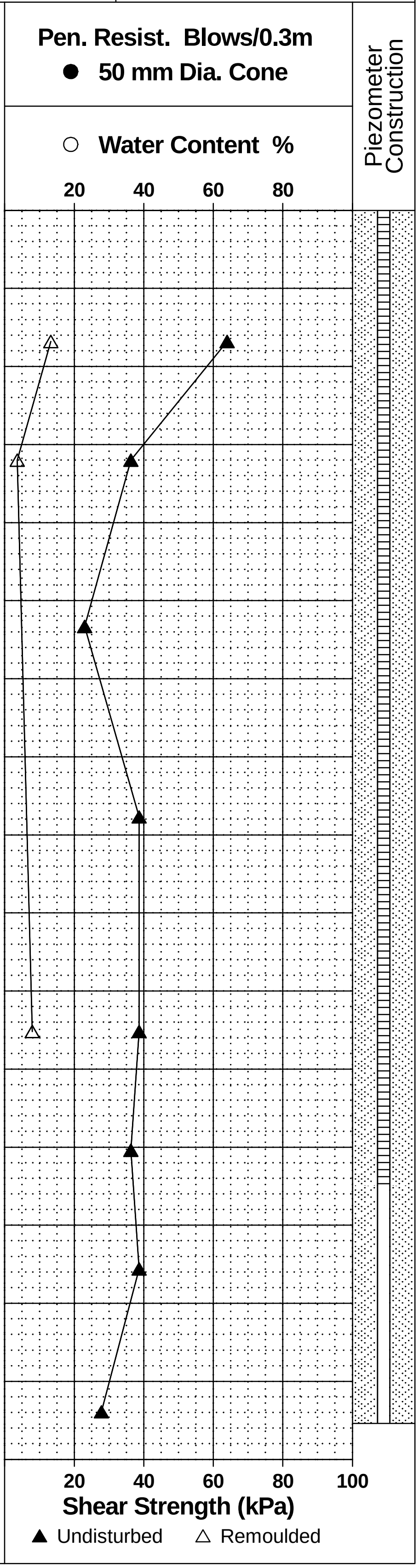
BORINGS BY CME 45 Power Auger

DATE Feb 16, 06

FILE NO. PG0538

HOLE NO. BH16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.01
TOPSOIL 0.36							
Stiff, brown **CLAYEY SILT**, some sand seams 0.80		SS	1	4	1	1	92.01
						2	91.01
Stiff to firm, grey **SILTY CLAY**		SS	2	100	1	3	90.01
- soft by 4.0m depth		TW	3	47		4	89.01
		TW	4	100		5	88.01
						6	87.01
- firm by 6.6m depth		SS	5	100	1	7	86.01
						8	85.01
						9	84.01
						10	83.01
		SS	6	100	1	11	82.01
						12	81.01
		SS	7	33	1	13	80.01
						14	79.01
- soft by 14.5m depth						15	78.01
15.54							
Dynamic Cone Penetration						16	77.01



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154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS

BORINGS BY CME 45 Power Auger

DATE Feb 16, 06

FILE NO. PG0538

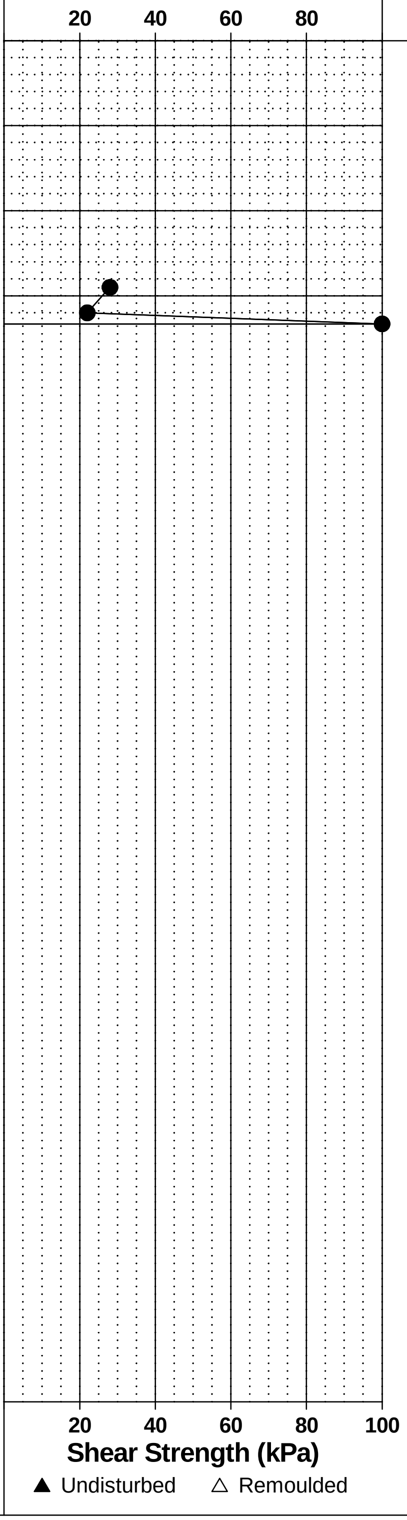
HOLE NO. BH16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Test commenced @ 15.54m depth. Cone pushed to 18.6m depth Inferred **SILTY CLAY** 18.60	▨					16 17 18	77.01 76.01 75.01
Inferred **GLACIAL TILL** 19.33	^^^					19	74.01
End of Borehole DCPT refusal @ 19.33m depth (Ground surface flooded - Apr. 11/06)							

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

○ **Water Content %**

Piezometer Construction



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE Oct 17, 05

FILE NO. PG0538

HOLE NO. BH17

SOIL DESCRIPTION	Depth
GROUND SURFACE	
TOPSOIL	0.08
Stiff, brown **CLAYEY SILT**, some sand	1.20
Stiff, brown **SILTY CLAY** - firm and grey by 1.8m depth	13.08

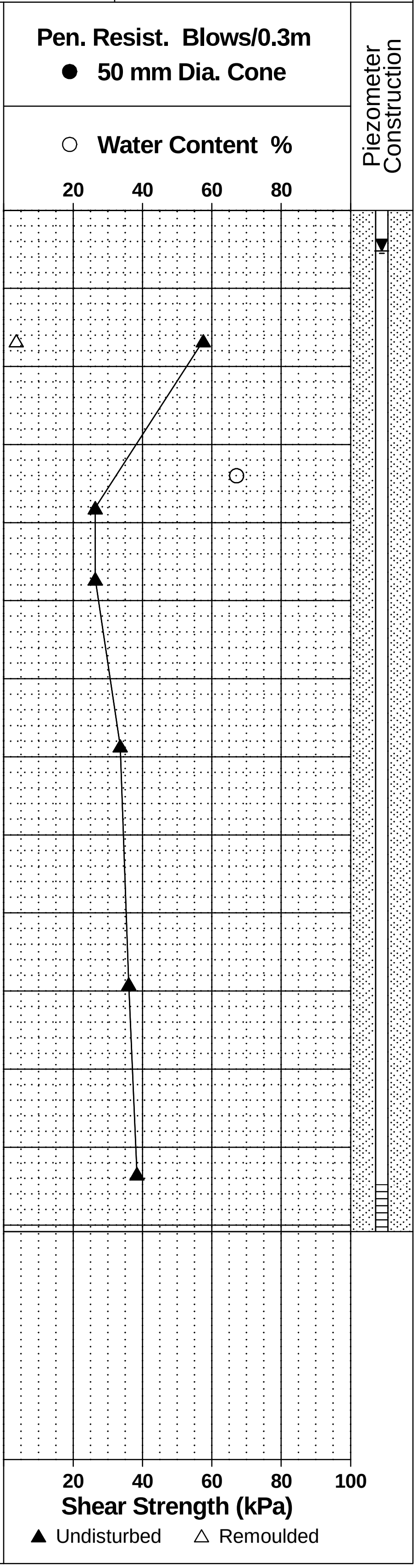
End of Borehole

Practical refusal to augering @ 13.08m depth

(GWL @ 0.52m-Feb. 16/06)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	75	2
SS	2	75	1
TW	3	100	
TW	4	0	
SS	5	100	1
TW	6	62	
SS	7	100	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
8	84.67
9	83.67
10	82.67
11	81.67
12	80.67
13	79.67



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154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS Wash boring methods used.

BORINGS BY CME 75 Power Auger

DATE Jun 2, 06

FILE NO. PG0538

HOLE NO. BH24

SOIL DESCRIPTION

GROUND SURFACE

TOPSOIL 0.10

Stiff to firm, brown **SILTY CLAY**

- firm and grey by 2.1m depth

- soft between approx. 5.5 and 7.5m depth

9.90

GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders

11.17

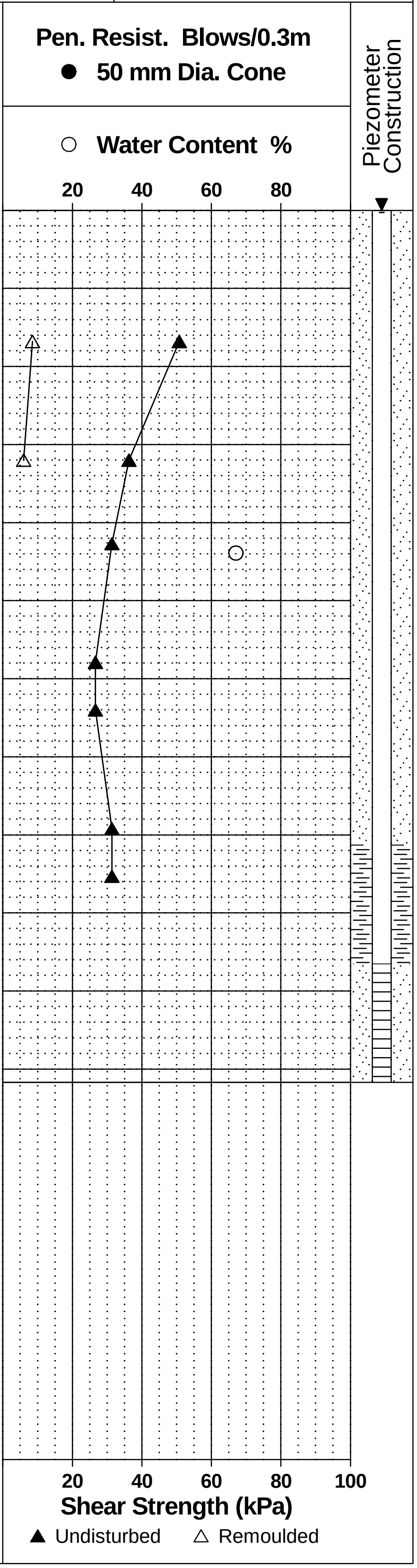
End of Borehole

Practical refusal to advancement of NW casing by wash boring @ 11.17m depth.

(GWL @ 1.20m above ground surface in PVC standpipe - June 18/06. Standpipe installed in till)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	100	1
SS	2	100	1
TW	3	96	
TW	4	50	
SS	5		89+

DEPTH (m)	ELEV. (m)
0	92.64
1	91.64
2	90.64
3	89.64
4	88.64
5	87.64
6	86.64
7	85.64
8	84.64
9	83.64
10	82.64
11	81.64



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS Wash boring methods used.

BORINGS BY CME 75 Power Auger

DATE Jun 5, 06

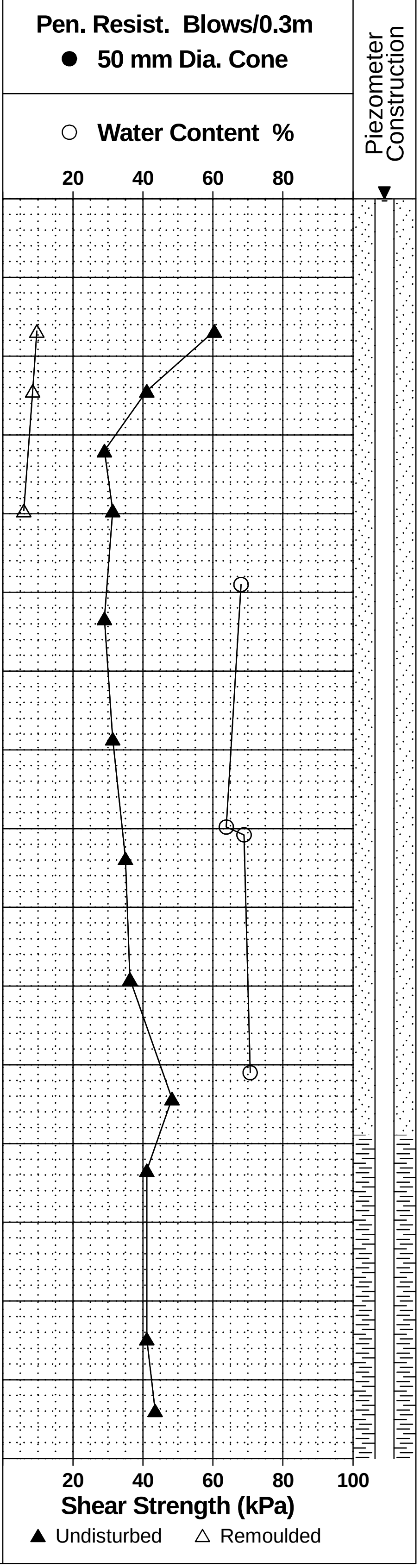
FILE NO. PG0538

HOLE NO. BH25

SOIL DESCRIPTION			
GROUND SURFACE			
TOPSOIL	0.30		
Grey **SILTY SAND**	0.60		
Stiff to firm, brown **SILTY CLAY**			
- firm and grey by 2.7m depth			

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	75	3
TW	2	100	
TW	3	100	
TW	4	100	
TW	5	100	
TW	6	100	
TW	7	100	

DEPTH (m)	ELEV. (m)
0	92.55
1	91.55
2	90.55
3	89.55
4	88.55
5	87.55
6	86.55
7	85.55
8	84.55
9	83.55
10	82.55
11	81.55
12	80.55
13	79.55
14	78.55
15	77.55
16	76.55



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS Wash boring methods used.

BORINGS BY CME 75 Power Auger

DATE Jun 5, 06

FILE NO. **PG0538**

HOLE NO. **BH25**

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
Firm, grey **SILTY CLAY**						16	76.55		
						17	75.55		
18.87						18	74.55		
GLACIAL TILL 19.13						19	73.55		

End of Borehole

Practical refusal to advancement of NW casing by wash boring @ 19.13m depth

(GWL @ 0.35m above ground surface in PVC standpipe - June 18/06. Standpipe installed in till)

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS Wash boring methods used.

BORINGS BY CME 45 Power Auger

DATE Jun 6, 07

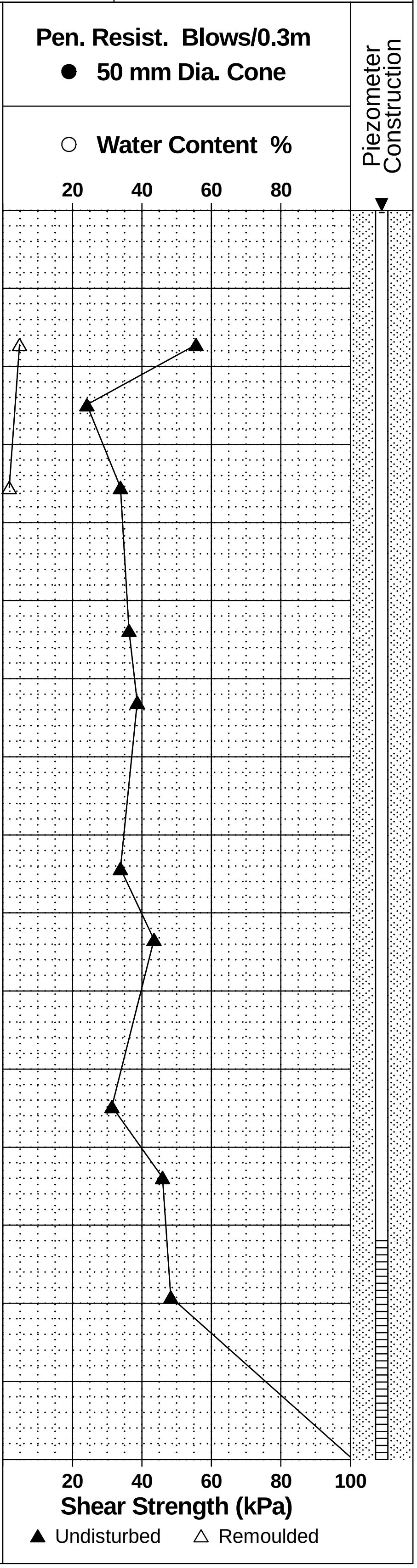
FILE NO. PG0538

HOLE NO. BH41

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOIL	0.36
Brown **SILTY** fine **SAND**	0.76
Stiff, brown **SILTY CLAY**, trace sand	2.13
Firm, grey **SILTY CLAY**	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	100	2
TW	2	100	
TW	3	100	
TW	4	71	
TW	5	100	
TW	6	100	

DEPTH (m)	ELEV. (m)
0	93.43
1	92.43
2	91.43
3	90.43
4	89.43
5	88.43
6	87.43
7	86.43
8	85.43
9	84.43
10	83.43
11	82.43
12	81.43
13	80.43
14	79.43
15	78.43
16	77.43



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154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development-Huntmar Road, Kanata
Ottawa, Ontario

DATUM Ground surface elevation provided by Webster and Simmonds Surveying Limited.

REMARKS Wash boring methods used.

BORINGS BY CME 45 Power Auger

DATE Jun 6, 07

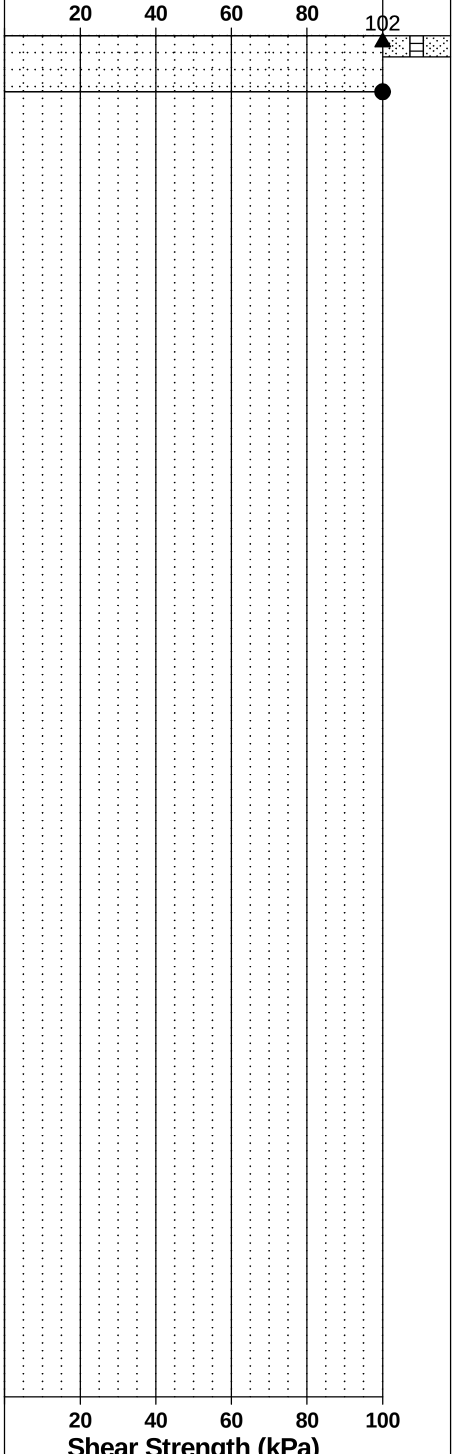
FILE NO. PG0538

HOLE NO. BH41

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GLACIAL TILL	16.08						16	77.43
Dynamic Cone Penetration Test commenced @ 16.25m depth	16.25							
Inferred **GLACIAL TILL**	16.66							
End of Borehole DCPT refusal @ 16.66m depth (GWL @ 0.21m above ground surface - July 4/07)								

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Arcadia Development - Huntmar Road - Stage 3A
Ottawa, Ontario

DATUM Approximate geodetic.

REMARKS

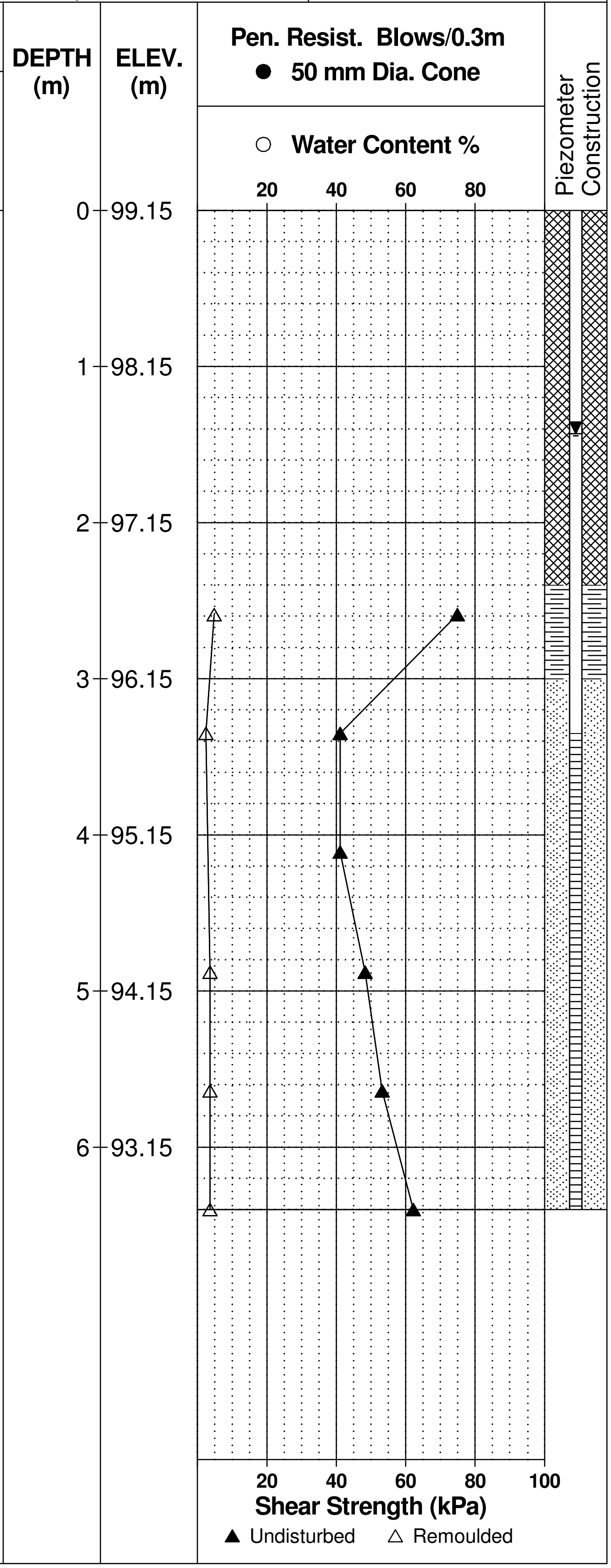
BORINGS BY CME 55 Power Auger

DATE 2012 April 16

FILE NO. **PG0538**

HOLE NO. **BH 5-12**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
Stiff, brown **SILTY CLAY**	AU	1		
	SS	2	100	9
	SS	3	100	6
2.80				
Firm to stiff, grey **SILTY CLAY**				
6.40				
End of Borehole				
(GWL @ 1.43m-April 25, 202)				



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

Cc - Concavity coefficient = $(D30)^2$ / (D10 x 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)

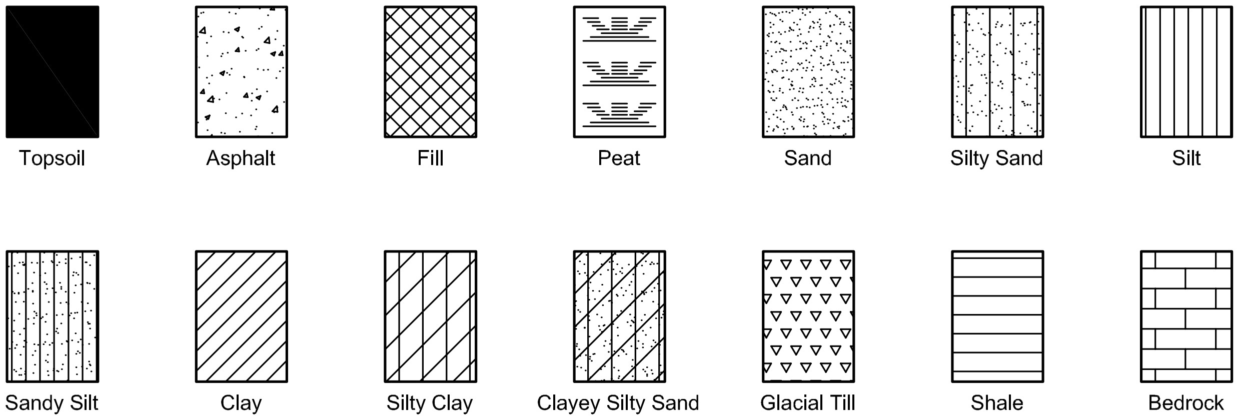
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

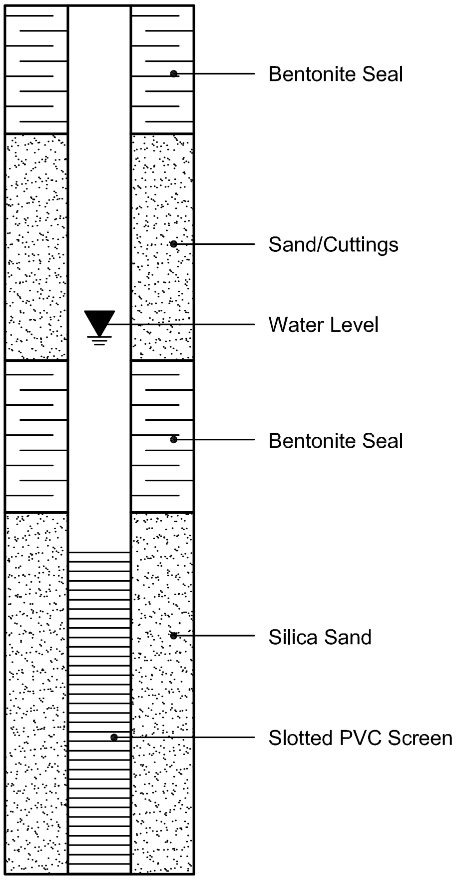
SYMBOLS AND TERMS (continued)

STRATA PLOT

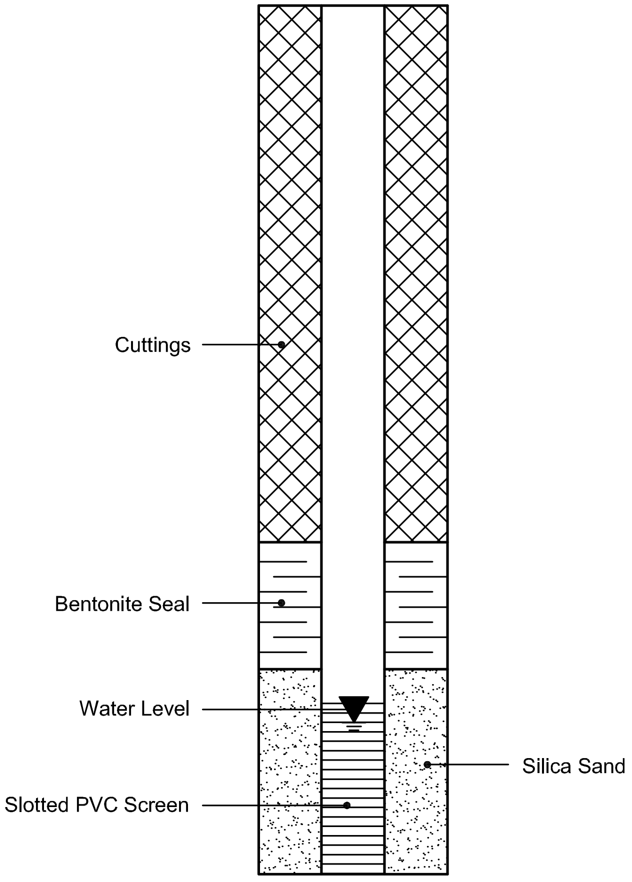


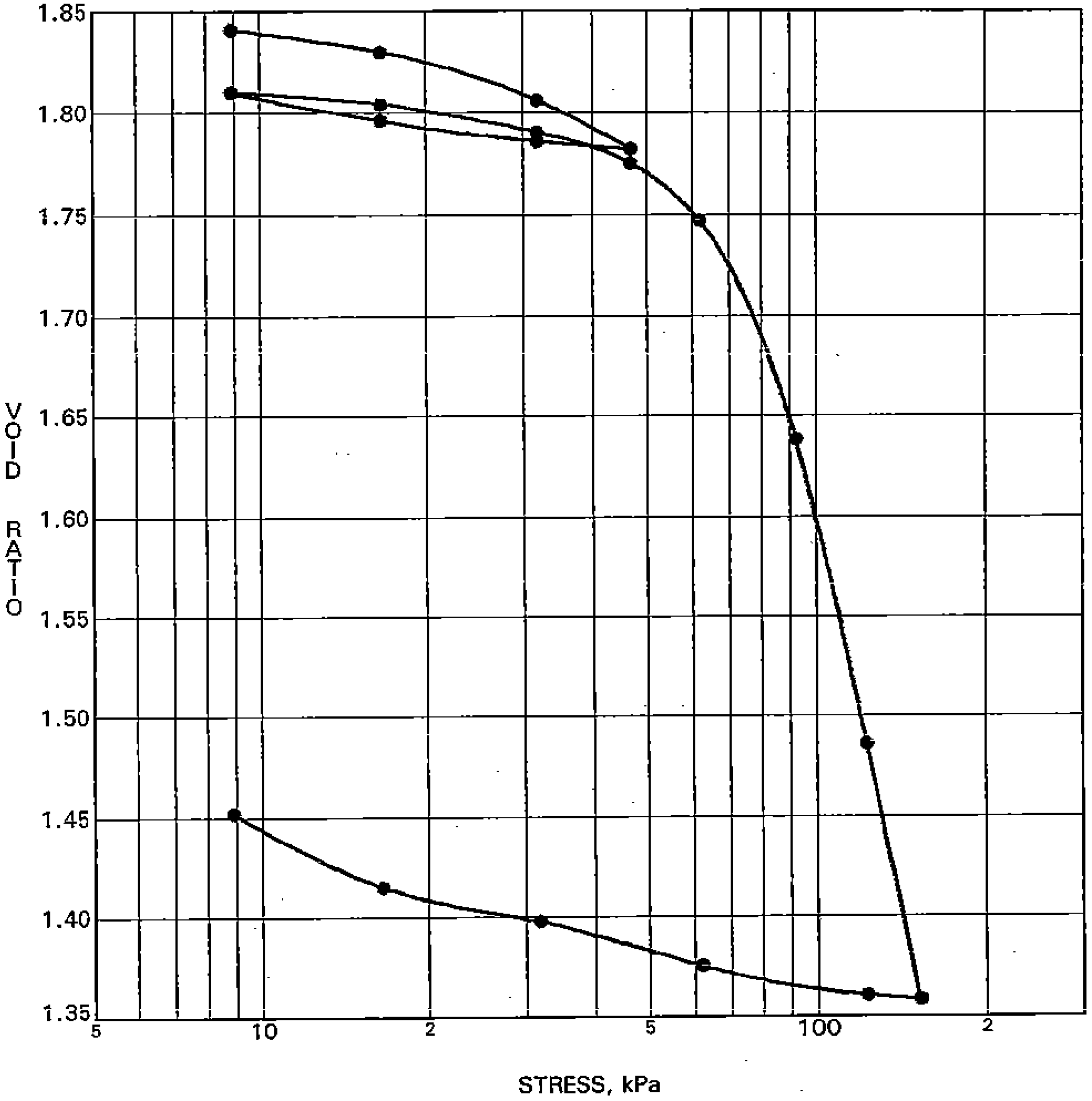
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





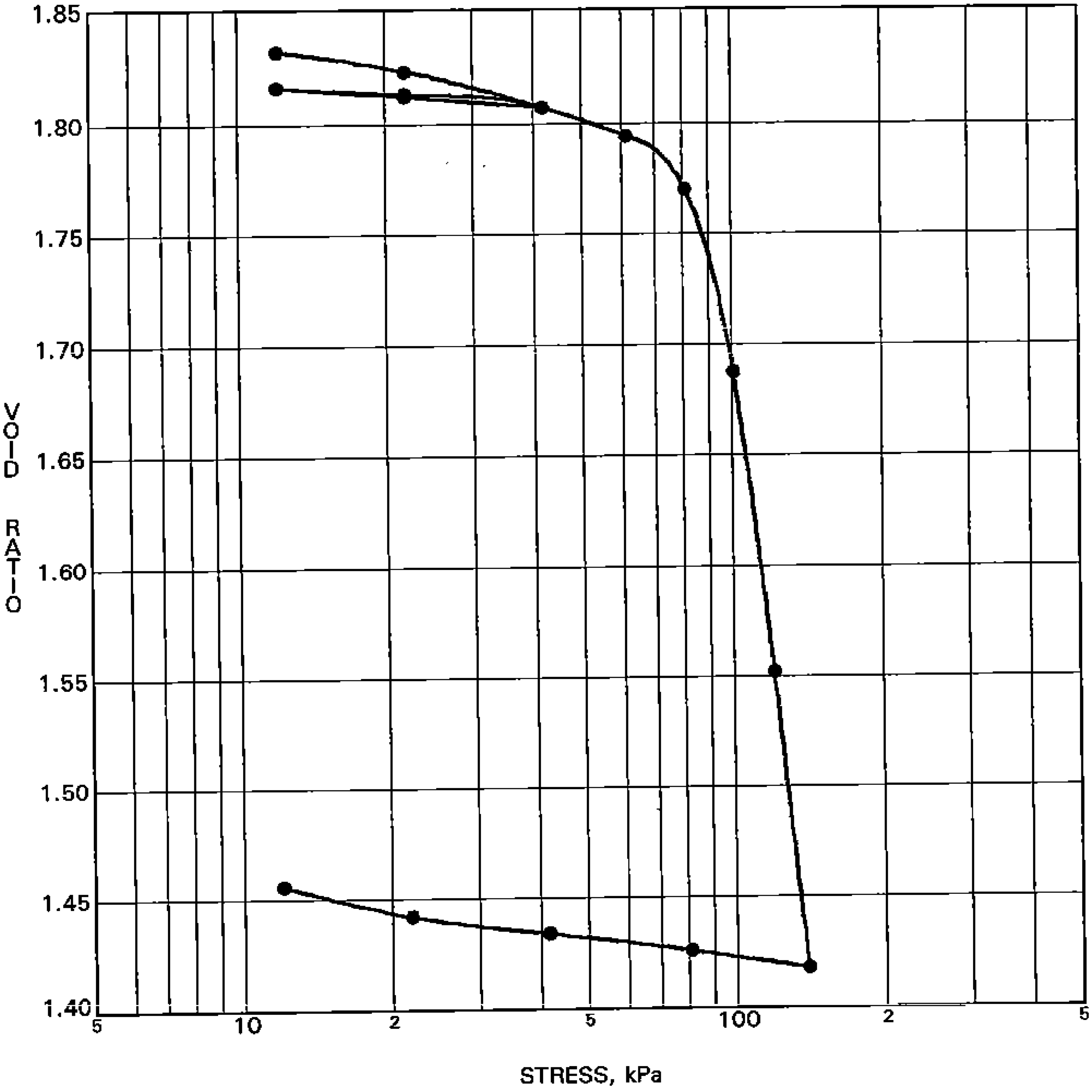
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH17	p'$_o$	29 kPa	Ccr	0.046
Sample No.	TW 3	p'$_c$	72 kPa	Cc	1.226
Sample Depth	3.40 m	OC Ratio	2.5	Wo	67.1 %
Sample Elev.	89.27 m	Void Ratio	1.846	Unit Wt.	15.9 kN/m^3

CLIENT	Minto Developments Inc.	FILE NO.	PG0538
PROJECT	Geotechnical Investigation - Kanata West Lands-Huntmar Road to Carp River	DATE	14/03/06

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH24	p'_o	35 kPa	Ccr	0.017
Sample No.	TW 3	p'_c	92 kPa	Cc	1.730
Sample Depth	4.39 m	OC Ratio	2.6	Wo	67.1 %
Sample Elev.	88.25 m	Void Ratio	1.844	Unit Wt.	15.8 kN/m^3

CLIENT Minto Developments Inc.

PROJECT Geotechnical Investigation - Kanata West
Lands-Huntmar Road to Carp River

FILE NO. PG0538

DATE 04/07/06

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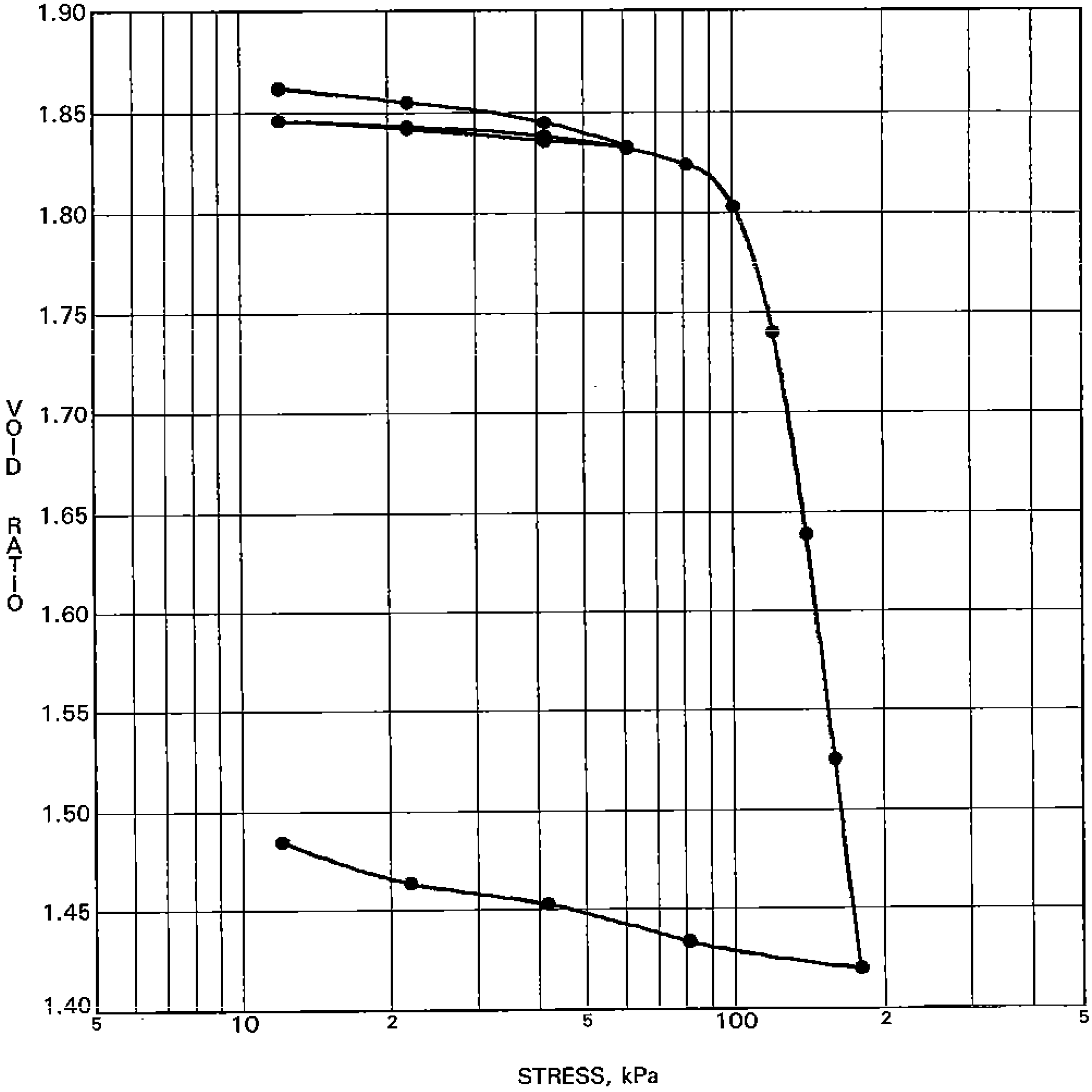
28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH25	p'_o	38 kPa	Ccr	0.018
Sample No.	TW 2	p'_c	115 kPa	Cc	1.239
Sample Depth	4.90 m	OC Ratio	3.0	Wo	68.0 %
Sample Elev.	87.65 m	Void Ratio	1.871	Unit Wt.	15.8 kN/m^3

CLIENT **Minto Developments Inc.**

PROJECT **Geotechnical Investigation - Kanata West**
Lands-Huntmar Road to Carp River

FILE NO. **PG0538**

DATE **21/09/06**

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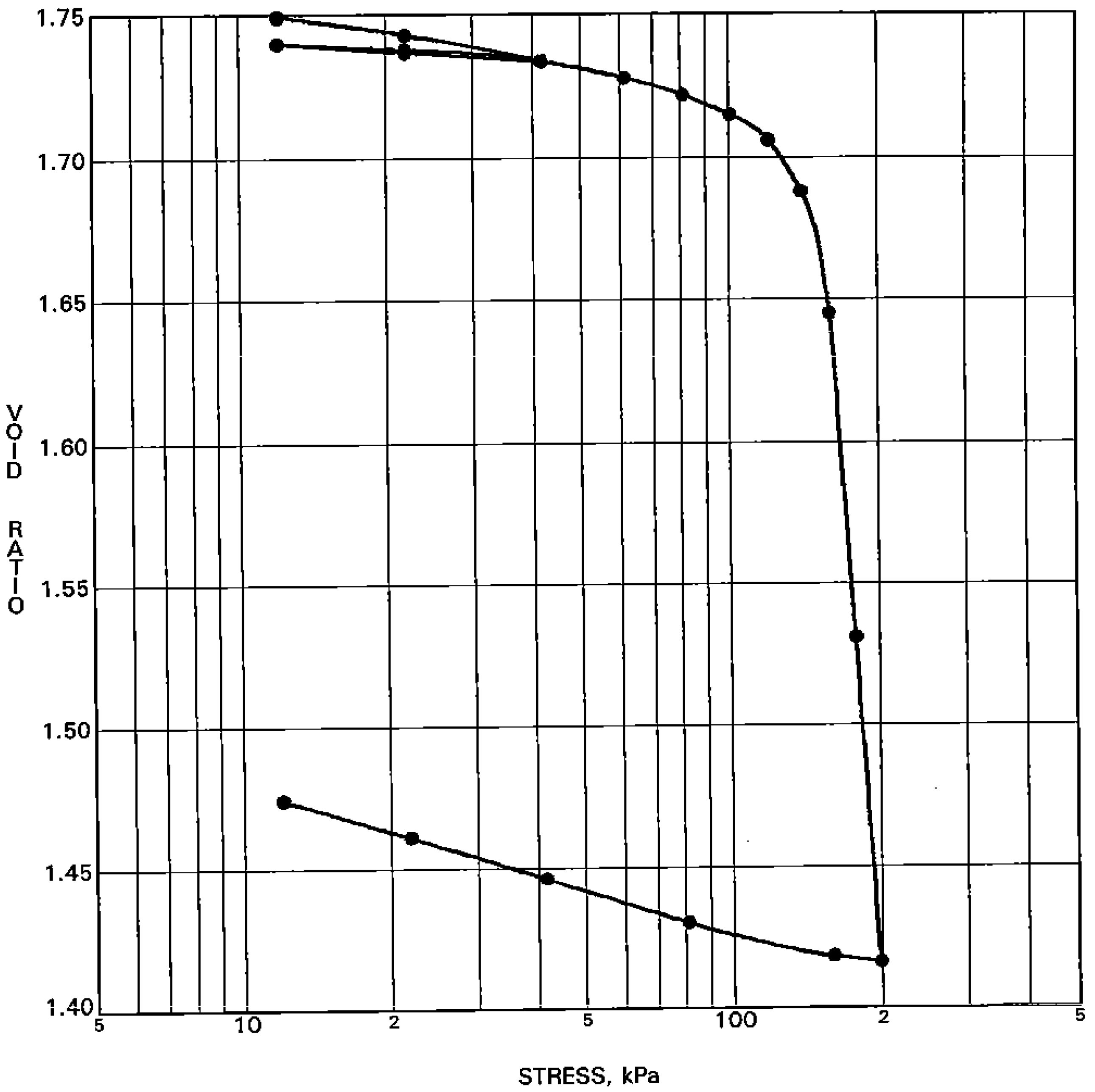
28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY

Borehole No.	BH25	p'_o	55 kPa	Ccr	0.011
Sample No.	TW 4 Vert.	p'_c	154 kPa	Cc	1.178
Sample Depth	7.98 m	OC Ratio	2.8	Wo	63.8 %
Sample Elev.	84.57 m	Void Ratio	1.755	Unit Wt.	16.0 kN/m^3

CLIENT **Minto Developments Inc.**

PROJECT **Geotechnical Investigation - Kanata West**

Lands-Huntmar Road to Carp River

FILE NO. **PG0538**

DATE **16/06/06**

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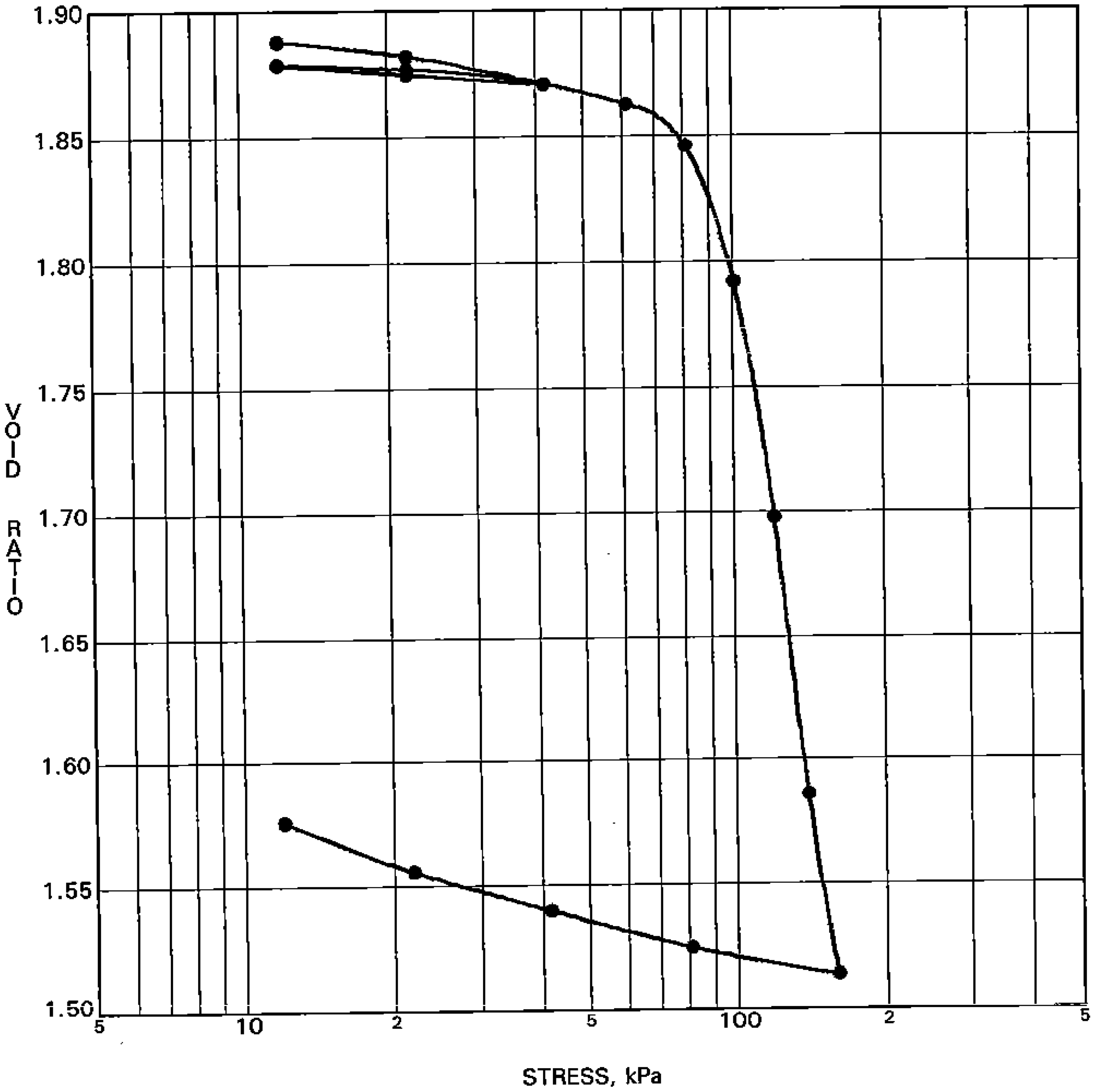
28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY

Borehole No.	BH25	p'_o	35 kPa	Ccr	0.015
Sample No.	TW 4 Trans.	p'_c	95 kPa	Cc	2.884
Sample Depth	8.08 m	OC Ratio	2.7	Wo	68.9 %
Sample Elev.	84.47 m	Void Ratio	1.895	Unit Wt.	15.7 kN/m^3

CLIENT **Minto Developments Inc.**

PROJECT **Geotechnical Investigation - Kanata West**
Lands-Huntmar Road to Carp River

FILE NO. **PG0538**

DATE **15/06/06**

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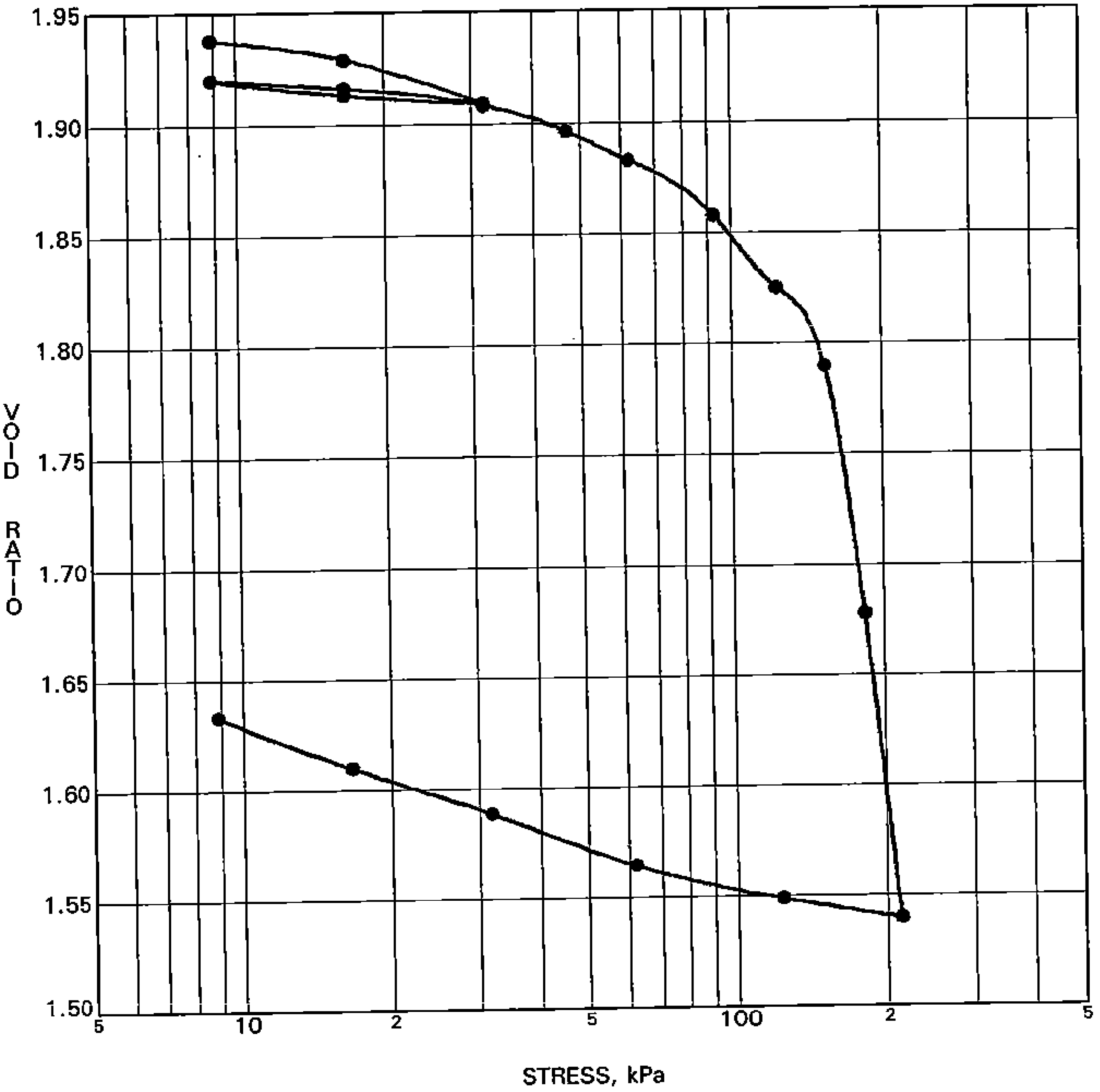
CONSOLIDATION TEST



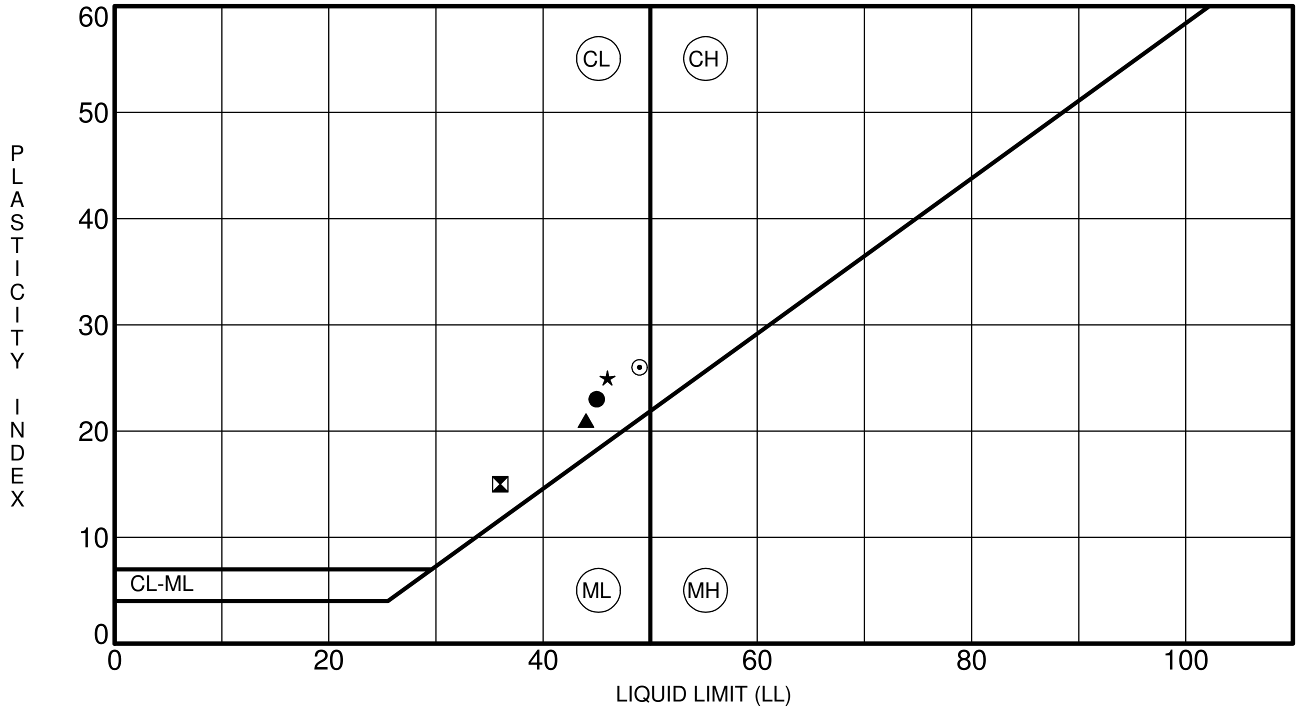
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH25	p'_o	73 kPa	Ccr	0.023
Sample No.	TW 6	p'_c	149 kPa	Cc	2.064
Sample Depth	11.10 m	OC Ratio	2.0	Wo	70.6 %
Sample Elev.	81.45 m	Void Ratio	1.942	Unit Wt.	15.6 kN/m^3

CLIENT	**Minto Developments Inc.**	FILE NO.	PG0538
PROJECT	**Geotechnical Investigation - Kanata West Lands-Huntmar Road to Carp River**	DATE	14/06/06

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

CONSOLIDATION TEST



	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 1-21 SS3**	**45**	**22**	**23**		**CL = Inorganic Clays of Low Plasticity**
⧗	**BH 2-21 SS3**	**36**	**21**	**15**	**95.8**	**CL = Inorganic Clays of Low Plasticity**
▲	**BH 3-21 SS8**	**44**	**23**	**21**	**98.5**	**CL = Inorganic Clays of Low Plasticity**
★	**BH 4-21 SS6**	**46**	**21**	**25**		**CL= Inorganic Clays of Low Plasticity**
⊙	**BH 5-21 SS3**	**49**	**23**	**26**	**92.4**	**CL = Inorganic Clays of Low Plasticity**

CLIENT **Minto Communities Inc.**

PROJECT **Geotechnical Investigation - Arcadia Stage 5 - Campeau Drive**

FILE NO. **PG4933**

DATE **13 Jan 21**

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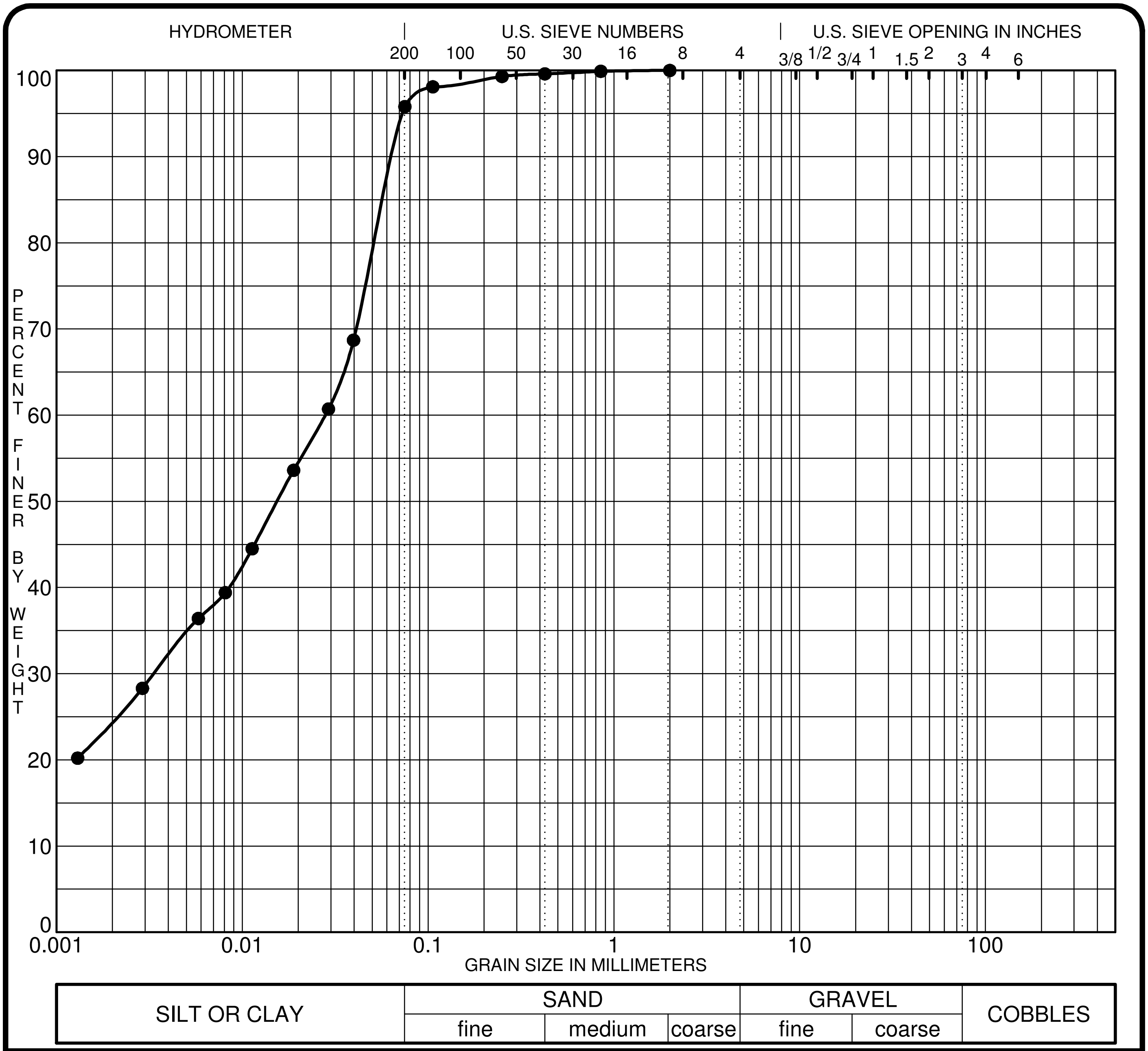
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
● **BH 2-21 SS3**	**CL = Inorganic Clays of Low Plasticity**		**36**	**21**	**15**		

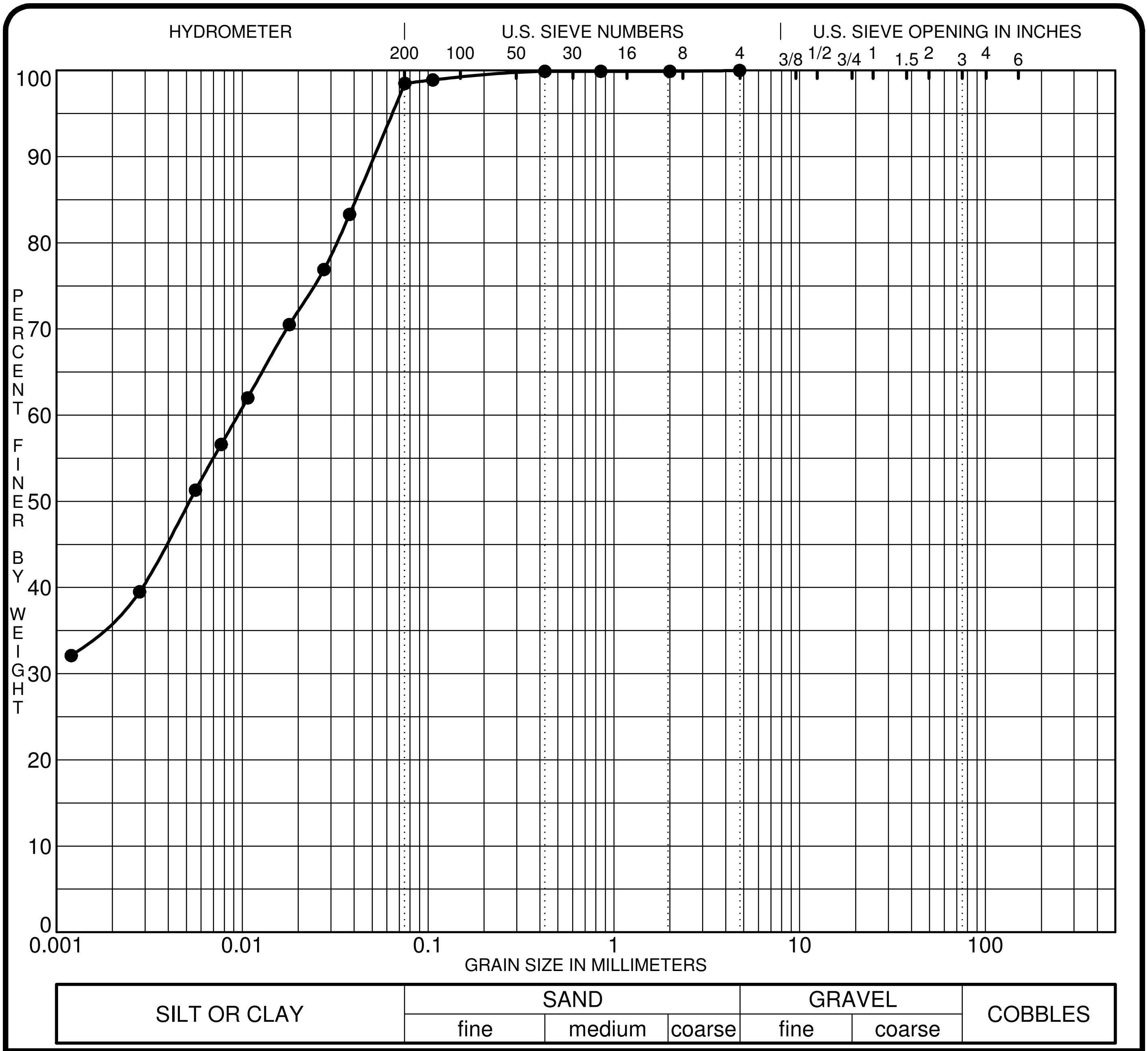
Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **BH 2-21 SS3**	**2.00**	**0.03**	**0.003**		**0.0**	**4.2**	**95.8**	

CLIENT	**Minto Communities Inc.**	FILE NO.	**PG4933**
PROJECT	**Geotechnical Investigation - Arcadia Stage 5**	DATE	**13 Jan 21**
	Campeau Drive		

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 3-21 SS8**	**CL = Inorganic Clays of Low Plasticity**		**44**	**23**	**21**		

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 3-21 SS8**	**4.75**	**0.01**			**0.0**	**1.5**	**98.5**	

CLIENT **Minto Communities Inc.**

FILE NO. **PG4933**

PROJECT **Geotechnical Investigation - Arcadia Stage 5**
Campeau Drive

DATE **13 Jan 21**

patersongroup **Consulting Engineers**
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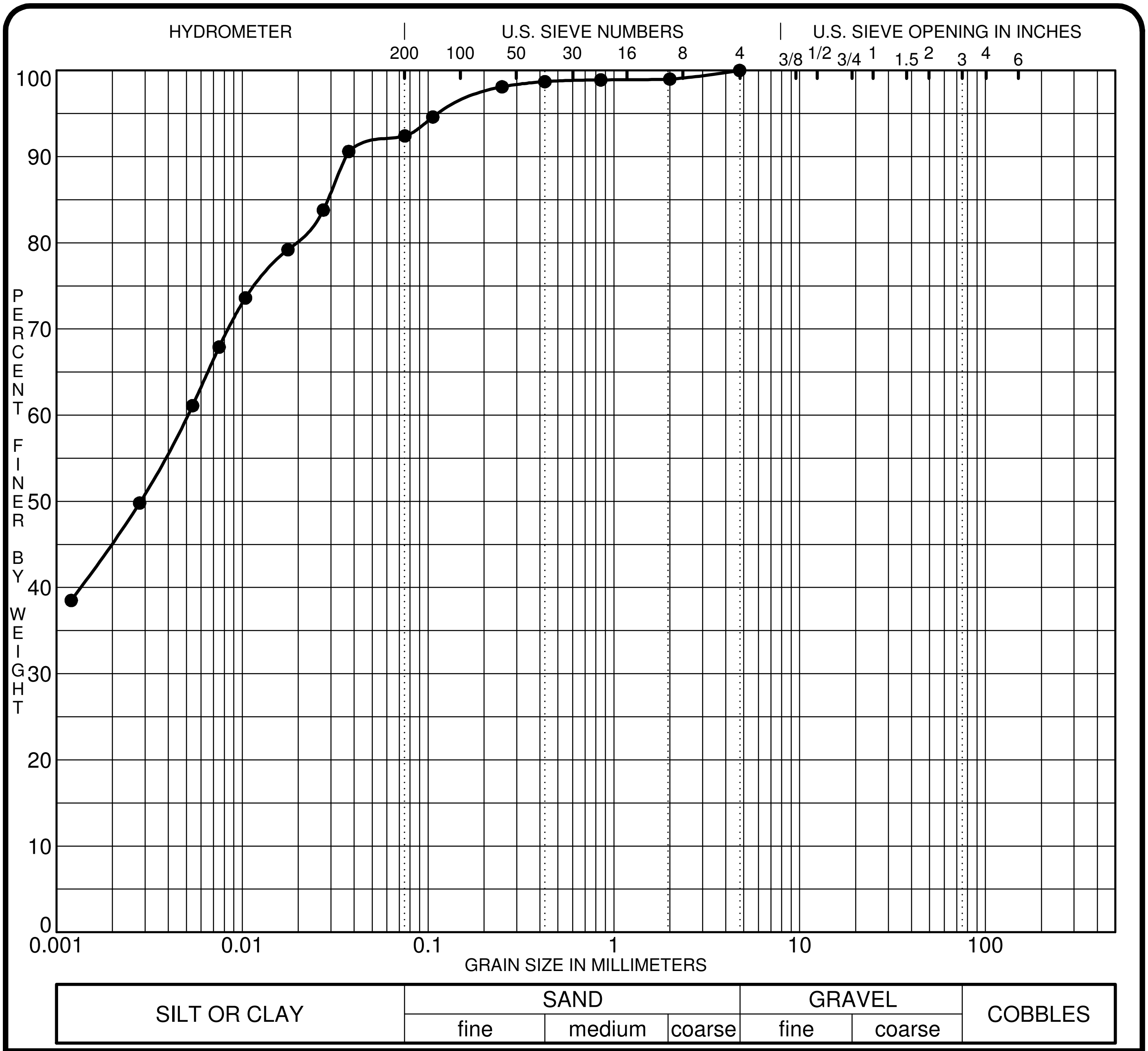
GRAIN SIZE DISTRIBUTION



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 5-21 SS3**	**CL = Inorganic Clays of Low Plasticity**		**49**	**23**	**26**		

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 5-21 SS3**	**4.75**	**0.01**			**0.0**	**7.6**	**92.4**	

CLIENT **Minto Communities Inc.**
FILE NO. **PG4933**
PROJECT **Geotechnical Investigation - Arcadia Stage 5**
Campeau Drive
DATE **13 Jan 21**

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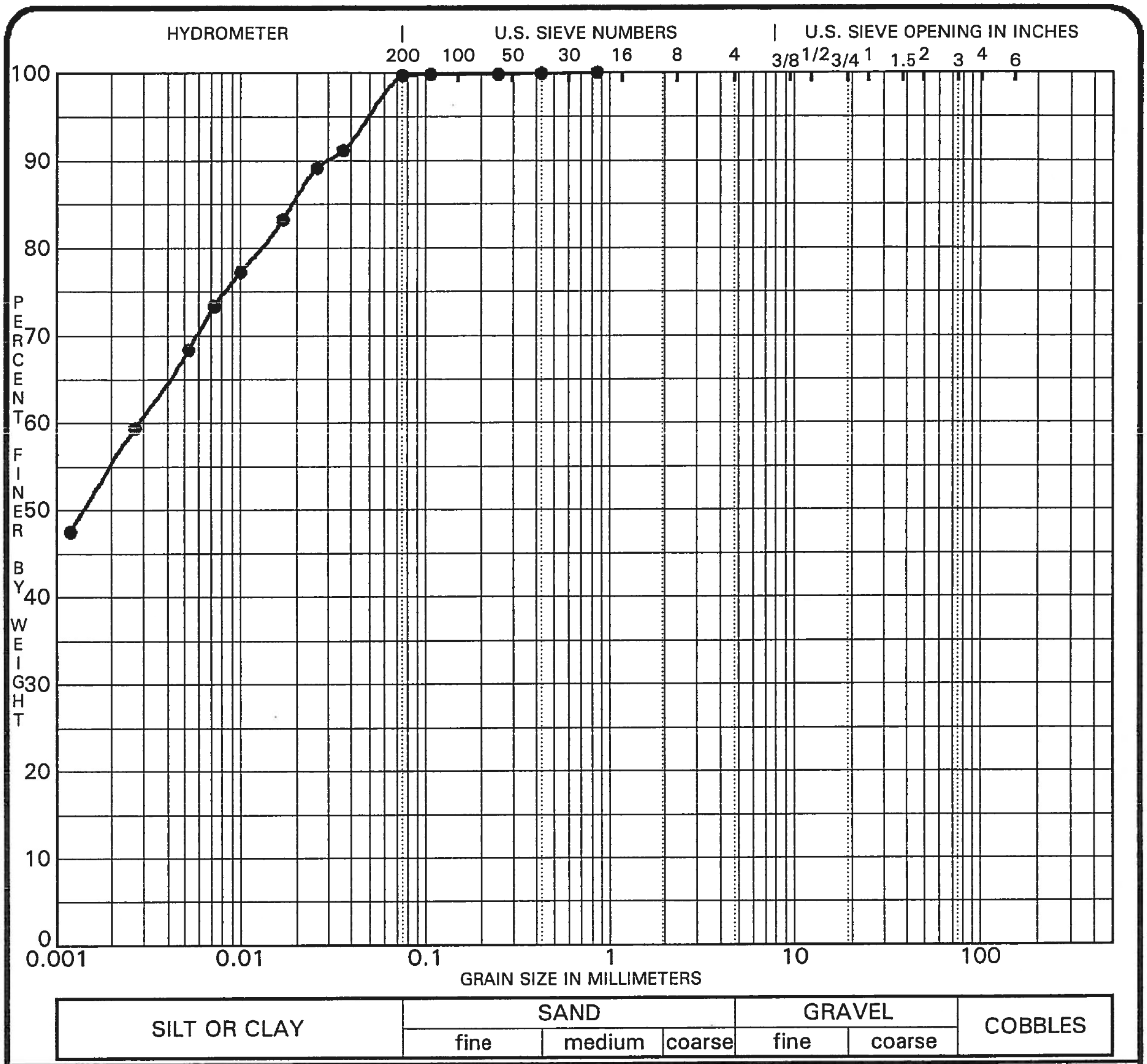
GRAIN SIZE DISTRIBUTION



Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
● **BH16 SS 7**	**SILTY CLAY** (CL)		**34**	**20**	**15**		

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **BH16 SS 7**	**0.85**	**0.003**			**0.0**	**0.3**	99.7	

CLIENT **Minto Developments Inc.**

PROJECT **Geotechnical Investigation - Kanata West**
Lands-Huntmar Road to Carp River

FILE NO. **PG0538**

DATE **16 FEB 06**

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

GRAIN SIZE DISTRIBUTION

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - SECTION A - STATIC CONDITIONS

FIGURE 3 - SECTION A - SEISMIC CONDITIONS

FIGURE 4 - SECTION B - STATIC CONDITIONS

FIGURE 5 - SECTION B - SEISMIC CONDITIONS

DRAWING PG4933-1 - TEST HOLE LOCATION PLAN

DRAWING PG4933-2 - PERMISSIBLE GRADE RAISE PLAN

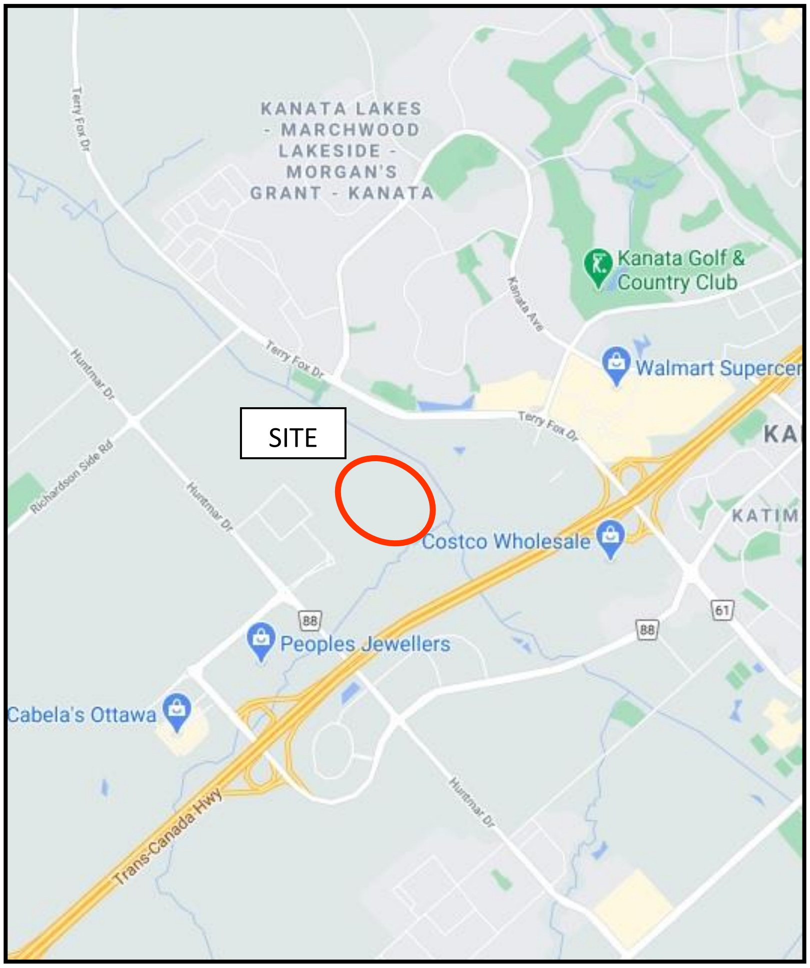


FIGURE 1

KEY PLAN

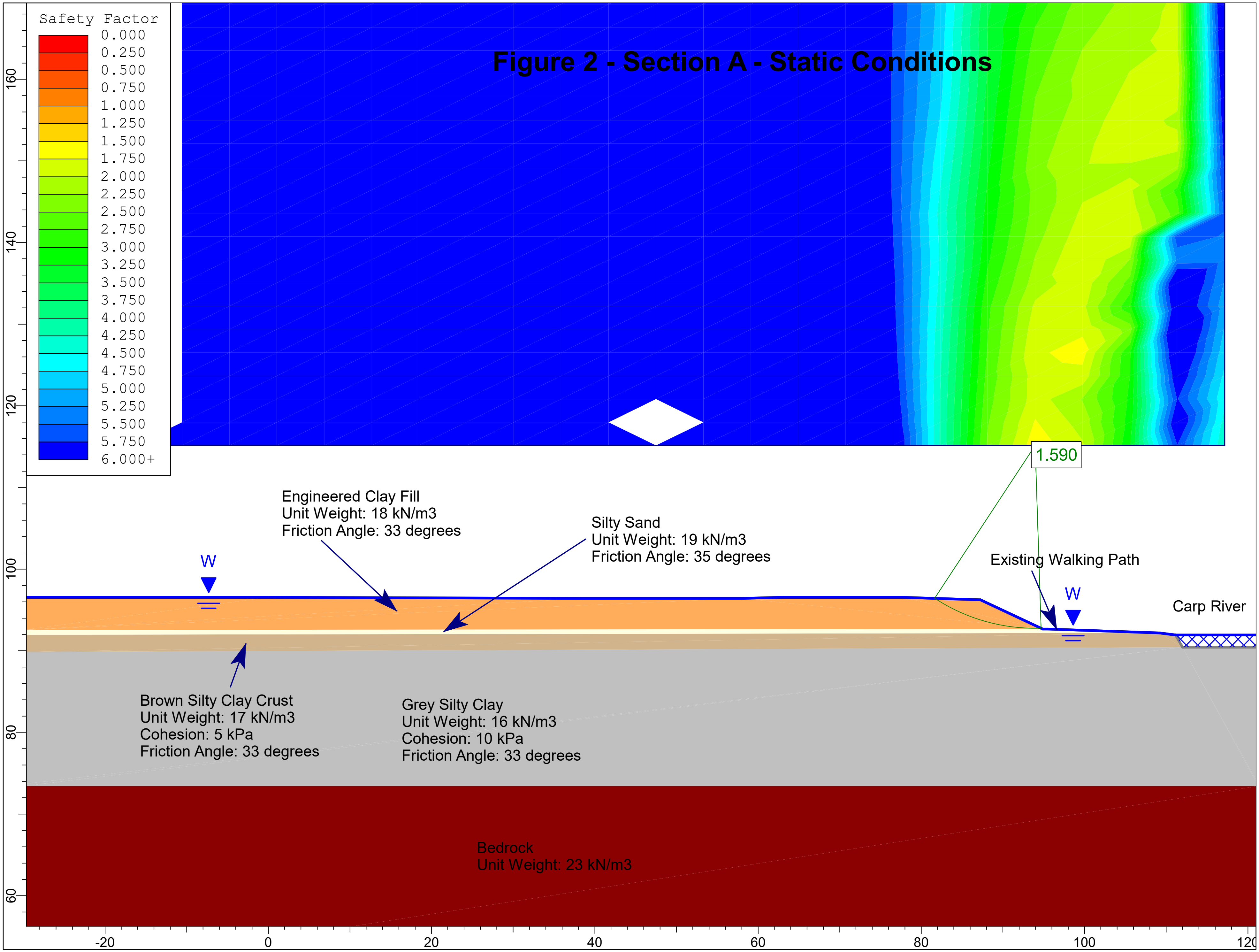


Figure 2 - Section A - Static Conditions

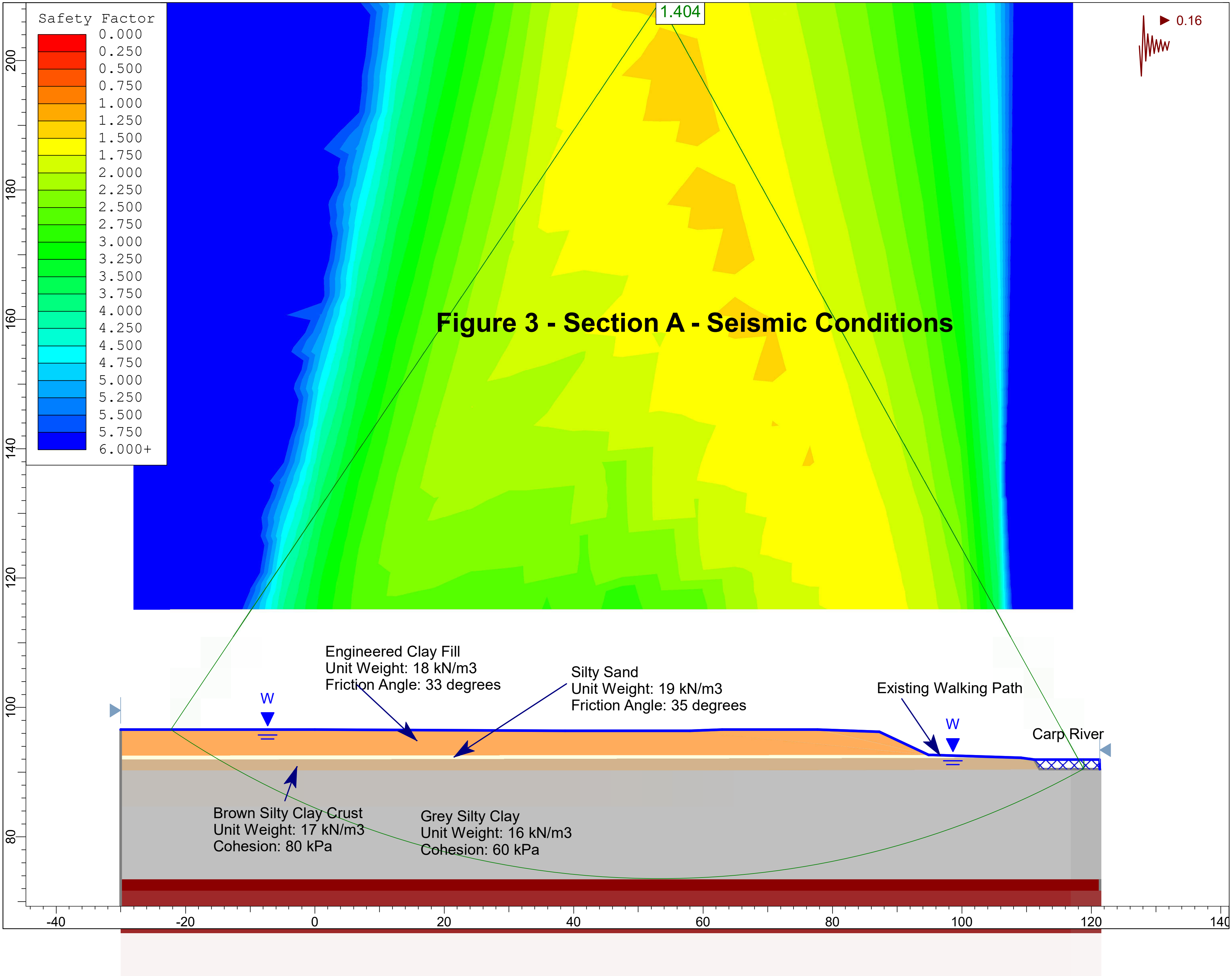


Figure 3 - Section A - Seismic Conditions

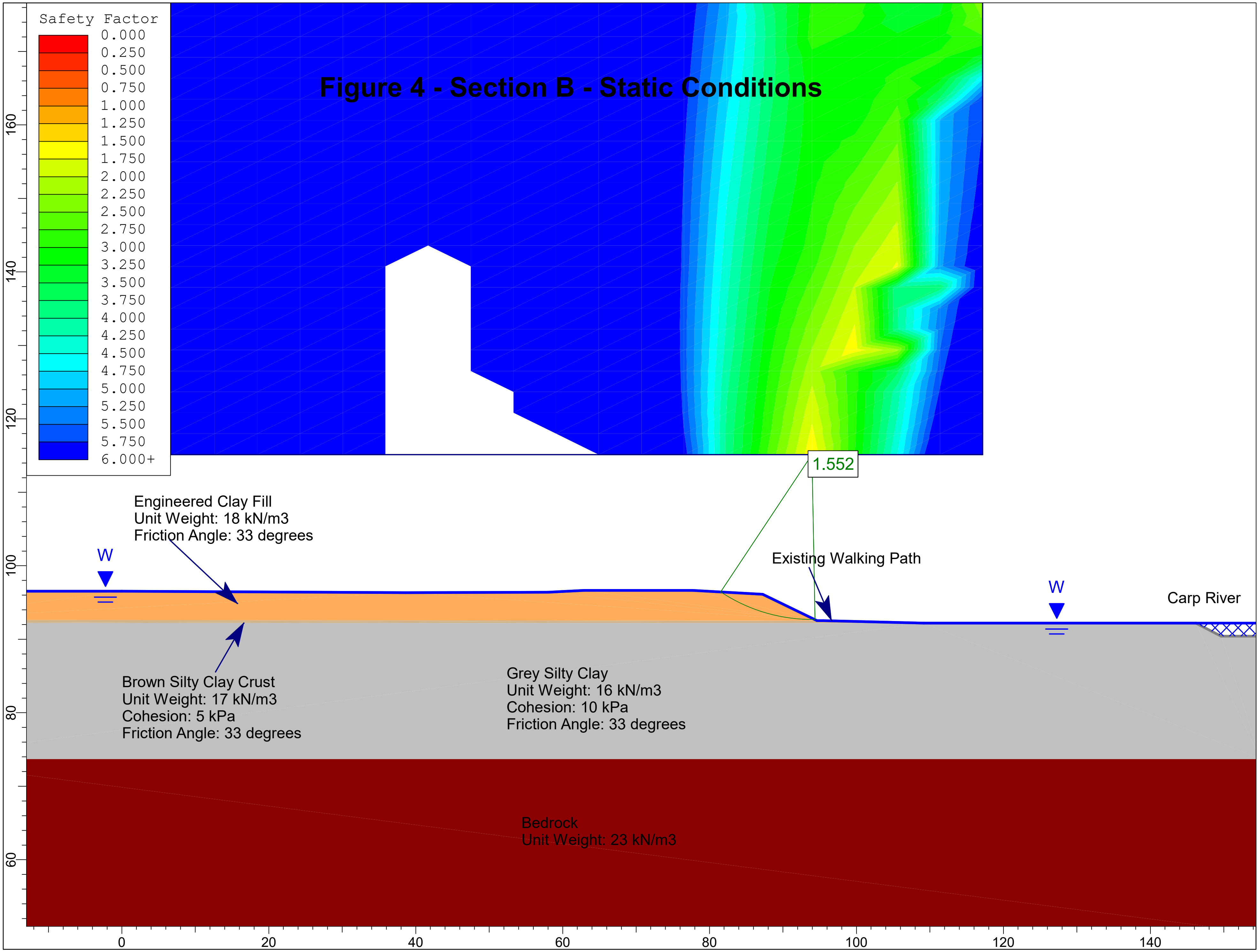


Figure 4 - Section B - Static Conditions

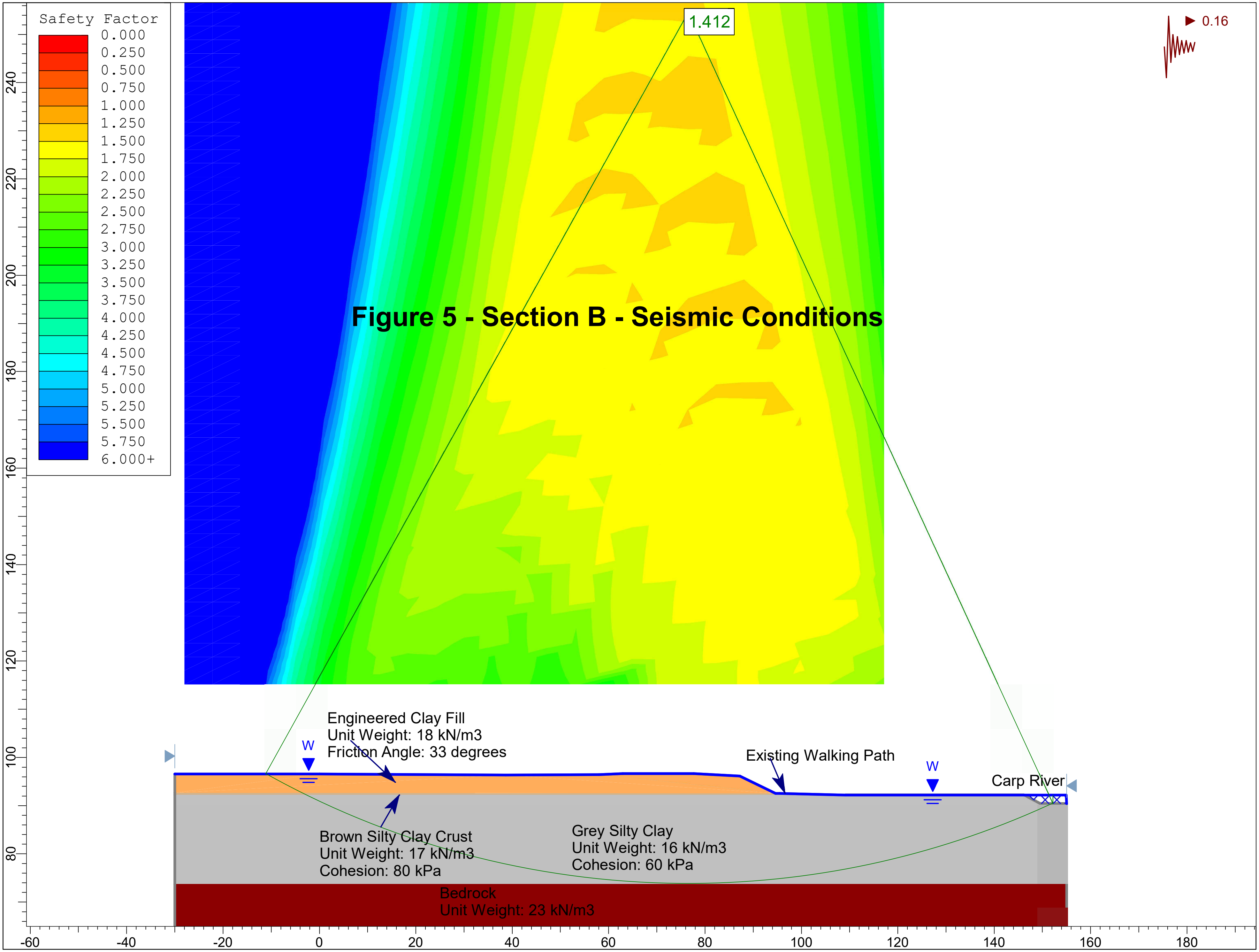
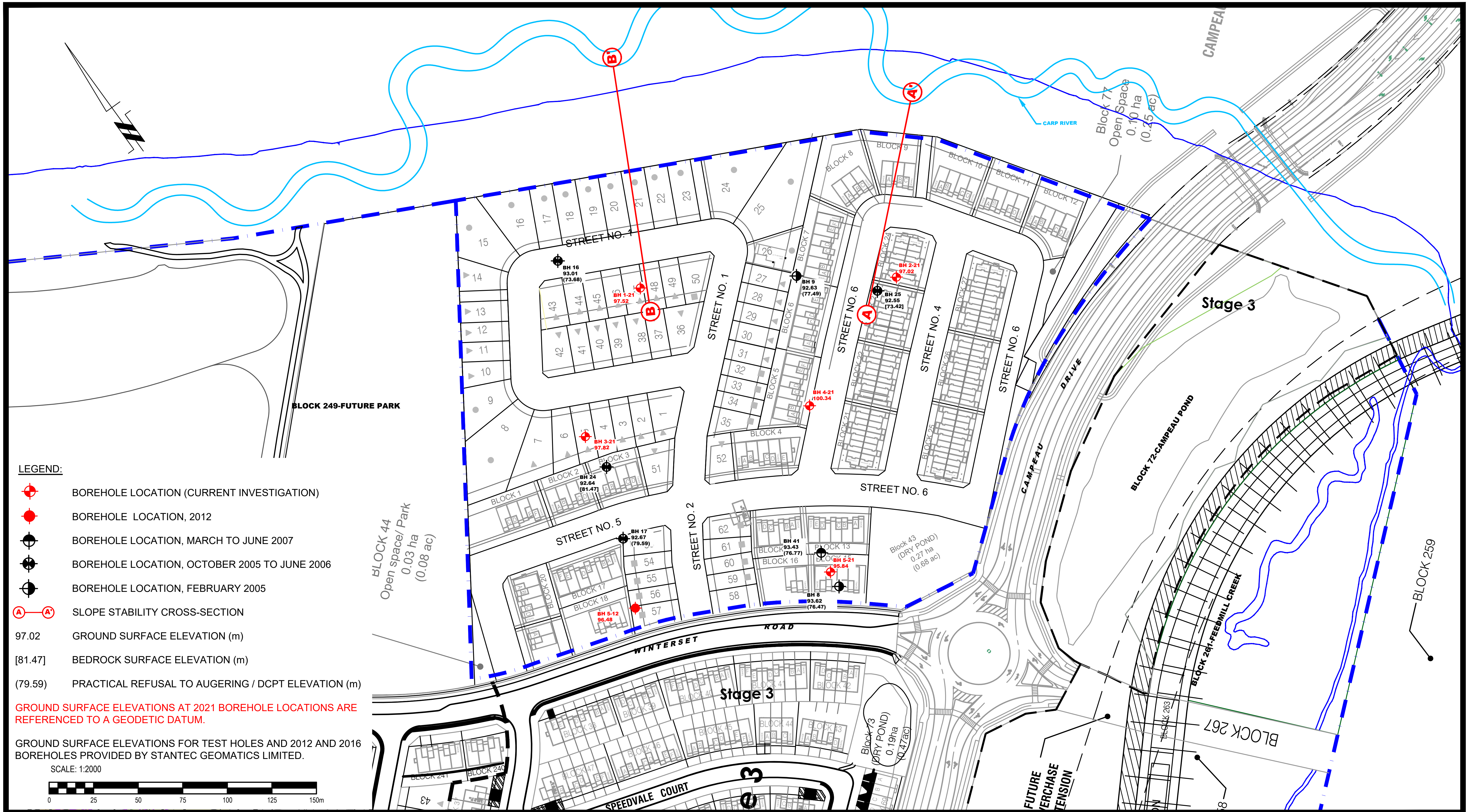


Figure 5 - Section B - Seismic Conditions



LEGEND:

⌖	BOREHOLE LOCATION (CURRENT INVESTIGATION)
●	BOREHOLE LOCATION, 2012
⊕	BOREHOLE LOCATION, MARCH TO JUNE 2007
⊗	BOREHOLE LOCATION, OCTOBER 2005 TO JUNE 2006
⊕	BOREHOLE LOCATION, FEBRUARY 2005
Ⓐ—Ⓐ'	SLOPE STABILITY CROSS-SECTION
97.02	GROUND SURFACE ELEVATION (m)
[81.47]	BEDROCK SURFACE ELEVATION (m)
(79.59)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

GROUND SURFACE ELEVATIONS AT 2021 BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

GROUND SURFACE ELEVATIONS FOR TEST HOLES AND 2012 AND 2016 BOREHOLES PROVIDED BY STANTEC GEOMATICS LIMITED.

SCALE: 1:2000

0 25 50 75 100 125 150m

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Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
1	SLOPE STABILITY CROSS-SECTION ADDED	18/11/2021	NP

MINTO COMMUNITIES

GEOTECHNICAL INVESTIGATION
ARCADIA STAGE 5 - CAMPEAU DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:2000	Date:	02/2021
Drawn by:	NFRV	Report No.:	PG4933-1
Checked by:	NP	Dwg. No.:	PG4933-1
Approved by:	FA	Revision No.:	1

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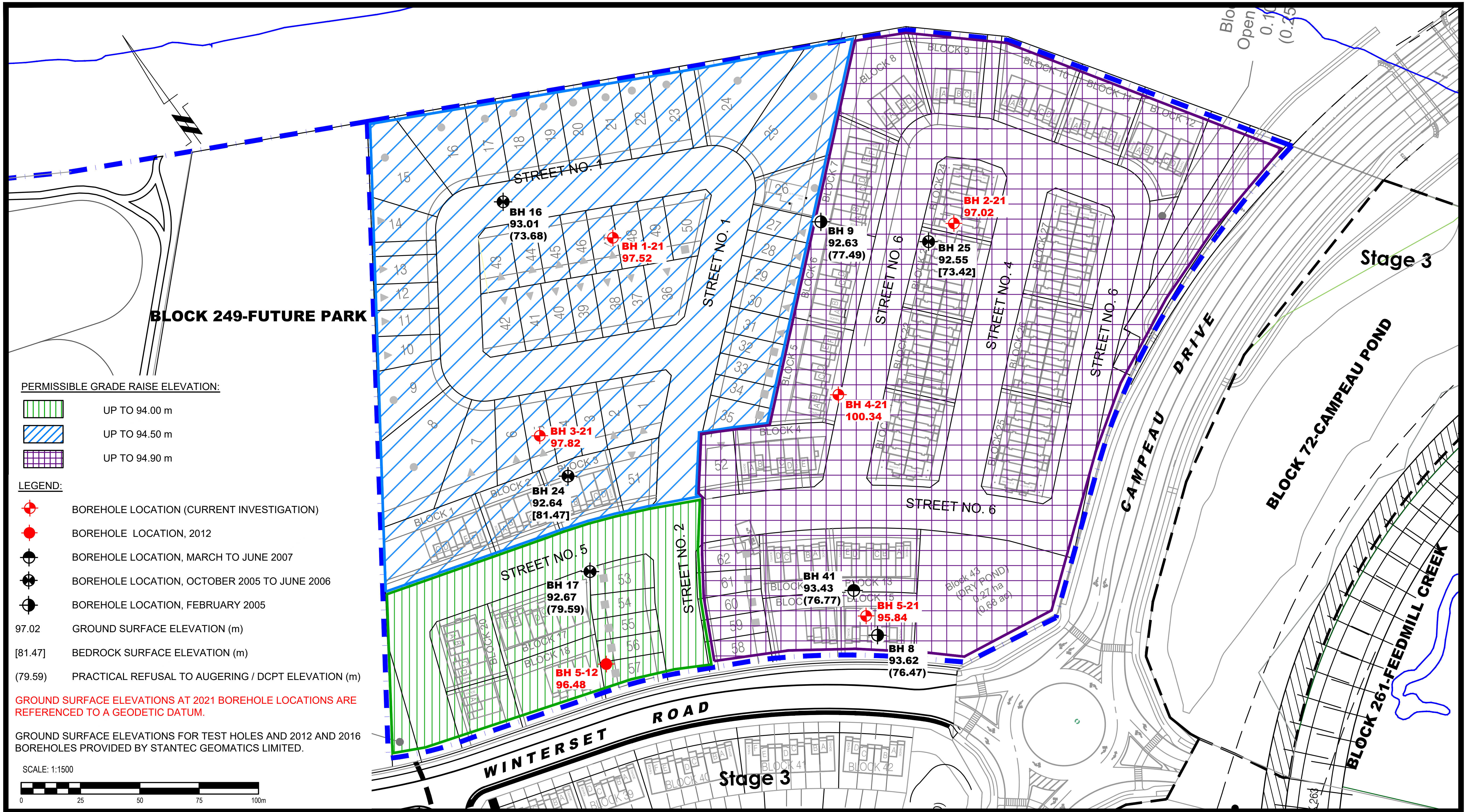

PERMISSIBLE GRADE RAISE ELEVATION:

🟩	UP TO 94.00 m
▨	UP TO 94.50 m
▦	UP TO 94.90 m

LEGEND:

⊕	BOREHOLE LOCATION (CURRENT INVESTIGATION)
●	BOREHOLE LOCATION, 2012
⊕	BOREHOLE LOCATION, MARCH TO JUNE 2007
⊗	BOREHOLE LOCATION, OCTOBER 2005 TO JUNE 2006
⊕	BOREHOLE LOCATION, FEBRUARY 2005
97.02	GROUND SURFACE ELEVATION (m)
[81.47]	BEDROCK SURFACE ELEVATION (m)
(79.59)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

GROUND SURFACE ELEVATIONS AT 2021 BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

GROUND SURFACE ELEVATIONS FOR TEST HOLES AND 2012 AND 2016 BOREHOLES PROVIDED BY STANTEC GEOMATICS LIMITED.

SCALE: 1:1500

0 25 50 75 100m

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NO.	REVISIONS	DATE	INITIAL

MINTO COMMUNITIES

GEOTECHNICAL INVESTIGATION
ARCADIA STAGE 5 - CAMPEAU DRIVE

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:1500	Date: 02/2021
Drawn by: YA	Report No.: PG4933-1
Checked by: KP	Dwg. No.: PG4933-2
Approved by: FA	Revision No.:

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