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Geotechnical Investigation

Proposed Residential Development
The Meadows - Phase 4
Greenbank Road - Ottawa

Prepared For

Tamarack (Nepean) Corporation

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Report: PG3786-1-Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Tamarack (Nepean) Corporation to conduct a geotechnical investigation for Phase 4 of The Meadows residential development located adjacent to the future Greenbank Road, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

❏ determine the subsoil and groundwater conditions at this site by means of boreholes and relevant test holes completed as part of the previous geotechnical investigation.

❏ provide geotechnical recommendations for the design of the proposed development based on the results of the boreholes and other soil information available. These recommendations include permissible grade raises, long term settlements and other construction considerations which may affect its design.

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

Investigating the presence or potential presence of contamination on the proposed development was not part of the scope of work. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

It is understood that the current phase of the proposed development will consist of townhouses, residential dwellings with attached garages, associated driveways, local roadways and landscaping areas.

It is further understood 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 investigations was carried out on February 27, 2018 and on March 11 and 14, 2016. At that time, a total of five (5) test pits and six (6) boreholes were placed in a manner to provide general coverage of the subject site taking into consideration site features, underground utilities and existing test holes completed during the previous investigations. The location of the test holes are presented on Drawing PG3786-1 - 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 pits were completed using a rubber tired back-hoe at selected locations. The test hole procedure consisted of augering or excavated 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. Soil samples from the test pits were recovered from the side walls of the open excavation. All soil samples were visually inspected and initially classified on site. The auger, split-spoon and grab samples were placed in sealed plastic bags and the Shelby tubes were sealed at both ends on site. All samples were transported to the our laboratory for examination and classification. The depths at which the auger, split-spoon, grab and Shelby tube samples were recovered from the test holes are shown as AU, SS, G and TW, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

The thickness of the sensitive silty clay deposit was evaluated by a dynamic cone penetration testing (DCPT) completed at BH 4-16, BH 5-16 and BH 6-16. 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.

It should be noted that practical refusal to augering was encountered at BH 1-16 on an inferred boulder at a depth of 5 m below existing ground surface.

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

51 mm diameter PVC groundwater monitoring well was installed at BH1-16, BH3-16 and BH 5-16 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

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

The remainder of the boreholes (BH 2-16, BH 4-16 and BH 6-16) were instrumented with flexible standpipes to monitor the groundwater level subsequent to the completion of the sampling program.

The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations and ground surface elevations at the test hole locations completed during the current investigation were provided by Stantec Geomatics Ltd.. It is understood that the ground surface elevations are referenced to a geodetic datum.

The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG3786-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging.

A total of two (2) soil samples collected from the boreholes during the current investigation were submitted for grain size distribution analysis. The current grain size distribution analyses and relevant testing from previous investigations are also presented in Appendix 1.

A total of twenty-five (25) soil samples were submitted for moisture contents.

A total of three (3) Shelby tube samples collected from the boreholes during the current investigation were submitted for unidimensional consolidation. The results of the consolidation are presented on the Unidimensional Consolidation Test Results in Appendix 1 and are further discussed in Sections 4 and 5.

A total of three (3) representative soils samples were submitted for Atterberg limit testing during the current investigation. The results of the Atterberg testing are presented on the Atterberg Limit Results sheet presented in Appendix 1 and are further discussed in Sections 4 and 5.

All 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.

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is former agricultural land. The bulk of the current phase of the proposed development has been recently cleared of topsoil and peat which has been stockpiled in several piles across the site.

It should be further noted that a settlement surcharge monitoring program is currently underway within the west portion of Brambling Way which is located to the north of the northwest corner of the current phase of the residential development.

4.2 Subsurface Profile

Generally, the soil conditions encountered at the test hole locations consist of a thin layer of topsoil/organic layer followed by a loose to very loose silty sand overlying a sensitive silty clay deposit which in turn is overlying a compact to dense glacial till and/or a loose, brown silty fine sand.

Practical refusal to augering/DCPT was encountered between 5.0 to 9.3 m below existing ground surface at BH 1-16 and BH 4-16, respectively.

Based on available geological mapping, dolostone of the Oxford formation is present in this area with an overburden drift thickness ranging between 15 to 25 m.

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.

It should be noted that running sand was encountered within the silty sand deposit at BH 1-16, BH 5-16 and BH 6-16 at depths ranging between 2.8 to 5.0 m, 1.5 to 3.0 m and 1.4 and 2.5 m, respectively, during our sampling program.

The results of the five (5) soil samples which were submitted for grain size analysis from the test holes from the current and previous investigation are summarized in Table 1.

Table 1 - Grain Size Distribution

Test Hole	Sample	Gravel (%)	Sand (%)	Silt and Clay (%)
TP 3-18	G1	0.9	85.2	13.9
TP 4-18	G1	0.0	81.2	18.8
BH 3-16	TW4	0.0	53.1	46.9
BH 5-16	SS2	1.0	88.7	10.3
TP 12	G2	6.1	87.8	6.1

Silty Clay

Grey silty clay was encountered below the silty sand deposit at all test holes completed during current investigation extended to depths ranging between 2.7 and 9.3 m below existing ground surface. In situ shear vane field testing carried out in the grey silty clay yielded undrained shear strength values ranging between 19 and 82 kPa. These values are indicative of a soft to stiff consistency.

A total of three (3) silty clay samples collected at this site during the current investigation were subjected to unidimensional consolidation testing. Two (2) relevant unidimensional consolidation testing results completed during the previous investigation has been emended to the current geotechnical report. The results of the testing are presented in Appendix 1 and are discussed in Subsection 5.3.

The consolidation test results indicate that the silty clay is overconsolidated with overconsolidation ratios (OCR) for the tested samples varying between 1.5 and 2.7. The OCR is the ratio of the preconsolidation pressure to the effective pressure at the sample depth. This is further discussed in Subsection 5.3.

Four (4) silty clay samples were submitted for Atterberg Limits testing during the current investigation and one (1) Atterberg Limit testing result completed during the previous investigation has been provided. The tested materials vary across the site and are classified as inorganic clays of high plasticity (CH), inorganic clays of low plasticity (CL) or inorganic silts of high plasticity (MH). The results are summarized in Table 2 and presented on the Atterberg Limits results sheet in Appendix 1.

Table 2 - Summary of Atterberg Limits Tests

Sample	Moisture Content %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Classification
TP 1-18 G2	-	33	15	18	CL
TP 2-18 G2	-	24	15	9	CL
TP 5-18 G2	-	52	31	21	MH
BH 2-16 TW3	46.6	65	23	41	CH
BH 3B-09 - TW1	-	56	23	33	CH

4.3 Groundwater

Groundwater level readings were recorded on April 2, 2016 at the boreholes completed during the current geotechnical investigation. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1 and provided in Table 3. Groundwater levels are subject to seasonal fluctuations and therefore could vary during time of construction.

Table 3 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1-16 *	94.86	0.20	94.66	April 2, 2016
BH 2-16	94.10	0.20	93.90	April 2, 2016
BH 3-16 *	93.93	0.10 **	94.03	April 2, 2016
BH 4-16	94.10	0.26	93.84	April 2, 2016
BH 5-16 *	94.92	0.10	94.82	April 2, 2016
BH 6-16	94.78	0.10	94.68	April 2, 2016

Note:
* - Denotes boreholes instrumented with monitoring wells.

As part of the current geotechnical investigation, slug testing was performed at BH 1-16, BH 3-16 and BH 5-16. The slug testing was performed with a groundwater data logger at the base of the water column within the monitoring well and a barometric data logger adjacent to the well to provide atmospheric pressure correction to the data logger readings. A slug consisting of a 1 m long, 40 mm diameter aluminum rod fully submersed within the water column to provide an 'instantaneous' increase in head. The results of the slug testing are further discussed in Subsection 6.5 and the results of our testing are presented in the data sheets presented in Appendix 1.

5.0 Discussion

5.1 Geotechnical Assessment

It is anticipated that the proposed buildings will be supported by shallow footings placed over stiff to firm silty clay or very loose to loose silty sand bearing surface. However, due to the presence of the sensitive silty clay layer, the proposed development will be subjected to grade raise restrictions.

Permissible grade raise recommendations are discussed in Subsection 5.3 and recommended permissible grade raise areas are presented in Drawing PG3786-2-Permissible Grade Raise Areas - Housing in Appendix 2. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

It is anticipated that deep services will be completed through mainly OHSA Type 3 soils, which has the potential for basal heave. It is assumed that the excavations will be carried out within the confines of a fully-braced steel trench box or other acceptable shoring system designed by a qualified structural engineer to resist the design lateral earth pressures and potential basal heave issues.

Due to the shallow groundwater level observed, it is highly recommended that a dewatering program be completed as part of the deep service installation within the subject site. The dewatering program should consist of a series of well points designed and installed by a licensed contractor specializing in dewatering.

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.

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).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If excavated brown silty clay or silty sand, free of organics and deleterious materials, is to be used to build up the subgrade level for areas to be paved, it is recommended that the material be placed under dry conditions and in above freezing temperatures, compacted in thin lifts using a suitable compaction equipment for the lift thickness by making several passes and approved by the geotechnical consultant. Non-specified existing fill and 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.

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. 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)
Stiff Silty Clay	100	150
Compact Silty Sand	60	125
Firm Silty Clay	60	125

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.

Bearing resistance values for footing design should be determined on a per lot basis at the time of construction.

It should be noted that if the silty sand layer is noted to be in a loose state of compactness at the subgrade level. It is recommended to proof roll the silty sand layer under dry conditions. Additionally, the subgrade should be inspected by a geotechnical consultant 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

Consideration must be given to potential settlements which could occur due to the presence of the silty clay deposit and the combined loads from the proposed footings, any groundwater lowering effects, and grade raise fill. The foundation loads to be considered for the settlement case are the continuously applied loads which consist of the unfactored dead loads and the portion of the unfactored live load that is considered to be continuously applied. For dwellings, a minimum value of 50% of the live load is recommended by Paterson.

Generally, the potential long term settlement is evaluated based on the compressibility characteristics of the silty clay. These characteristics are estimated in the laboratory by conducting unidimensional consolidation tests on undisturbed soil samples collected using Shelby tubes in conjunction with a piston sampler. Three (3) site specific consolidation tests were conducted as part of the current investigation and two (2) relevant consolidation testing conducted during the previous investigation has been emended to the current report. The results of the consolidation tests from our investigation is presented in Table 5 and in 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 2-16	TW3	1.73	51.0	28.6	0.033	2.153	A
BH 3-16	TW5	4.17	94.0	49.5	0.029	1.966	A
BH 6-16	TW5	3.56	80.7	35.1	0.033	2.015	A
BH 3-19	TW5	4.01	67.0	44.0	0.025	1.720	P
BH 3B-09	TW1	2.84	95.0	35.0	0.018	1.002	G

* - Q - Quality assessment of sample - G: Good 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 1 m was assumed.

The potential post construction total and differential settlements are dependent on the position of the long term groundwater level when building 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 consolidation testing results and undrained shear strength values at the borehole locations, we have determined our permissible grade raise recommendations for the current phase of the proposed development. Our permissible grade raise recommendations for housing are presented in Drawing PG3786-2 - Permissible Grade Raise Areas - Housing in Appendix 2. It should be noted that our permissible grade raise recommendations for the roadways can be taken as the permissible grade raise values presented in Drawing PG3786-2 increased by 0.4 m at each area.

Based on the above discussion, several options could be considered to accommodate proposed grade raises with respect to our permissible grade raise recommendations, such as, the use of lightweight fill, which allow for raising the grade without adding a significant load to the underlying soils. Alternatively, it is possible to preload or surcharge the subject site in localized areas provided sufficient time is available to achieve the desired settlements.

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.

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

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Site Class E** 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

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed buildings, undisturbed native soil surface will be considered acceptable subgrade on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. 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-slab fill consist of 19 mm clear crushed stone.

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 - Driveways

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 a 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 a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system is provided.

Based on our groundwater level observations, a sub-floor drain system for the proposed buildings with basements is recommended. It is recommended that a geosock wrapped, 150 mm diameter perforated corrugated plastic pipe be placed below the floor slab across the building footprint. A sleeve through the footing should be provided to connect the sub-floor drain to the perimeter foundation drainage system. It is further recommended that the perimeter drainage system have a positive outlet to the storm sewer.

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

Excavations will be mostly through very loose to loose silty sand and sensitive grey silty clay. Above the groundwater level, for excavations to depths of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Flatter slopes could be required for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be used.

Based on observations at the test hole locations at the time of the field program and review of the recovered soil samples, the subsoil at this site is considered to be mainly a Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. 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.

It should be noted that the very loose to loose wet silty sand and sandy silt with clay is very sensitive when wet, and upon disturbance is prone to running below the water table unless it is completely supported before excavation procedures. Unsupported excavations below the water table, specifically within the loose silty sand should be cut back at 0.5H:1V or flatter.

As a result, it is recommended that a dewatering program be completed as part of the deep service installation within that subject site. The dewatering program should consist of a series of well points designed and installed by a licensed contractor specializing in dewatering.

The slope cross-sections recommended above are for temporary slopes. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. A minimum of 4 to 6 m setback should be considered from the excavation face depending on the excavation depth and soil consistency.

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.

Excavation Base Stability

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

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

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

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

$$FS_b = N_b s_u / \sigma_z$$

where:

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

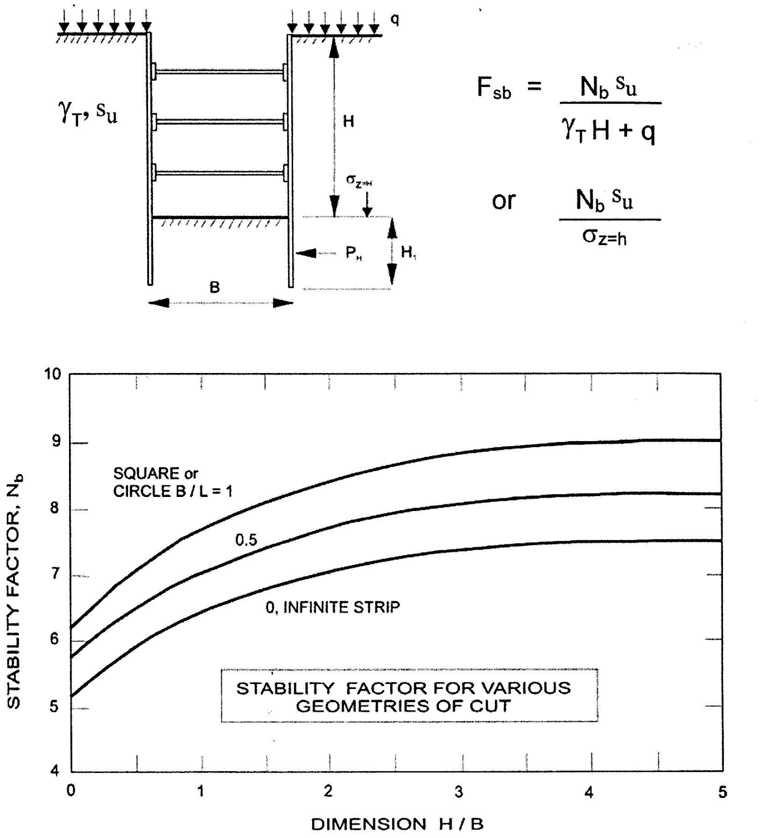


Figure 1 - Stability Factor for Various Geometries of Cut

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

6.4 Pipe Bedding and Backfill

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

It is expected that the invert level of the municipal services will be installed at or below the long term groundwater level within the very loose to loose silty sand to sandy silt deposit. As a result, it is recommended that a dewatering program should be implemented prior to construction to temporarily draw down the long term groundwater level during the construction phase. It is recommended that the dewatering program consisting of a serious of well points be designed and installed by a licensed contractor specialized in dewatering.

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.

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

Due to the relatively permeable silty sand to sandy silt deposit encountered within the shallow groundwater table within the subject site, it is anticipated that conventional pumping with open sumps will be difficult to control the groundwater influx through the sides of the temporary excavation. As a result, it is recommended that a dewatering specialist be consulted to review the most effective dewatering methods.

An existing MOECC permit to take water (PTTW) is in place and is valid until May 31, 2025. The daily volumes within the permit are to be considered as maximums for all phases combined. The MOECC PTTW Reference number is 2110-9X3RNE.

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.

Hydraulic Conductivity Analysis

Slug testing data was analyzed using the Hvorslev (1951) method and the results of our testing are presented in the data sheets and plotted in the figures presented in Appendix 1 with a summary presented in Table 9: Hydraulic Conductivity Test Results.

Monitoring wells were screened within various strata with BH1-16, BH3-16 and BH5-16. The hydraulic conductivity values show that BH5-16 intercepted a silty fine sand layer with higher potential groundwater inflows. Moderate potential was noted in the testing at BH3-16 and lower potential was noted in the screened interval within BH1-16.

Table 9: Hydraulic Conductivity Test Results

Well ID	**Depth of Well (mbgs)**	**Depth to Water Level (mbgs)**	**Test Type**	**Hydraulic Conductivity (m/sec)**
BH1-16	3.1	0.4	Falling Head	4.02E-07
BH3-16	6.1	0.1	Falling Head	5.59E-06
			Rising Head	2.52E-06
BH5-16	4.5	0.6	Falling Head	9.90E-05
			Rising Head	6.20E-05
			Falling Head	8.48E-05
			Rising Head	5.94E-05
Maximum				**9.90E-05**
Minimum				**4.02E-07**
Geometric Mean				**1.51E-05**

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. Sieve analysis testing was also completed on selected soil samples. The abovenoted 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.

The following grading plan prepared by IBI Group was reviewed to determine design underside of footing elevations for our tree planting soils review:

☐ The Meadows in Half Moon Bay, Phase 4, Grading Plan, Project No. 12054, Drawing No. 206, Revision No. 1 dated October 12, 2017.

Based on the results of our review, the two tree planting setback areas are present within the current phase of the proposed development. The two areas are detailed below and have been outlined in Drawing PG3786-3 - Tree Planting Setback Recommendations presented in Appendix 2.

Area 1 - No Tree Planting Setbacks

Based on the subsoil profile at the test hole locations, a silty sand deposit was encountered within 3.5 m of design finished grades. As a result, no tree planting restrictions are required for Area 1 illustrated on Drawing PG3786-3 - Tree Planting Setback Recommendations in Appendix 2.

Area 2 - Low/Medium Sensitivity Clay Soils

A low to medium sensitivity clay soil was encountered between design underside of footing elevations and 3.5 m below finished grade as per City Guidelines. Based on our Atterberg Limits test results, the modified plasticity limit generally does not exceed 40%. The following tree planting setbacks are recommended for Area 2. Large trees (mature height over 14 m) can be planted within Area 2 provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits may be reduced to 4.5 m for small (mature 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 m^3 of available soil volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

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

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

- ☐ Grading surround 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.

Swimming Pools

The in-situ soils are considered to be acceptable for swimming pools. Above ground swimming pools must be placed at least 5 m away from the residence foundation and neighbouring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer`s requirements.

Aboveground Hot Tubs

Additional grading around the hot tub should not exceed permissible grade raises. Otherwise, hot tub construction is considered routine, and can be constructed in accordance with the manufacturer’s specifications.

Installation of Decks or Additions

Additional grading around proposed deck or addition should not exceed permissible grade raises. Otherwise, standard construction practices are considered acceptable.

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 Tamarack (Nepean) Corporation 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.

LICENSED PROFESSIONAL ENGINEER
Mar. 9-2018
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Richard Groniger, C. Tech.

David J. Gilbert, P.Eng.

Report Distribution:

❑ Tamarack (Nepean) Corporation (3 copies)
❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

CONSOLIDATION TESTING RESULTS

ATTERBERG LIMITS’ TESTING RESULTS

GRAIN SIZE DISTRIBUTION SHEETS

SLUG TESTING DATA SHEETS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

DATUM Ground surface elevations were provided by Taggart Construction Ltd.

REMARKS

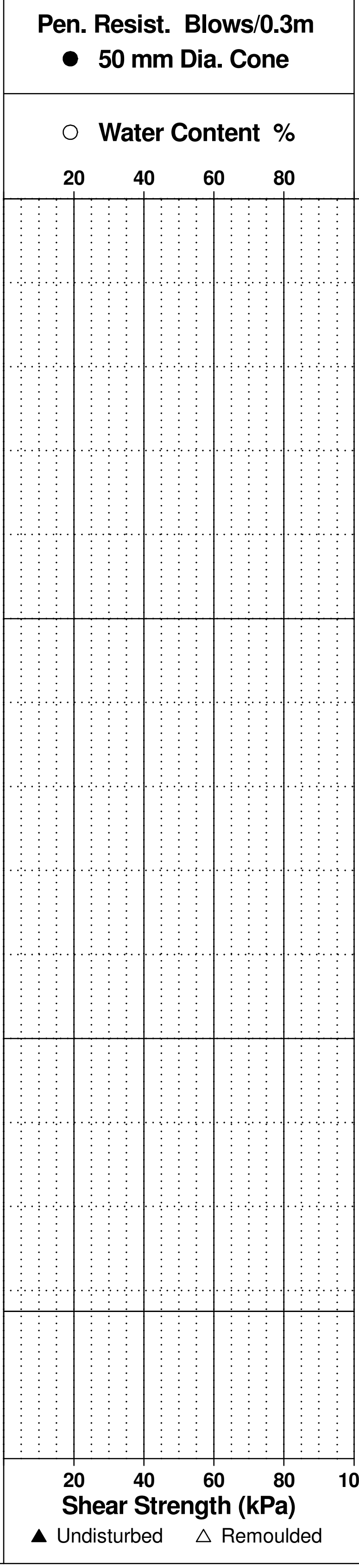
BORINGS BY Hydraulic Shovel

DATE February 27, 2018

FILE NO. PG3786

HOLE NO. TP 1-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.65
Loose, brown **SILTY SAND** with gravel, cobbles and boulders, trace clay						1	92.65
		G	1				
1.60							
Stiff to firm, grey **SILTY CLAY,** trace sand						2	91.65
		G	2				
2.65							
End of Test Pit (TP dry upon completion)							



Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

DATUM Ground surface elevations were provided by Taggart Construction Ltd.

REMARKS

BORINGS BY Hydraulic Shovel

DATE February 27, 2018

FILE NO. PG3786

HOLE NO. TP 2-18

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 %	Piezometer Construction
GROUND SURFACE						0	93.40		
Loose, brown **SILTY SAND,** trace clay		G	1						
0.70									
Stiff, brown **SILTY CLAY,** some sand		G	2			1	92.40		
1.20									
End of Test Pit (TP dry upon completion)									

Water Content %: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

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REMARKS

BORINGS BY Hydraulic Shovel

DATE February 27, 2018

FILE NO. PG3786

HOLE NO. TP 3-18

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	94.05		
Loose, brown **SILTY SAND to SANDY SILT**						1	93.05		
1.75		G	1						
End of Test Pit									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

DATUM Ground surface elevations were provided by Taggart Construction Ltd.

FILE NO. PG3786

REMARKS

HOLE NO. TP 4-18

BORINGS BY Hydraulic Shovel

DATE February 27, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	93.96		
Loose, brown **SILTY SAND,** trace clay									
						1	92.96		
		G	1						
1.30									
End of Test pit (Groundwater infiltration at 0.7m depth)									

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

DATUM Ground surface elevations were provided by Taggart Construction Ltd.

REMARKS

BORINGS BY Hydraulic Shovel

DATE February 27, 2018

FILE NO. PG3786

HOLE NO. TP 5-18

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 %	Piezometer Construction
GROUND SURFACE						0	95.23		
		G	1						
FILL: Brown sand and gravel, some silt, cobbles and boulders, trace clay									
						1	94.23		
1.62									
Stiff to very stiff, brown **SILTY CLAY,** trace sand		G	2						
						2	93.23		
2.30									
End of Test Pit									

Chart scale: 20 40 60 80 (top); 20 40 60 80 100 (bottom)

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

DATUM Ground surface elevations were provided by Stantec Geomatics Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 11, 2016

FILE NO. PG3786

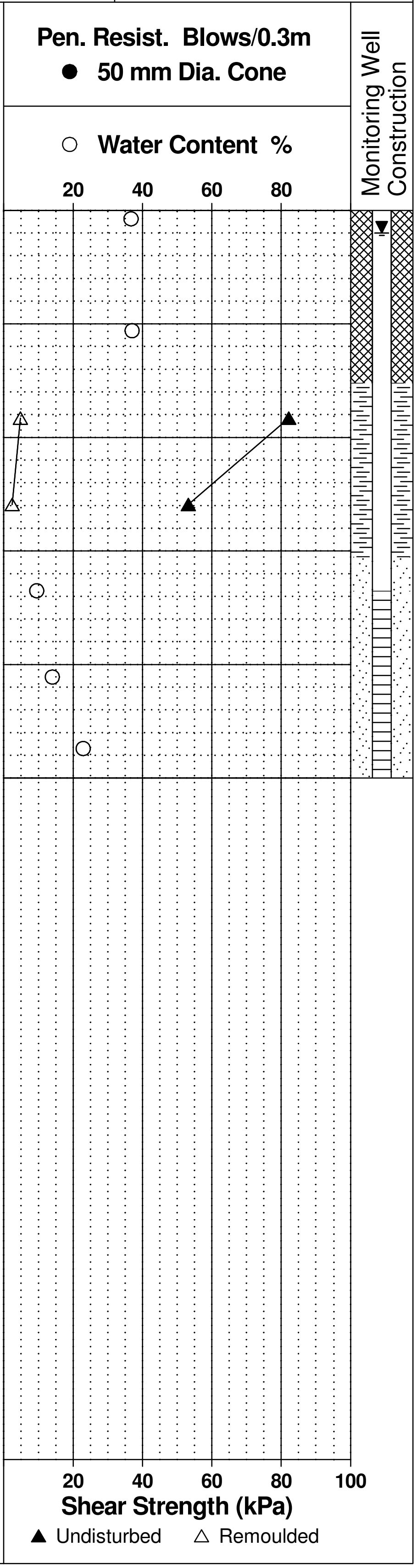
HOLE NO. BH 1-16

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
FILL: Brown silty sand with clay and topsoil — 0.30	AU	1			0	94.86
Stiff, brown **SILTY CLAY**	SS	2	100	3	1	93.86
- grey by 2.1m depth — 2.74					2	92.86
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles, trace boulders — 3.81	SS	3	33	9	3	91.86
Grey **SILTY FINE SAND,** trace gravel	SS	4	33	8	4	90.86
- running sand encountered between 3.8 and 5.0m depth — 5.00	SS	5	62	50+	5	89.86

End of Borehole

Practical refusal to augering at 5.00m depth

(GWL @ 0.20m-April 4, 2016)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

DATUM Ground surface elevations were provided by Stantec Geomatics Limited.

FILE NO. PG3786

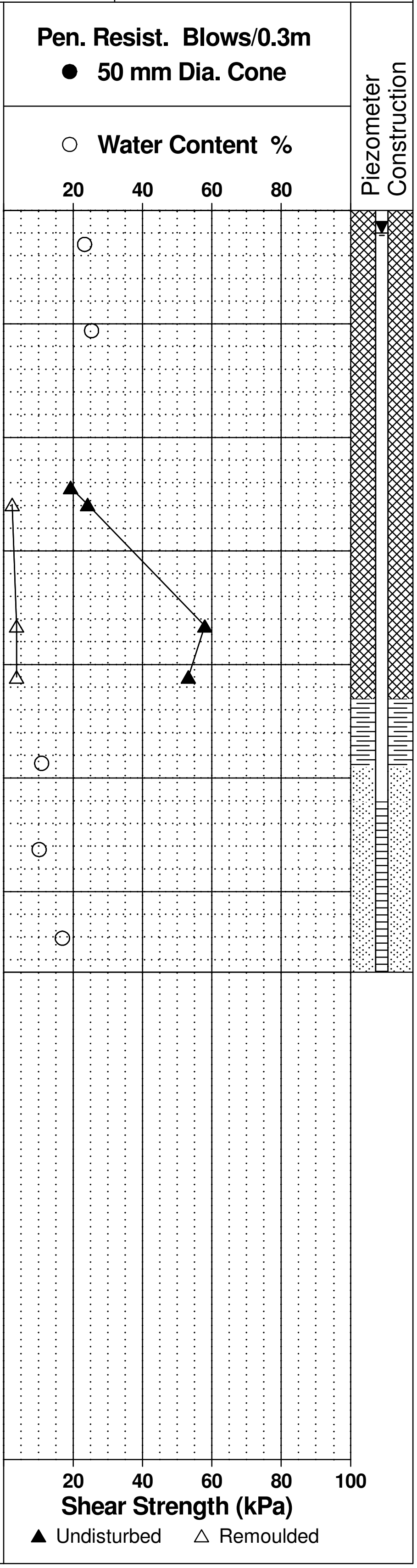
REMARKS

HOLE NO. BH 2-16

BORINGS BY CME 55 Power Auger

DATE March 11, 2016

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	94.10
Loose, light brown to brown **SILTY FINE SAND**		AU	1				
		SS	2	92	4	1	93.10
1.45							
Soft, grey **SILTY CLAY**		TW	3	100		2	92.10
		TW	4	92		3	91.10
- firm and grey by 3.35m depth						4	90.10
4.62							
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles, trace boulders		SS	5	88	21	5	89.10
		SS	6	71	47	6	88.10
		SS	7	83	35		
6.71							
End of Borehole							
(GWL @ 0.20m-April 4, 2016)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

DATUM Ground surface elevations were provided by Stantec Geomatics Limited.

FILE NO. PG3786

REMARKS

HOLE NO. BH 3-16

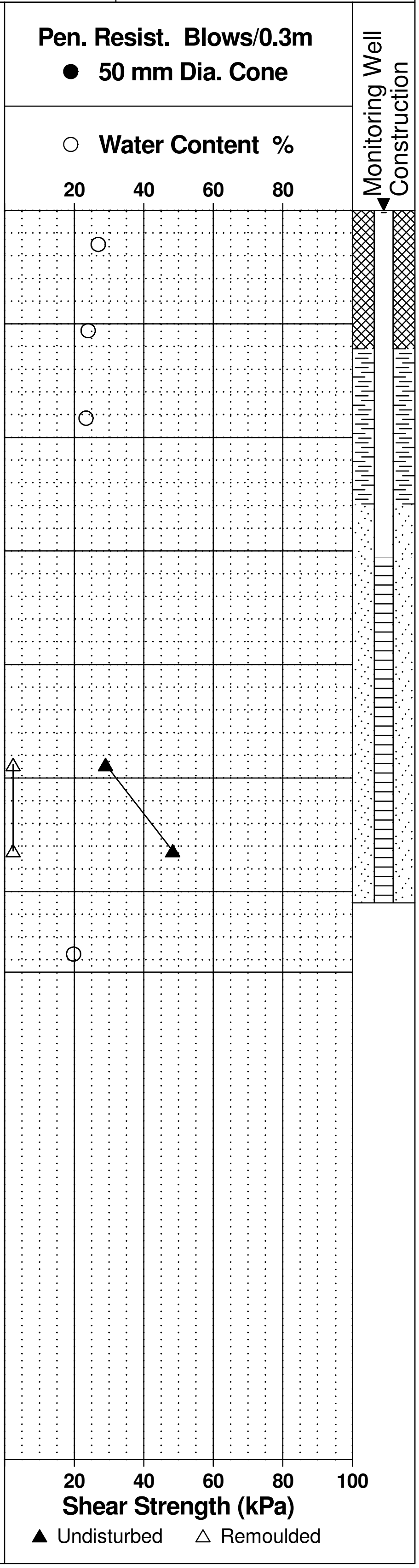
BORINGS BY CME 55 Power Auger

DATE March 11, 2016

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Topsoil with brown silty sand, trace gravel 0.30	AU	1		
Very loose, brown **SILTY FINE SAND,** trace clay 1.47	SS	2	96	2
Grey **SILTY FINE SAND to SANDY SILT,** trace clay	SS	3	71	2
	TW	4	96	
3.96				
Soft to firm, grey **SILTY CLAY**	TW	5	100	
	TW	6	100	
6.38	TW	7	100	
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles, trace boulders 6.71	SS	8	100	4
End of Borehole				

(GWL @ 0.1m above existing ground surface - April 4, 2016)

DEPTH (m)	ELEV. (m)
0	93.93
1	92.93
2	91.93
3	90.93
4	89.93
5	88.93
6	87.93



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

DATUM Ground surface elevations were provided by Stantec Geomatics Limited.

REMARKS

BORINGS BY CME 55 Power Auger

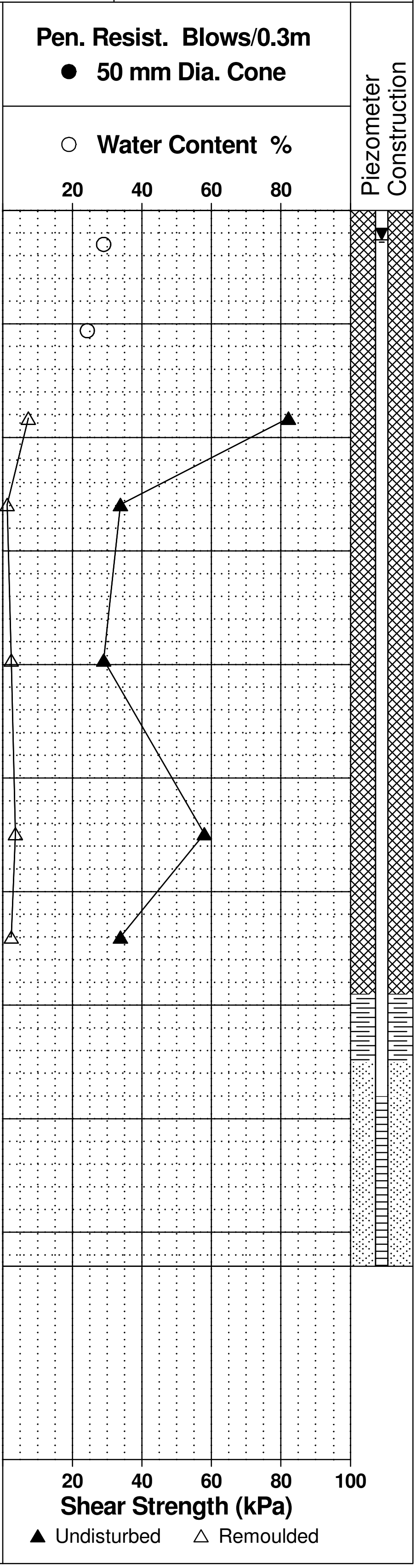
DATE March 14, 2016

FILE NO. PG3786

HOLE NO. BH 4-16

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL with silty sand	0.20	AU	1		
Brown **SILTY FINE SAND to CLAYEY SILT** - clay content increasing with depth	1.52	SS	2	83	3
Stiff to firm, grey **SILTY CLAY**, some sand seams	6.40	TW TW	3 4	100 77	
Dynamic Cone Penetration Test commenced at 6.40m depth. Cone pushed to 9.30m depth. Inferred **SILTY CLAY**	9.30				
End of Borehole Borehole terminated on inferred glacial till at 9.30m depth (GWL @ 0.26m-April 4, 2016)					

DEPTH (m)	ELEV. (m)
0	94.10
1	93.10
2	92.10
3	91.10
4	90.10
5	89.10
6	88.10
7	87.10
8	86.10
9	85.10



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

DATUM Ground surface elevations were provided by Stantec Geomatics Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 14, 2016

FILE NO. PG3786

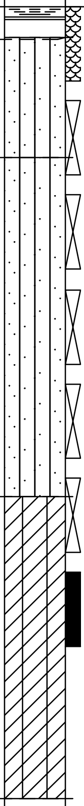
HOLE NO. BH 5-16

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
PEAT with organics	0.25
Loose, brown **SILTY SAND**	1.22
Loose to very loose, grey **SILTY FINE SAND** - running sand encountered between 1.5 and 3.0m depth	3.96
Firm, grey **SILTY CLAY**	6.40
Dynamic Cone Penetration Test commenced at 6.40m depth. Cone pushed to 7.3m depth.	7.67

End of Borehole

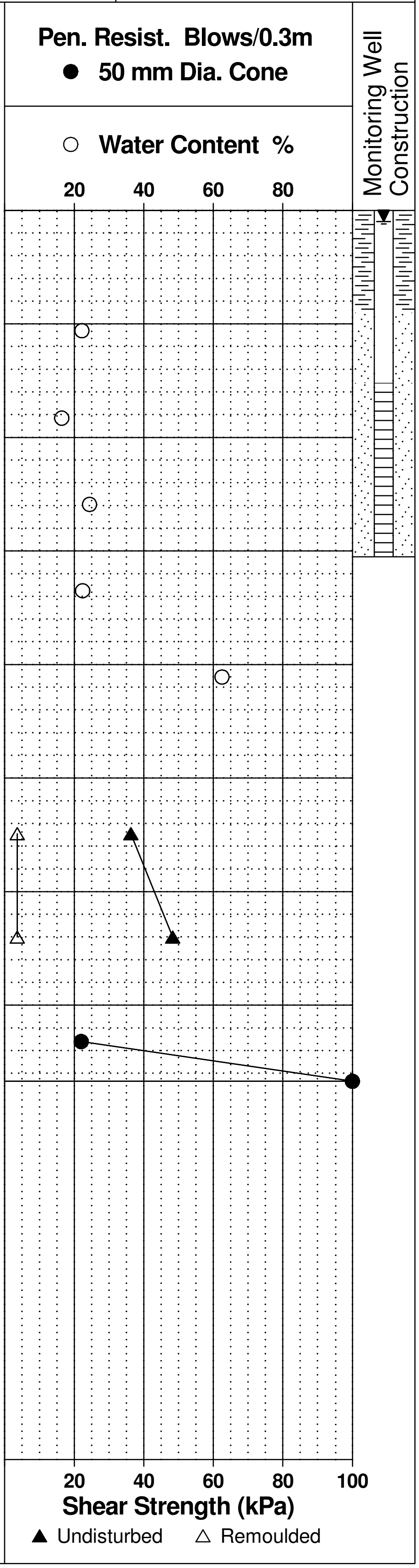
Practical DCPT refusal at 7.67m depth

(GWL @ 0.1m-April 4, 2016)



SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	54	6
SS	3	50	2
SS	4	42	3
SS	5	50	5
SS	6	100	1
TW	7	92	

DEPTH (m)	ELEV. (m)
0	94.92
1	93.92
2	92.92
3	91.92
4	90.92
5	89.92
6	88.92
7	87.92



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - The Meadows Phase 4
Greenbank Road, Ottawa, Ontario

DATUM Ground surface elevations were provided by Stantec Geomatics Limited.

FILE NO. **PG3786**

REMARKS

HOLE NO. **BH 6-16**

BORINGS BY CME 55 Power Auger

DATE March 14, 2016

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOIL with organics and silty sand	0.25
Loose, brown **SILTY SAND** - grey by 0.6m depth - running sand encountered between 1.4 and 2.5m depth	2.54
Soft to firm, grey **SILTY CLAY**	6.70
Dynamic Cone Penetration Test commenced at 6.70m depth	6.81

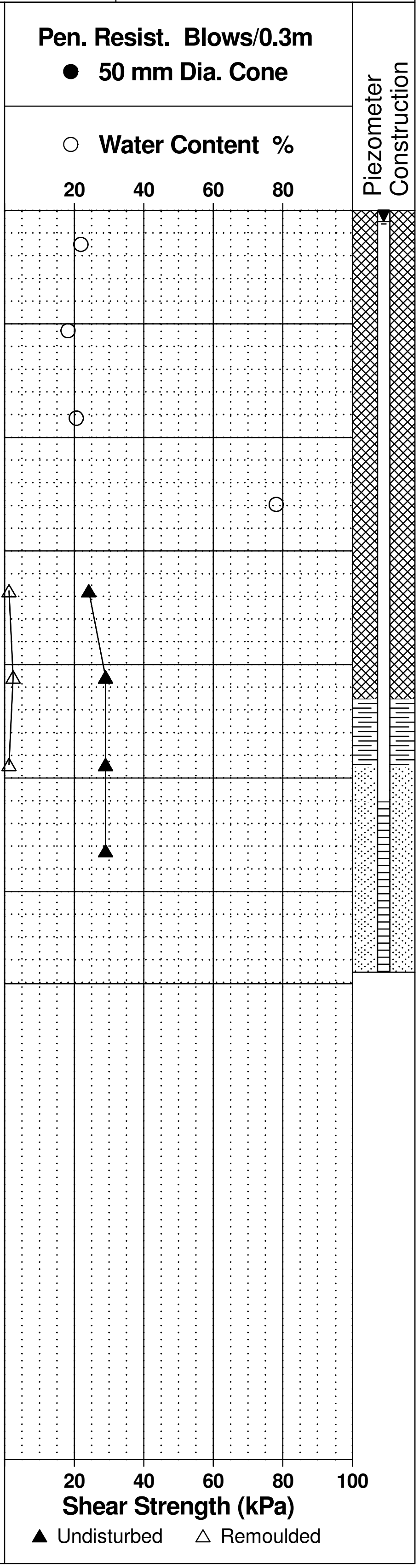
End of Borehole

Borehole terminated on inferred glacial till at 6.81m depth

(GWL @ 0.10m-April 4, 2016)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	71	4
SS	3	67	5
SS	4	67	2
TW	5	100	
TW	6	100	

DEPTH (m)	ELEV. (m)
0	94.78
1	93.78
2	92.78
3	91.78
4	90.78
5	89.78
6	88.78



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Residential Development-Greenbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Taggart Group of Companies

REMARKS

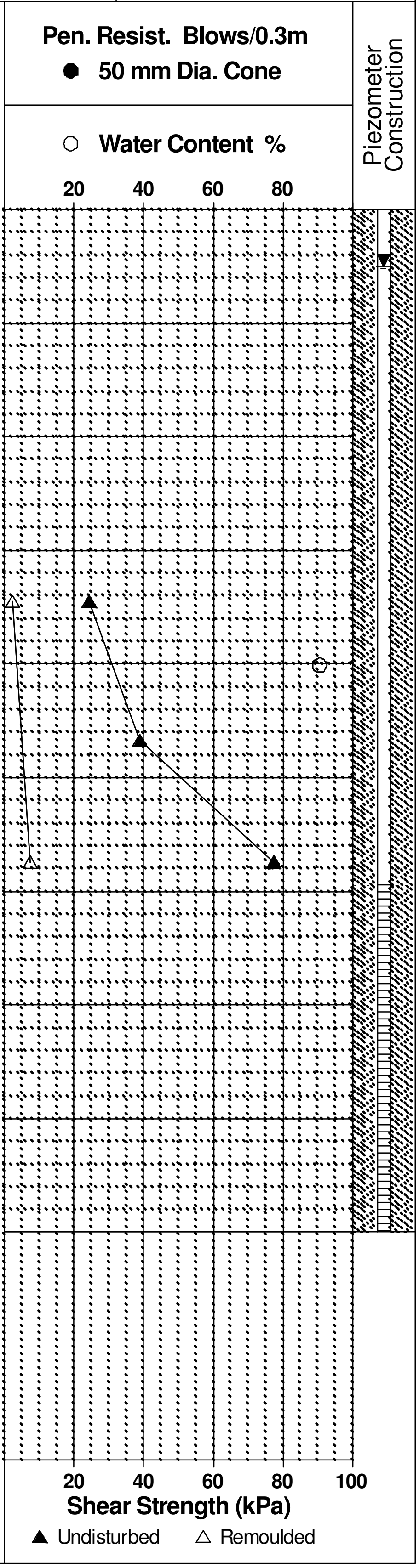
BORINGS BY CME 55 Power Auger

DATE 2 Mar 09

FILE NO. PG0214

HOLE NO. BH 3-09

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	93.33
TOPSOIL 0.08	AU	1				
CLAYEY SILT, some sand. High organic content in upper 600mm.	SS	2	58	2	1	92.33
	SS	3	83	2		
2.13					2	91.33
Firm to stiff, grey **SILTY CLAY**	SS	4	100	1		
					3	90.33
	TW	5	88		4	89.33
	SS	6	100	2	5	88.33
5.72						
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders					6	87.33
	SS	7	42	7		
	SS	8	25	11	7	86.33
	SS	9	0	50+		
					8	85.33
	SS	10	4	52		
8.99						
End of Borehole						
(GWL @ 0.50m-Mar. 11/09)						



patersongroup **Consulting Engineers**

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

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Residential Development-Greenbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Taggart Group of Companies

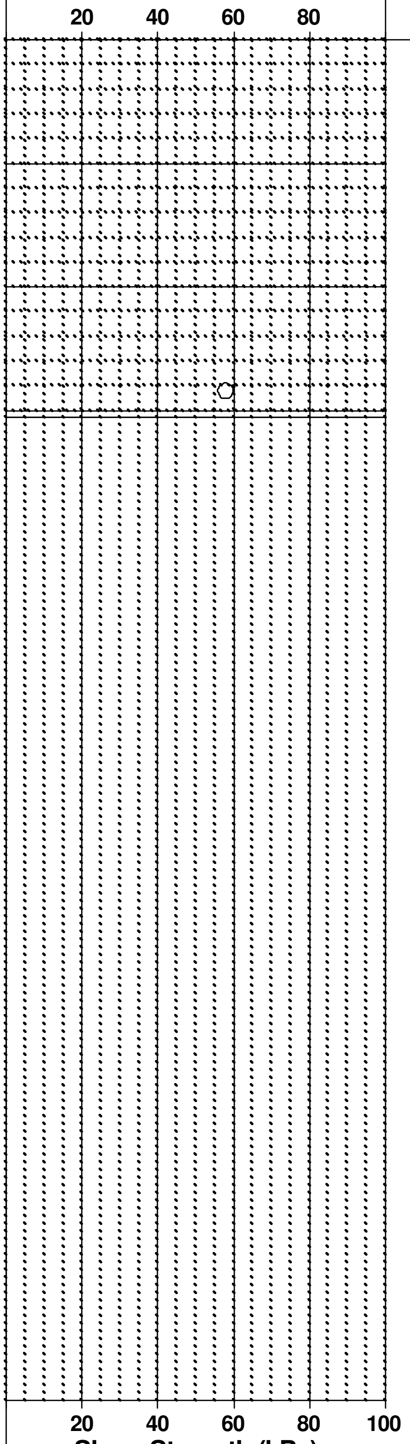
FILE NO. **PG0214**

REMARKS

HOLE NO. **BH 3B-09**

BORINGS BY CME 55 Power Auger **DATE** 3 Mar 09

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 %	Piezometer Construction
GROUND SURFACE						0	93.33		
OVERBURDEN						1	92.33		
						2	91.33		
2.44									
SILTY CLAY	▨	TW	1						
3.05						3	90.33		
End of Borehole									



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Part 1, Lot 10 and Part 1, Lot 9, Concession 3
Ottawa (Nepean), Ontario

DATUM Ground surface elevations provided by Webster and Simmonds Surveying Ltd.

REMARKS

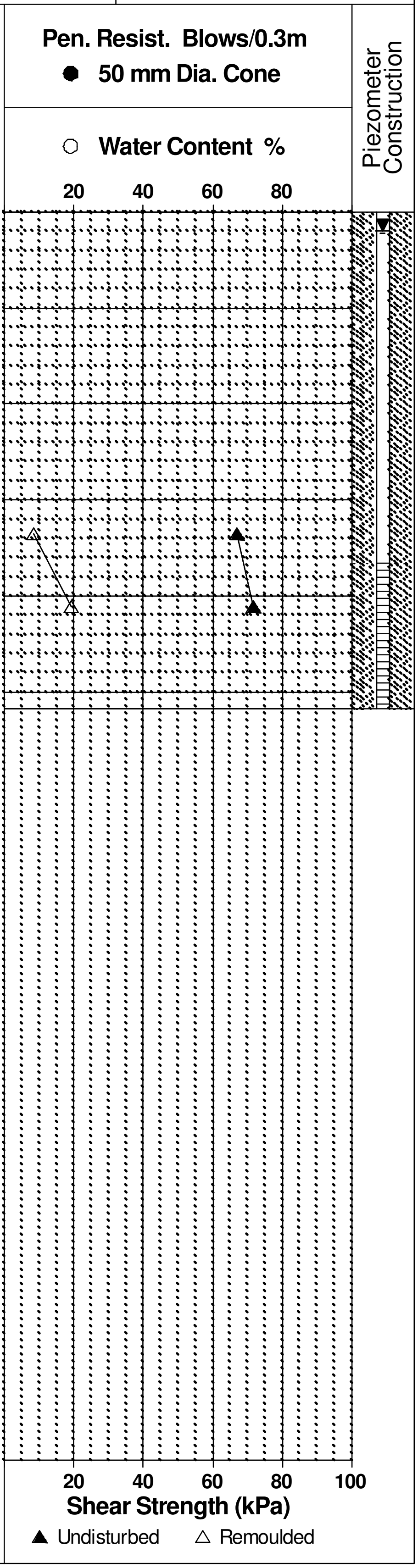
BORINGS BY CME 55 Power Auger

DATE 22 Apr 04

FILE NO. PG0214

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.32
TOPSOIL 0.30							
Brown **CLAYEY SILT**, some sand seams 1.37		SS	1	100	3	1	92.32
Stiff, grey-brown **CLAYEY SILT** to **SILTY CLAY**, some sand seams		SS	2	100	2	2	91.32
		SS	3	100	1	3	90.32
4.57						4	89.32
GLACIAL TILL: Stiff, grey clayey silt with sand, gravel, cobbles and boulders 5.18		SS	4	100	5	5	88.32
End of Borehole							
(GWL @ 0.20m-May 1/04)							



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

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Part 1, Lot 10 and Part 1, Lot 9, Concession 3
Ottawa (Nepean), Ontario

DATUM Ground surface elevations provided by Webster and Simmonds Surveying Ltd.

REMARKS

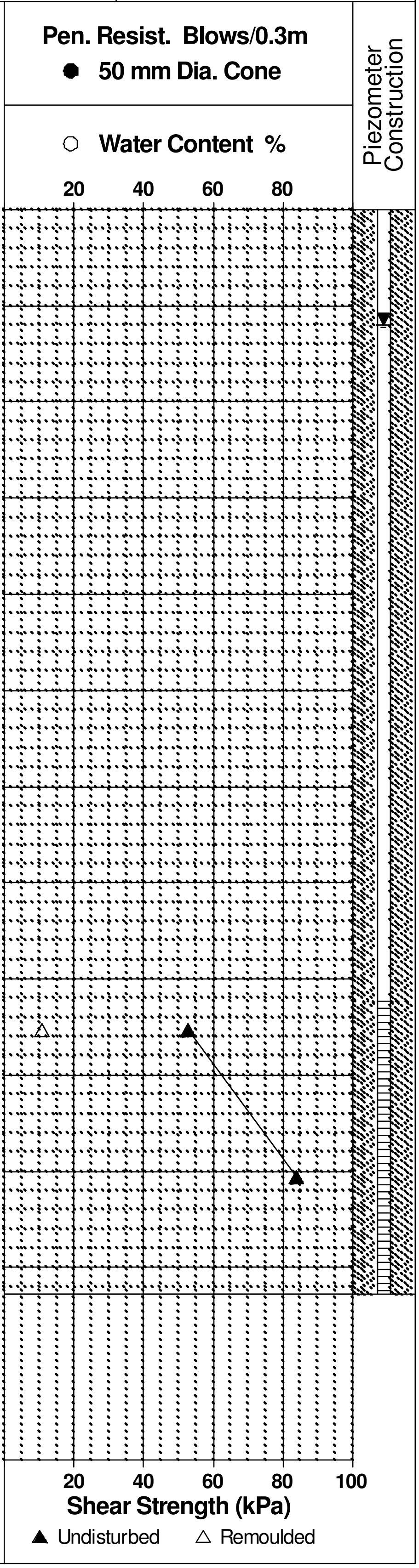
BORINGS BY CME 55 Power Auger

DATE 22 Apr 04

FILE NO. PG0214

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	94.97
TOPSOIL 0.36						
Stiff, brown **CLAYEY SAND**	SS	1	75	4	1	93.97
- grey by 1.5m depth	SS	2	100	W	2	92.97
	SS	3	71	W	3	91.97
3.80	SS	4	100	W		
Stiff, grey **CLAYEY SAND** to **SANDY SILT** 4.57	SS	5	100	1	4	90.97
Very loose, grey fine to medium **SAND** 5.60	SS	6	100	W	5	89.97
					6	88.97
Stiff, grey **SILTY CLAY**	SS	7	100	W	7	87.97
	SS	8	100	W	8	86.97
9.75	SS	9	100	1	9	85.97
GLACIAL TILL: Loose, grey silty sand with gravel, cobbles and boulders					10	84.97
11.28	SS	10	50	7	11	83.97
End of Borehole (GWL @ 1.20m-May 1/04)						



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Residential Development-Greenbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Taggart Group of Companies

FILE NO. **PG0214**

REMARKS

HOLE NO. **TP 9-09**

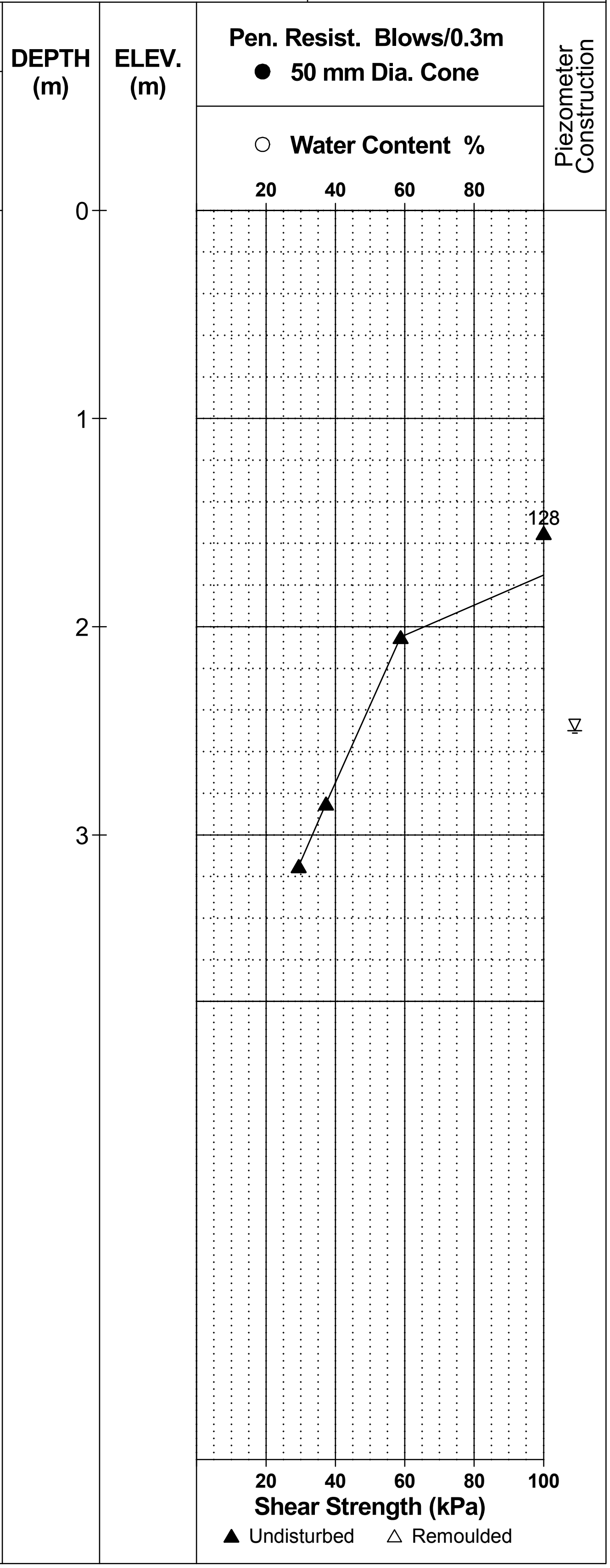
BORINGS BY Hydraulic Shovel

DATE December 17, 2009

SOIL DESCRIPTION	STRATA PLOT	SAMPLE			
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.35					
Brown fine to coarse **SAND** 0.90					
Very stiff to stiff, brown **SILTY CLAY** - firm and grey by 2.4m depth 3.30					
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders 3.60					
GLACIAL TILL: Very dense, grey sandy silt to silty sand with gravel, cobbles, boulders, trace clay 3.80					

End of Test Pit

(GWL @ 2.5m depth based on field observations)



patersongroup Consulting Engineers
28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Residential Development-Greenbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Taggart Group of Companies

REMARKS

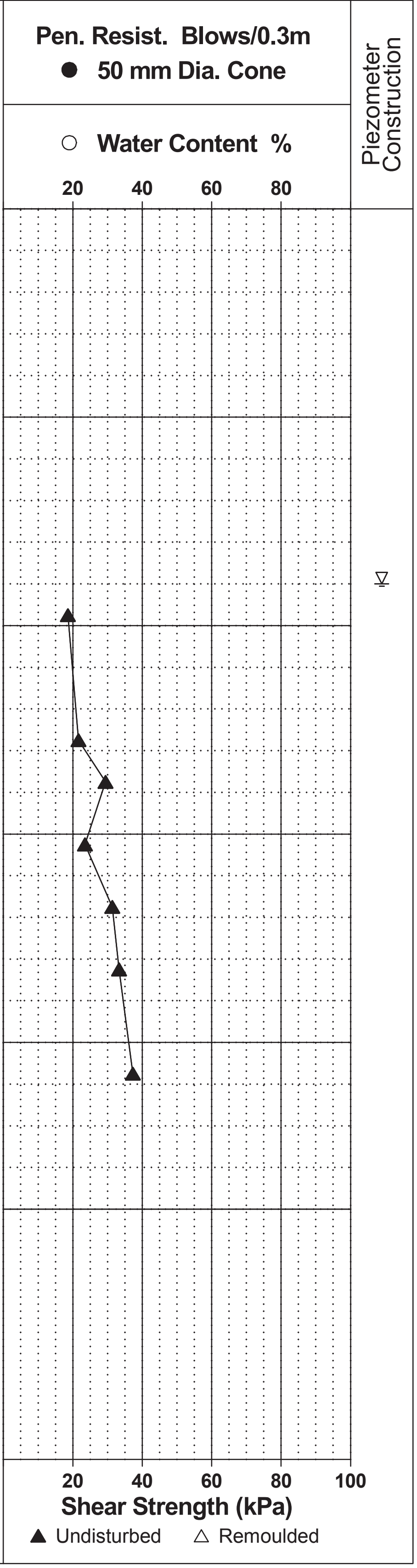
BORINGS BY Hydraulic Shovel

DATE December 17, 2009

FILE NO. PG0214

HOLE NO. TP10-09

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
TOPSOIL — 0.50							
Red-brown **SILTY CLAY** with sand — 0.85							
Brown **SILTY SAND** — 1.30						1	
Soft to firm, grey-brown **SILTY CLAY** with sand - grey by 1.9m depth — 4.80						2 3 4	
End of Test Pit (GWL @ 1.8m depth based on field observations)							



patersongroup **Consulting Engineers**

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

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Residential Development-Greenbank Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Taggart Group of Companies

REMARKS

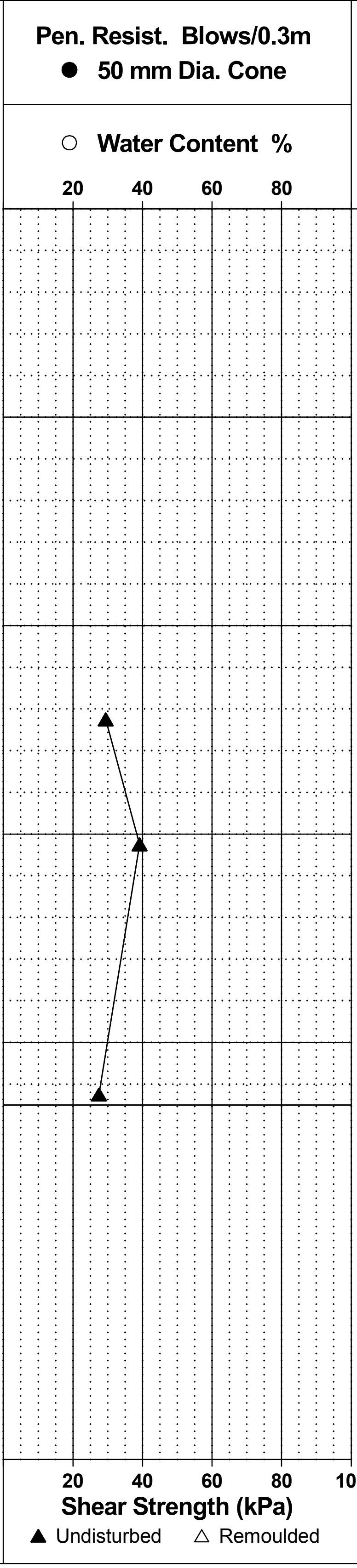
BORINGS BY Hydraulic Shovel

DATE December 17, 2009

FILE NO. **PG0214**

HOLE NO. **TP11-09**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0		
TOPSOIL 0.30	■							
Brown **SILTY CLAY** with sand 0.70	▨							
Brown **SILTY SAND** with clay and clay seams 1.50						1		∇
Soft to firm, grey **SILTY CLAY** with sand seams 4.30	▨					2 3 4		
End of Test Pit (GWL @ 1.5m depth based on field observations)								



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Francis Lands-Cambrian Road @ Greenbank Road
Ottawa (Nepean), Ontario

DATUM

REMARKS

BORINGS BY Backhoe

DATE 1 Apr 04

FILE NO. PG0177

HOLE NO. TP11

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0			
TOPSOIL 0.40	■								
Very stiff, grey-brown **SILTY CLAY**, trace roots 1.80	▨					1			∇
GLACIAL TILL: Grey sandy silt with gravel, cobbles and boulders 2.50	^^^					2			
End of Test Pit (Open hole GWL @ 1.6m depth)									

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

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Francis Lands-Cambrian Road @ Greenbank Road
Ottawa (Nepean), Ontario

DATUM

REMARKS

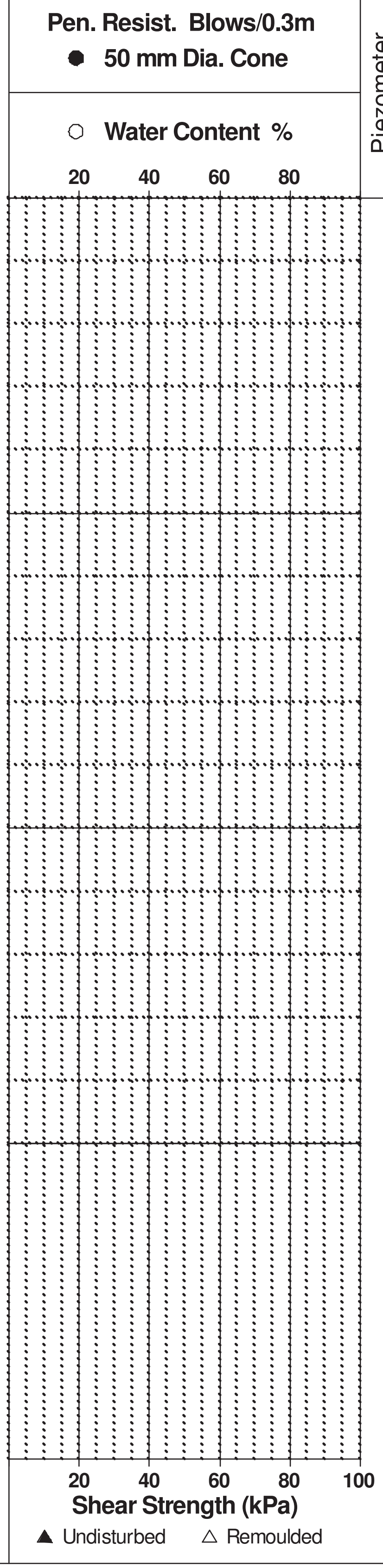
BORINGS BY Backhoe

DATE 1 Apr 04

FILE NO. PG0177

HOLE NO. TP12

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0		
TOPSOIL 0.40								
Reddish brown fine to medium **SAND**		G	1					
- coarse sand with some gravel and shells by 1.0m depth						1		∇
		G	2					
						2		
3.00						3		
End of Test Pit (Open hole GWL @ 1.0m depth)								



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

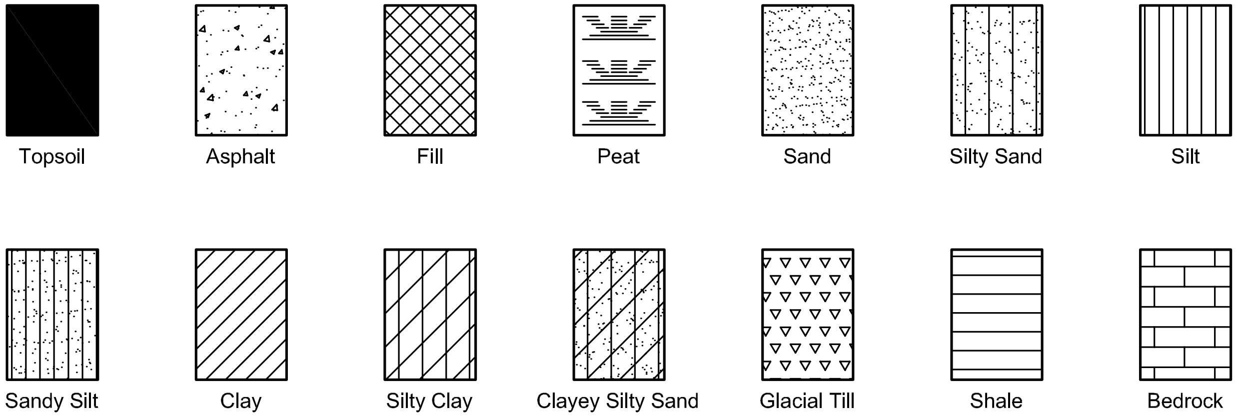
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

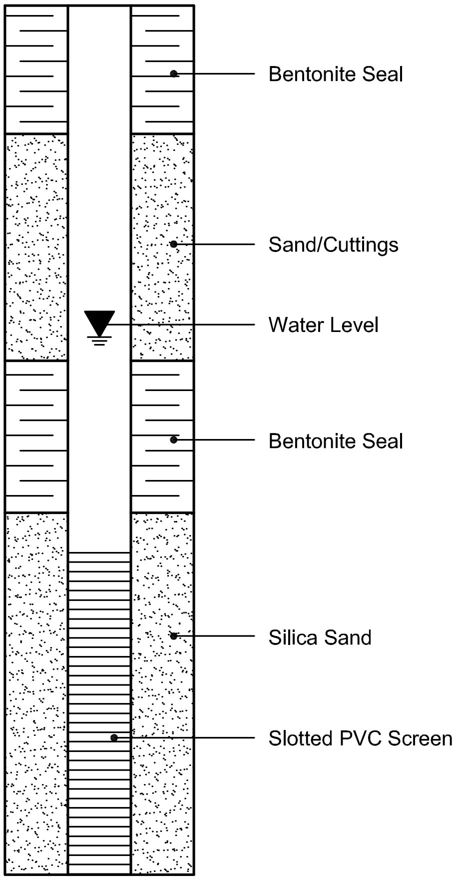
SYMBOLS AND TERMS (continued)

STRATA PLOT

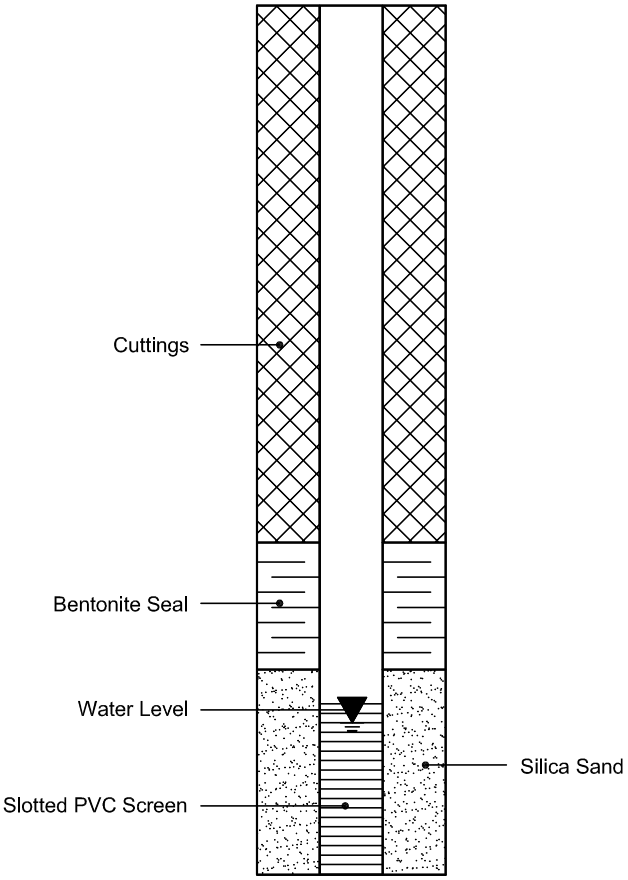


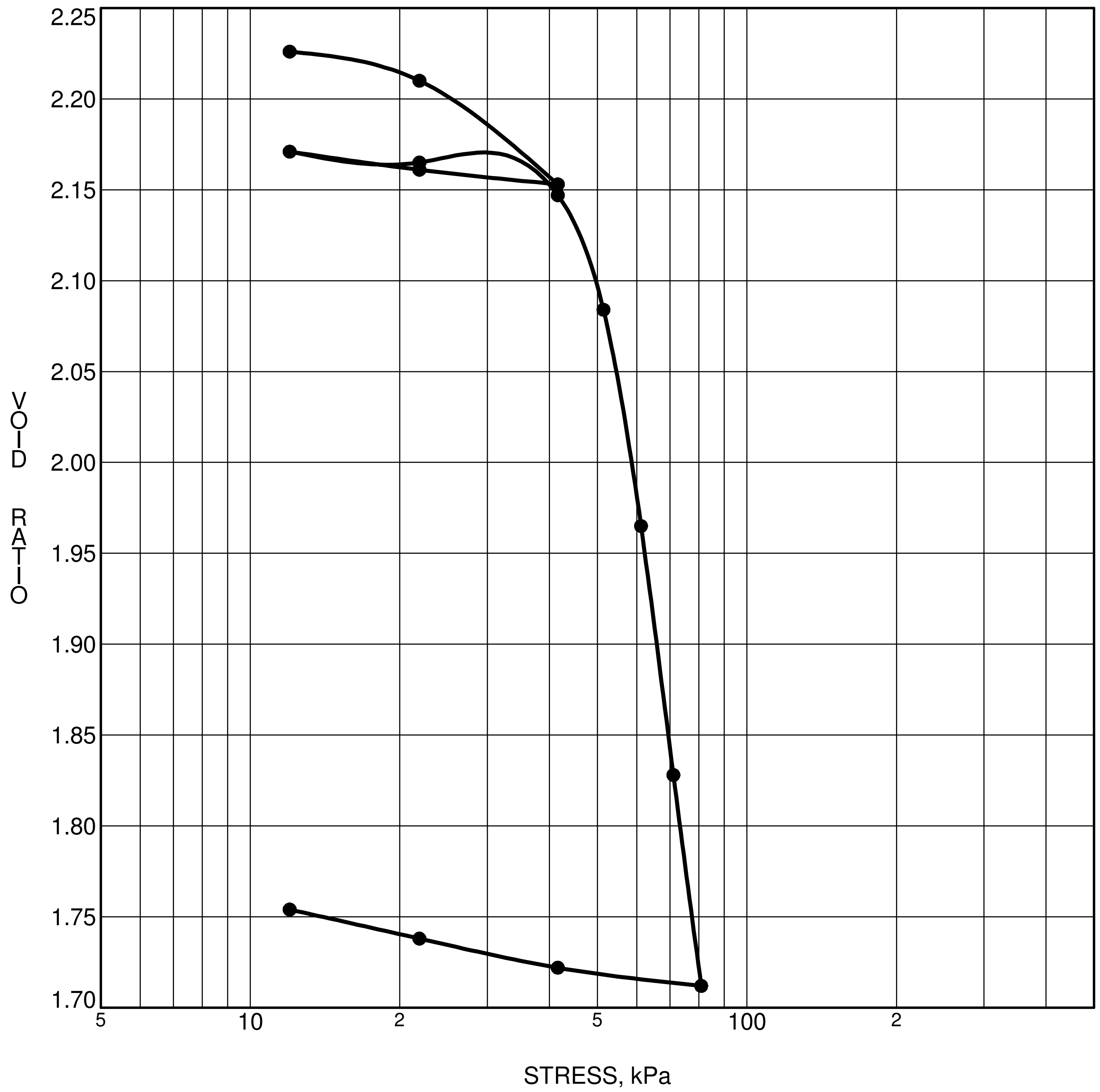
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 2-16**	p'o	**28.6** kPa	Ccr	**0.033**
Sample No.	**TW 3**	p'c	**51** kPa	Cc	**2.153**
Sample Depth	**1.73** m	OC Ratio	**1.8**	Wo	**82.0** %
Sample Elev.	**92.37** m	Void Ratio	**2.255**	Unit Wt.	**15.1** kN/m ^{3}

CLIENT **Tamarack (Nepean) Corporation**

FILE NO. **PG3786**

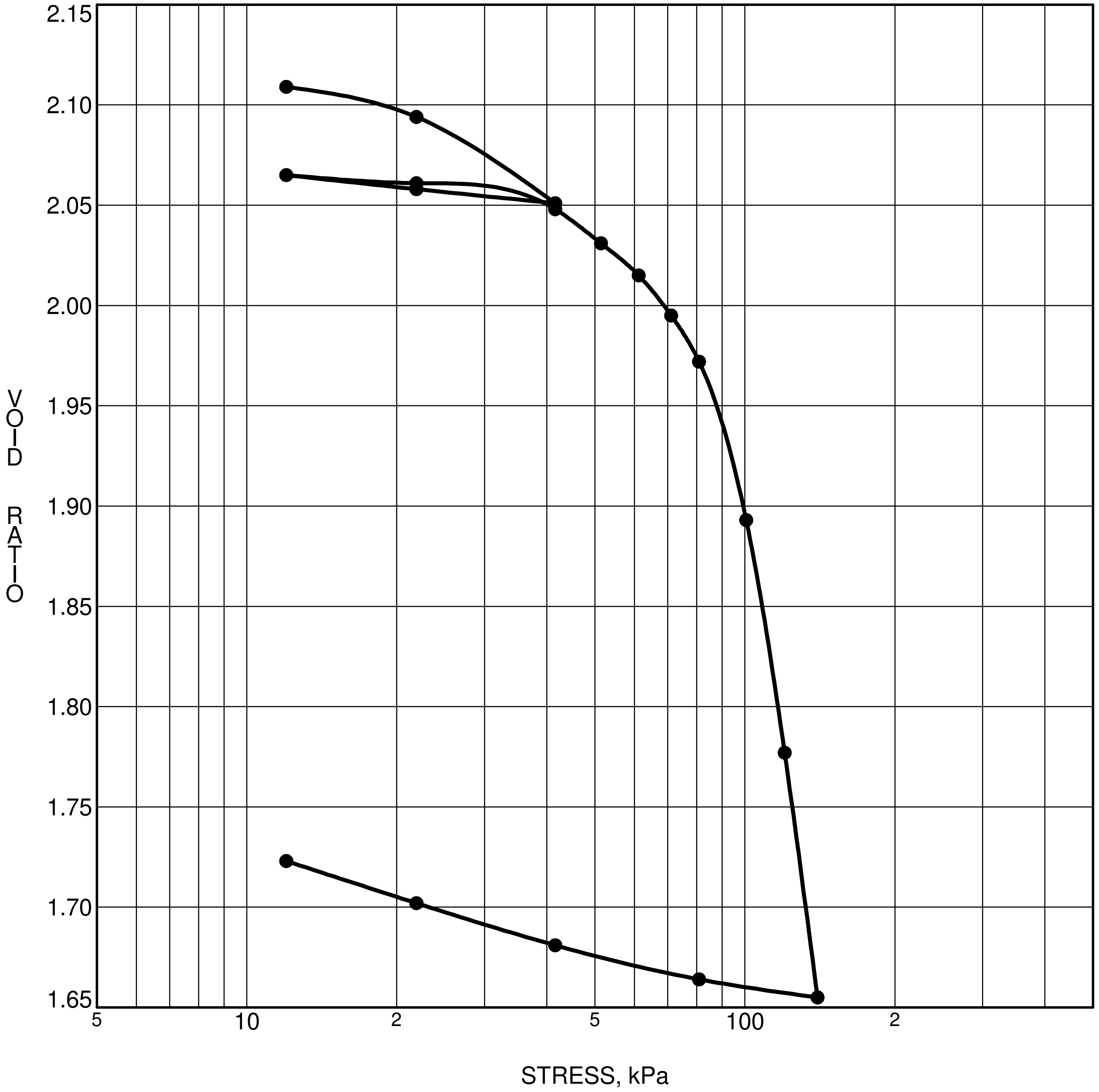
PROJECT **Geotechnical Investigation - Prop. Residential Development - The Meadows Phase 4**

DATE **01/04/2016**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 3-16**	p'o	**49.5** kPa	Ccr	**0.029**
Sample No.	**TW 5**	p'c	**94** kPa	Cc	**1.966**
Sample Depth	**4.17** m	OC Ratio	**1.9**	Wo	**77.8** %
Sample Elev.	**89.76** m	Void Ratio	**2.138**	Unit Wt.	**15.3** kN/m ^{3}

CLIENT **Tamarack (Nepean) Corporation**

FILE NO. **PG3786**

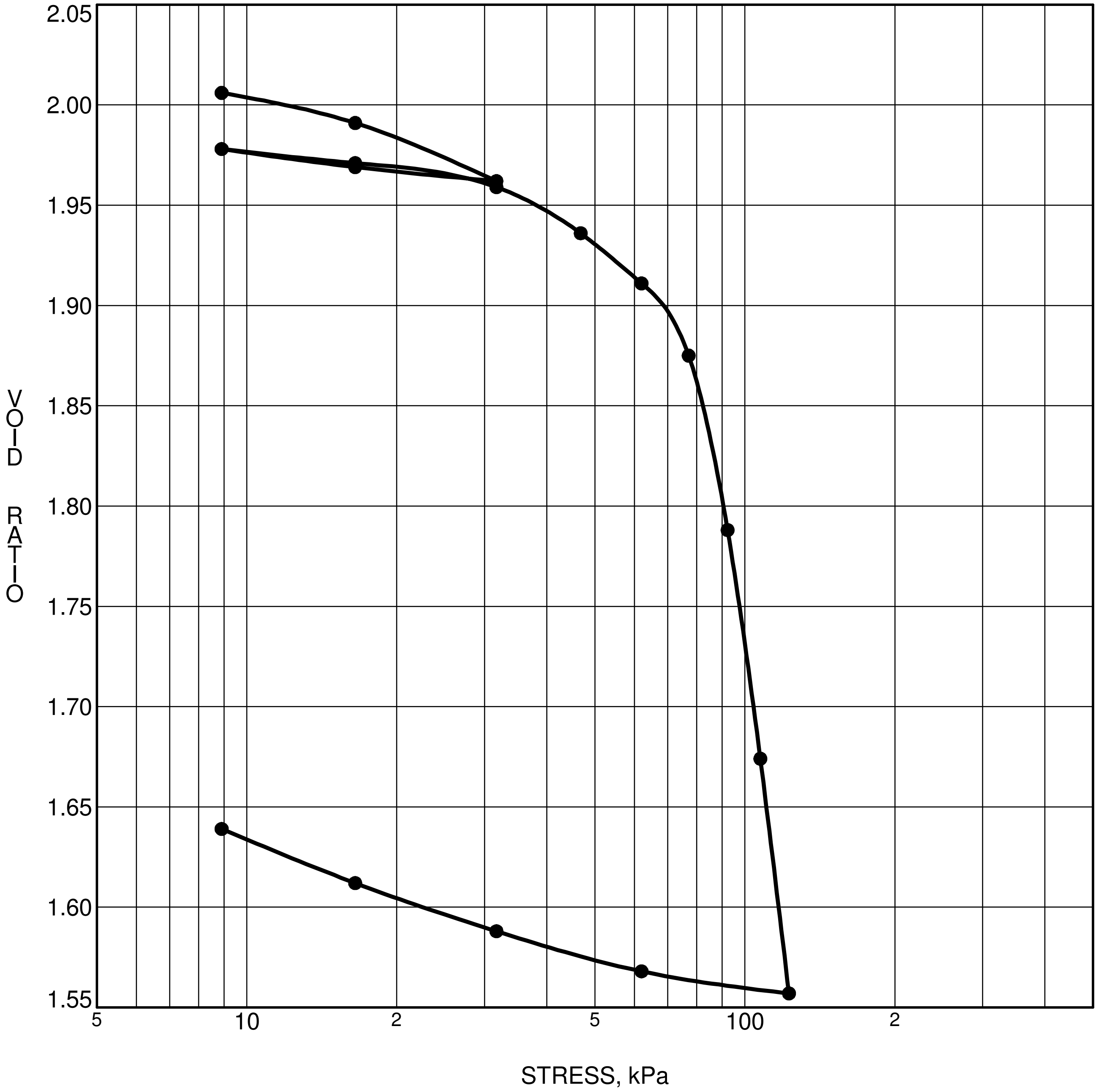
PROJECT **Geotechnical Investigation - Prop. Residential Development - The Meadows Phase 4**

DATE **31/03/2016**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 6-16**	p'_o	**35.1** kPa	Ccr	**0.033**
Sample No.	**TW 5**	p'_c	**80.7** kPa	Cc	**2.015**
Sample Depth	**3.56** m	OC Ratio	**2.3**	Wo	**73.2** %
Sample Elev.	**91.22** m	Void Ratio	**2.012**	Unit Wt.	**15.5** kN/m ^{3}

CLIENT **Tamarack (Nepean) Corporation**

FILE NO. **PG3786**

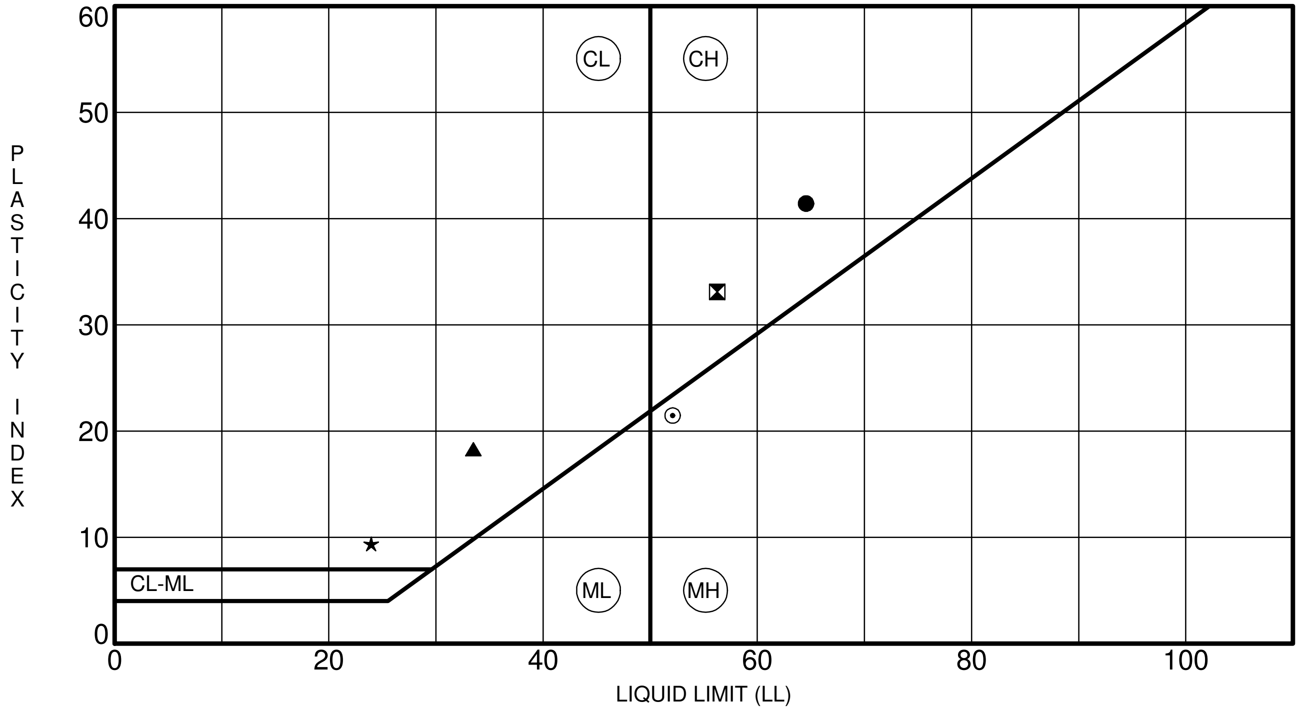
PROJECT **Geotechnical Investigation - Prop. Residential Development - The Meadows Phase 4**

DATE **01/04/2016**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 2-16 TW 3**	**65**	**23**	**41**		**CH - Inorganic clays of high plasticity**
⧗	**BH 3B-09 TW 1**	**56**	**23**	**33**		**CH - Inorganic clays of high plasticity**
▲	**TP 1-18 G2**	**33**	**15**	**18**		**CL - Inorganic clays of low plasticity**
★	**TP 2-18 G2**	**24**	**15**	**9**		**CL - Inorganic clays of low plasticity**
⊙	**TP 5-18 G2**	**52**	**31**	**21**		**MH - Inorganic silts of high plasticity**

CLIENT **Tamarack (Nepean) Corporation**

PROJECT **Geotechnical Investigation - Prop. Residential Development - The Meadows Phase 4**

FILE NO. **PG3786**

DATE **27 Feb 18**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

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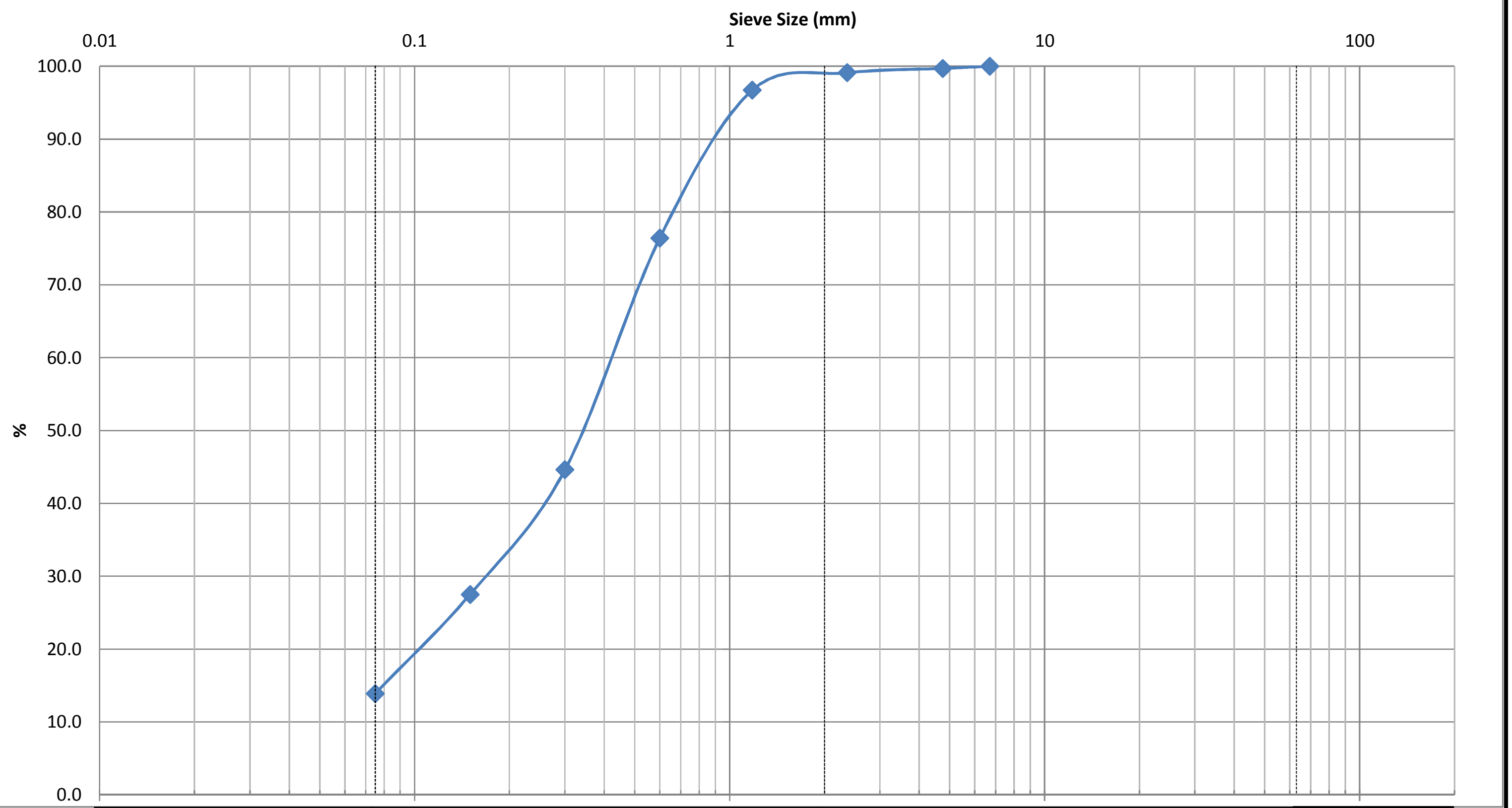
SIEVE ANALYSIS
ASTM C136

CLIENT:	Tamarack Homes	DESCRIPTION:	Aggregate	FILE NO:	PG3786
CONTRACT NO.:	-	SPECIFICATION:	**Silty Fine Sand**	LAB NO:	98978
PROJECT:	The Meadows Phase 4	INTENDED USE:	-	DATE RECEIVED:	6-Mar-18
		PIT OR QUARRY:	-	DATE TESTED:	7-Mar-18
DATE SAMPLED:	Nathan	SOURCE LOCATION:	TP3 - 18 G1	DATE REPORTED:	7-Mar-18
SAMPLED BY:	43158	SAMPLE LOCATION:	1.6 to 1.7	TESTED BY:	E.B/D.B



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	**Soil Classification**				**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
									1.19	6.5
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	7.8	0.42	0.18	0.065	0.9	85.2	13.9			

Comments	

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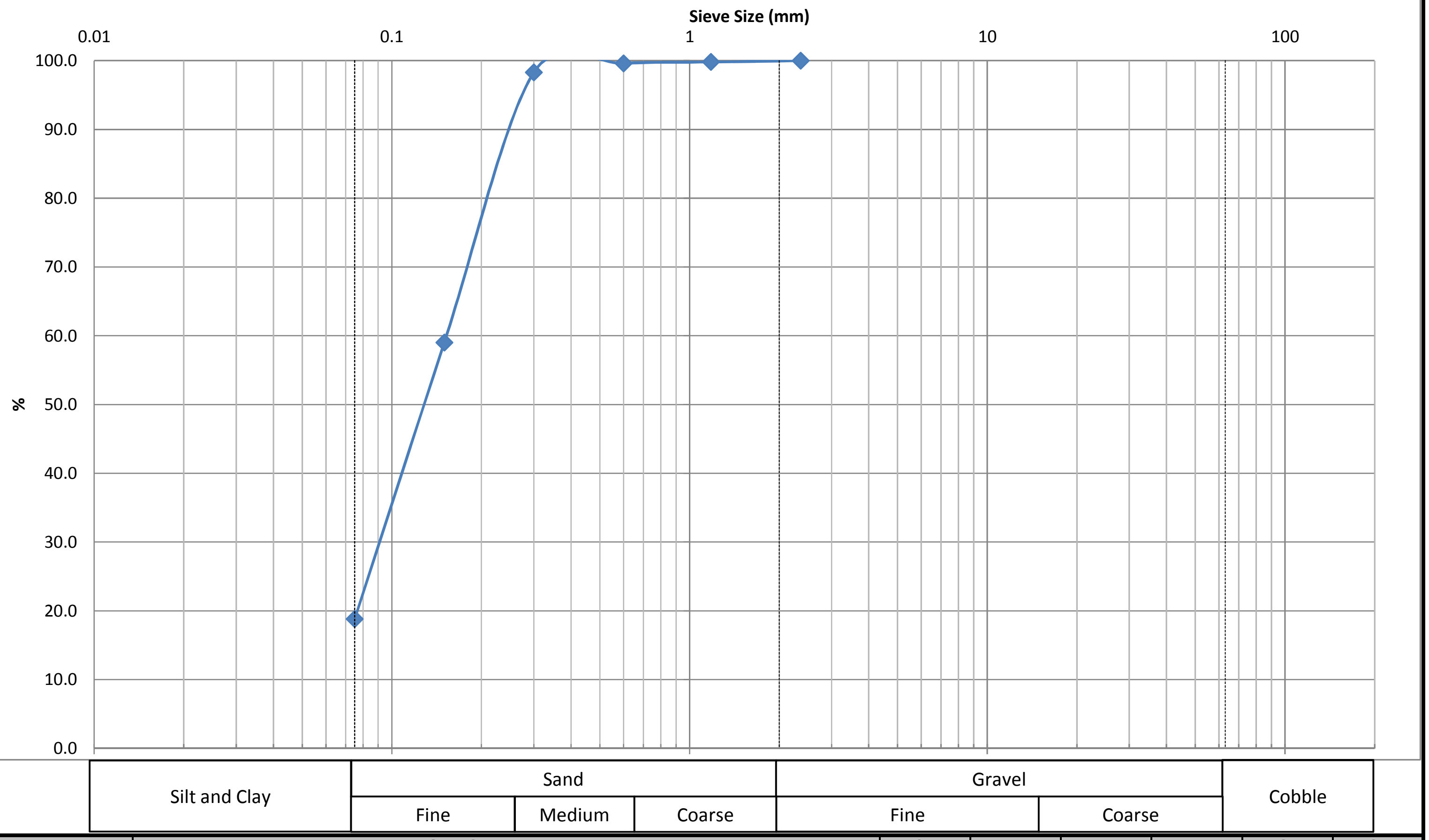
consulting engineers

SIEVE ANALYSIS
ASTM C136

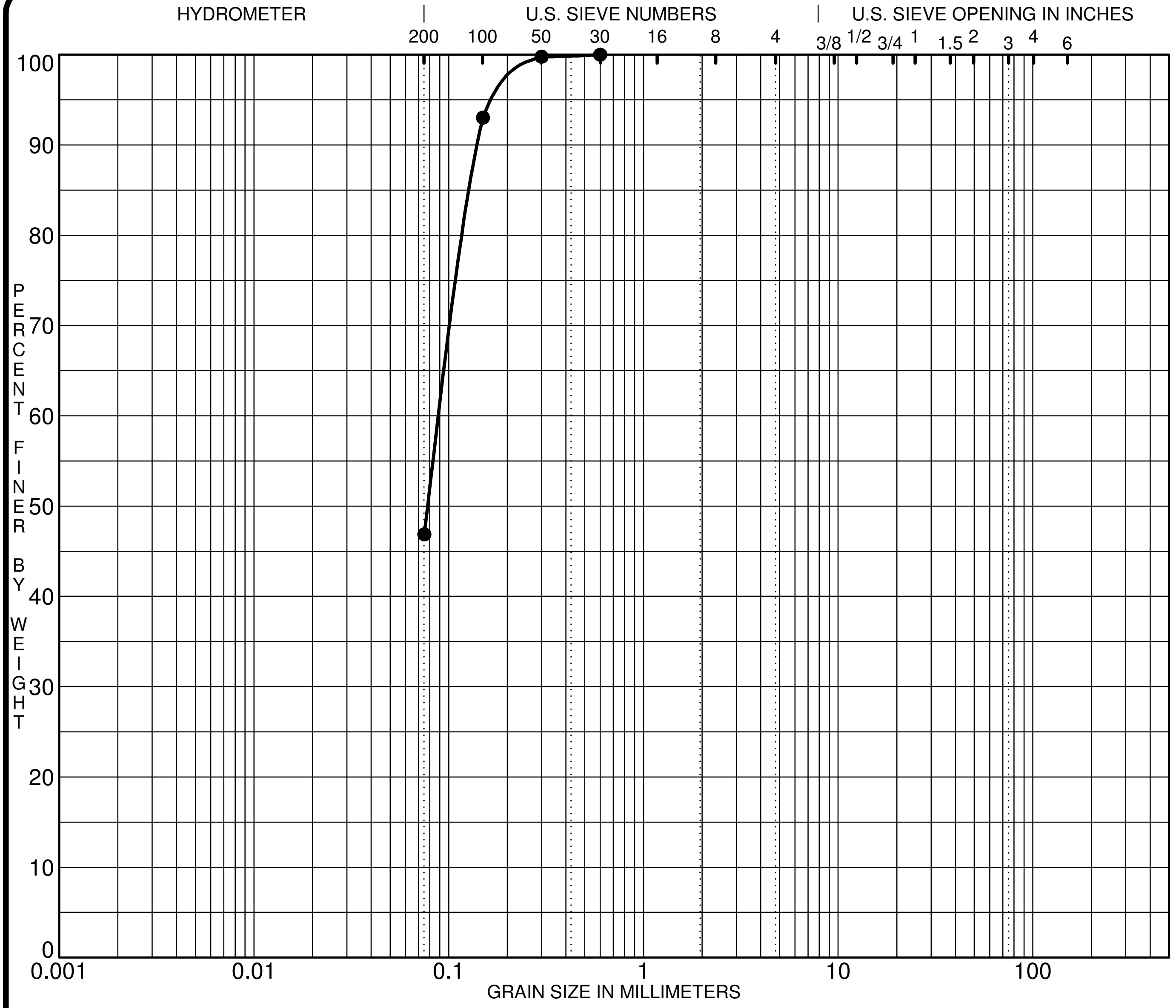
CLIENT:	Tamarack Homes	DESCRIPTION:	Aggregate	FILE NO:	PG3786
CONTRACT NO.:	-	SPECIFICATION:	**Silty Fine Sand**	LAB NO:	98979
PROJECT:	The Meadows Phase 4	INTENDED USE:	-	DATE RECEIVED:	6-Mar-18
		PIT OR QUARRY:	-	DATE TESTED:	7-Mar-18
DATE SAMPLED:	Nathan	SOURCE LOCATION:	TP4 - 18 G1	DATE REPORTED:	7-Mar-18
SAMPLED BY:	43158	SAMPLE LOCATION:	1.1 to 1.2	TESTED BY:	E.B/D.B



Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
						0.80	2.5

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
2.5	0.16	0.091	0.065	0.0	81.2	18.8	

Comments	



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 3-16**	**TW 4**	**MH - SILTY FINE SAND TO SANDY SILT**						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 3-16**	**TW 4**	**0.60**	**0.09**			**0.0**	**53.1**	**46.9**	

CLIENT **Tamarack (Nepean) Corporation**

FILE NO. **PG3786**

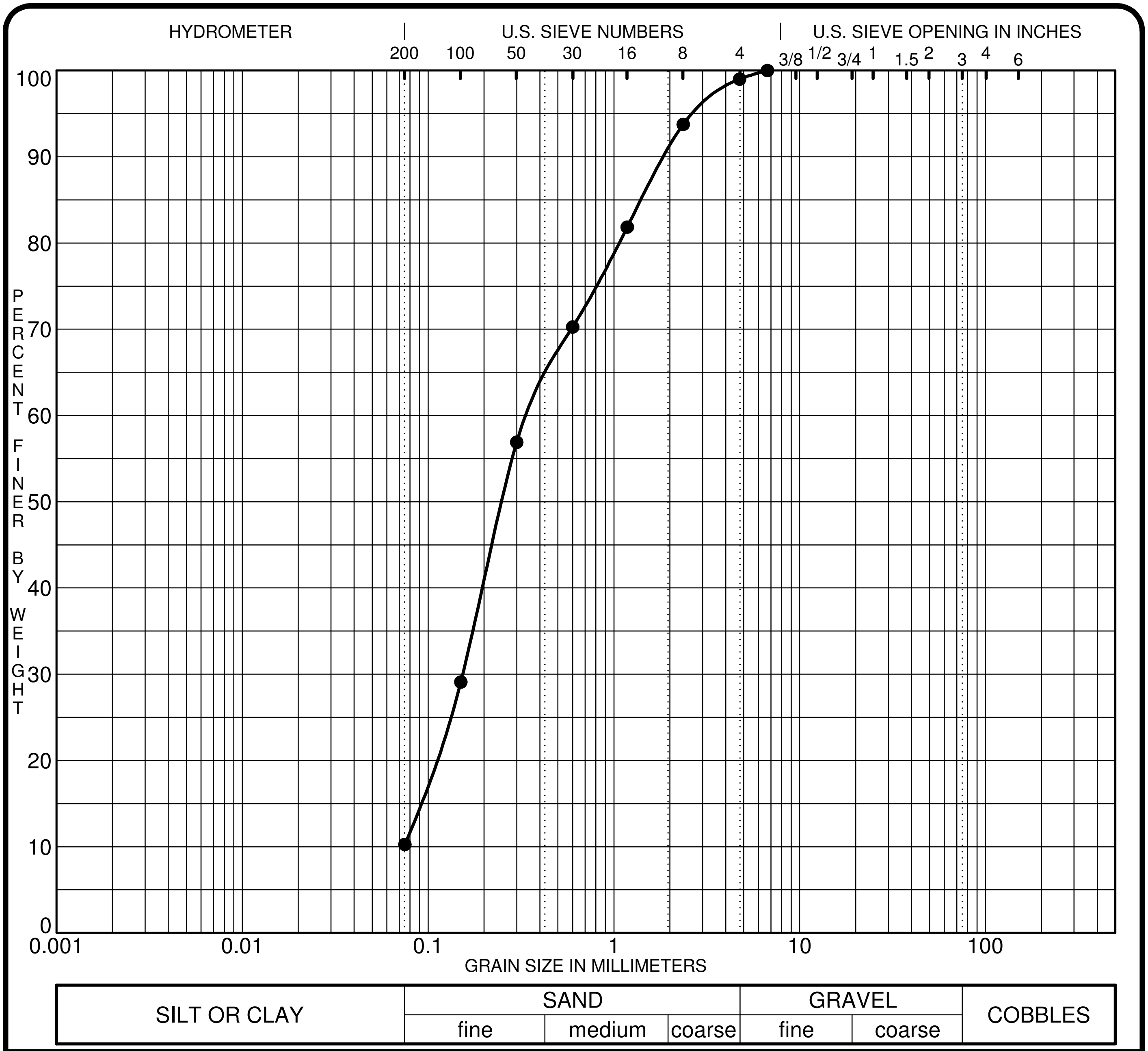
PROJECT **Geotechnical Investigation - Prop. Residential Development - The Meadows Phase 4**

DATE **11 Mar 16**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
● **BH 5-16** **SS 2**	**SM - SILTY SAND**	**22**				**0.90**	**4.7**

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **BH 5-16** **SS 2**	**6.70**	**0.35**	**0.153**		**1.0**	**88.7**	**10.3**	

CLIENT **Tamarack (Nepean) Corporation**

FILE NO. **PG3786**

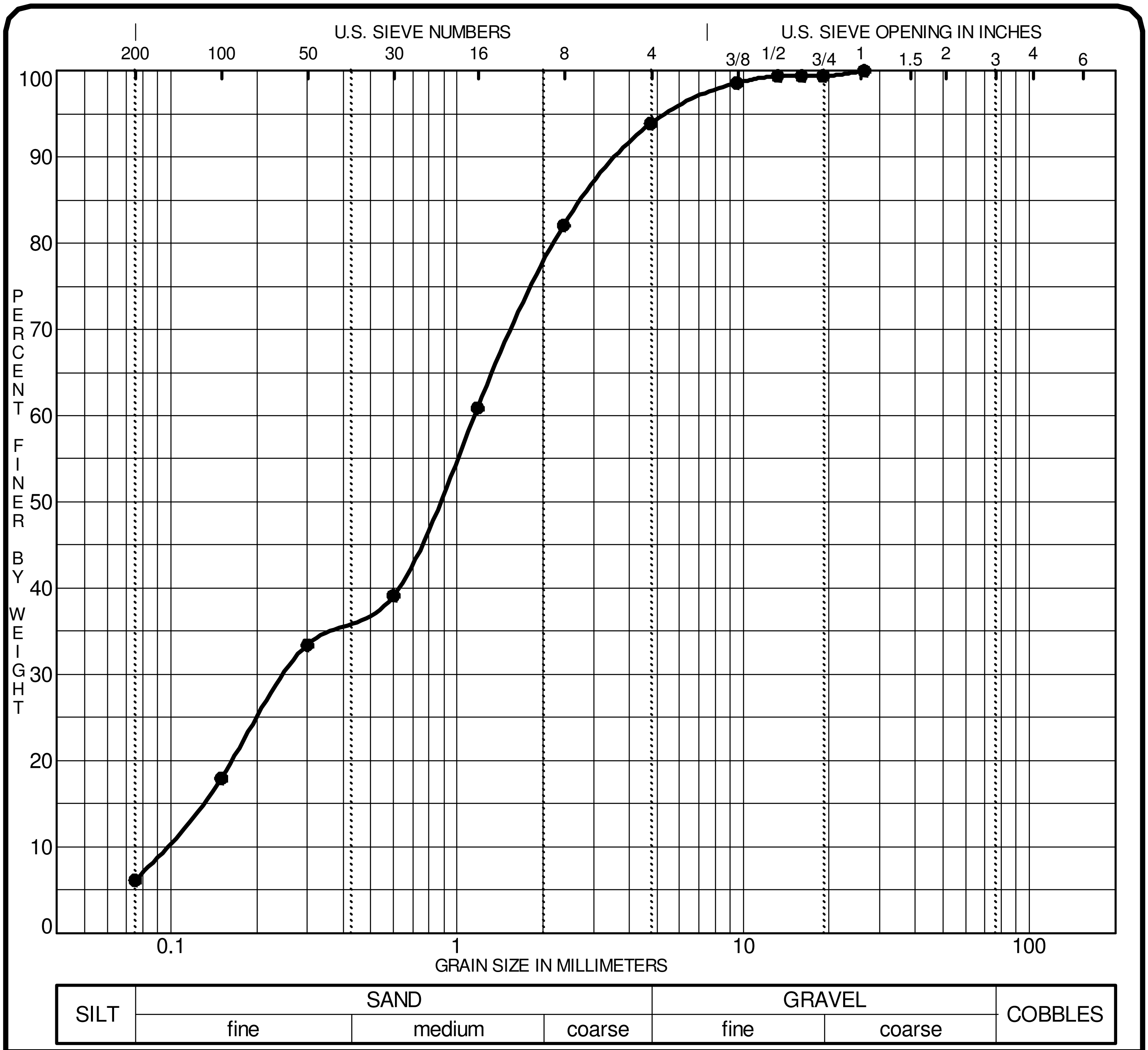
PROJECT **Geotechnical Investigation - Prop. Residential Development - The Meadows Phase 4**

DATE **14 Mar 16**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**TP12**	**G 2**	**Well graded SAND with silt (SW-SM)**					**0.61**	**12.2**
			Based on ASTM D 2487						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP12**	**G 2**	**26.50**	**1.15**	**0.258**	**0.0943**	**6.1**	**87.8**	**6.1**	

CLIENT **Mattamy Homes**

FILE NO. **PG0177**

PROJECT **Geotechnical Investigation - Francis Lands-Cambrian Road @ Greenbank Road**

DATE **1 Apr 04**

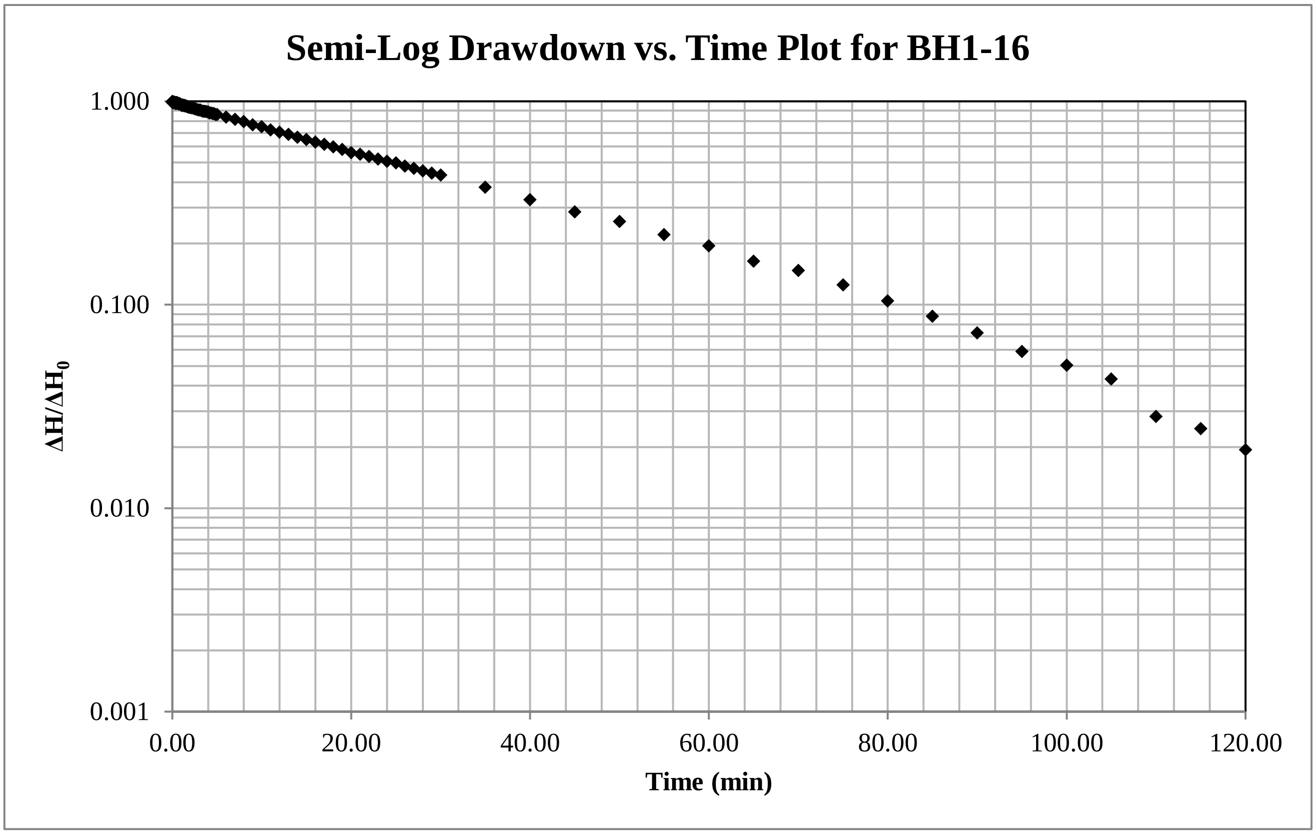
patersongroup **Consulting Engineers**

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

GRAIN SIZE DISTRIBUTION

Hvorslev Hydraulic Conductivity Analysis

Project:	PG3786	
Test hole:	BH1-16	
Test:	Falling Head	Test 1
Date:	May 4 - 2016	



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.33873

Well Parameters:

L	1.524	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

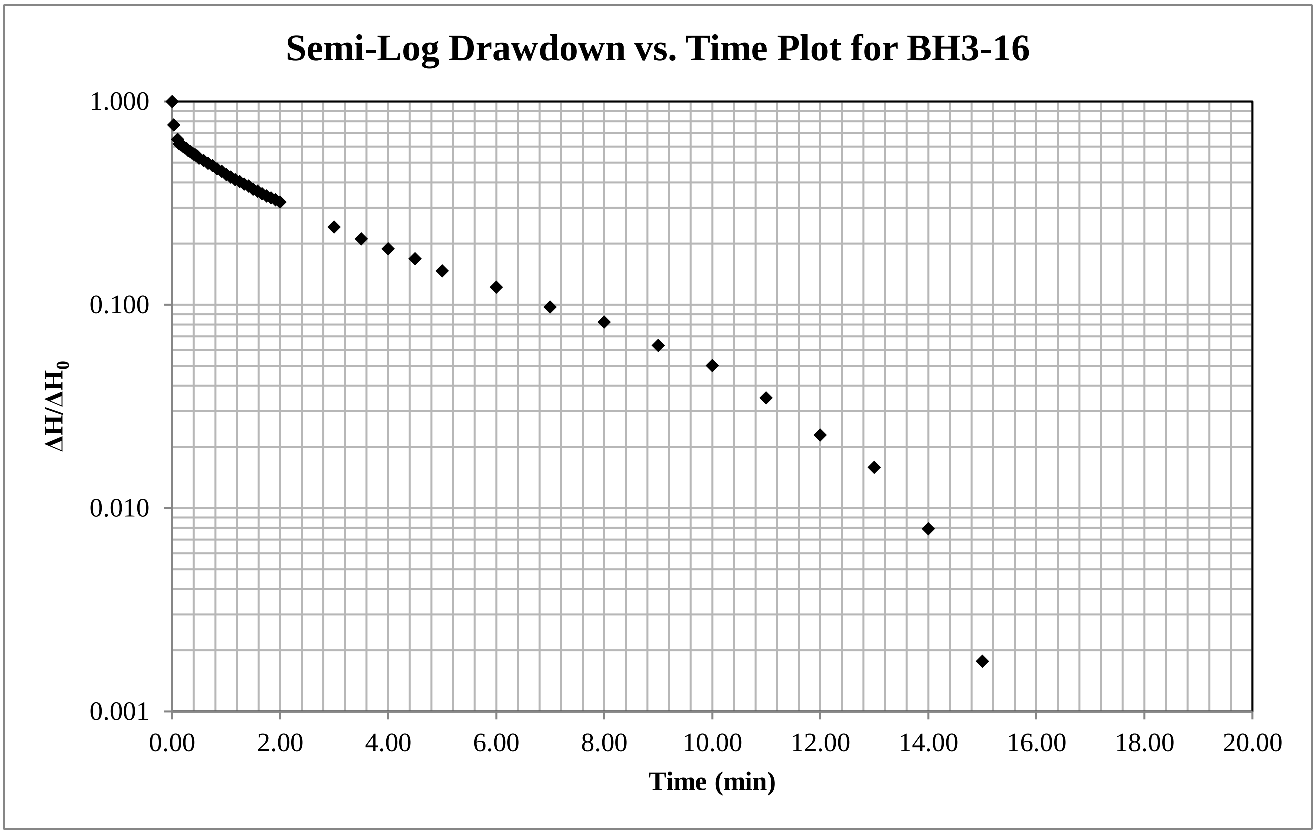
Data Points (from plot):

t*: 35.714 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 4.02E-07 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project:	PG3786	
Test hole:	BH3-16	
Test:	Falling Head	Test 1
Date:	May 4 - 2016	



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t*} \ln\left(\frac{\Delta H *}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 4.00025

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

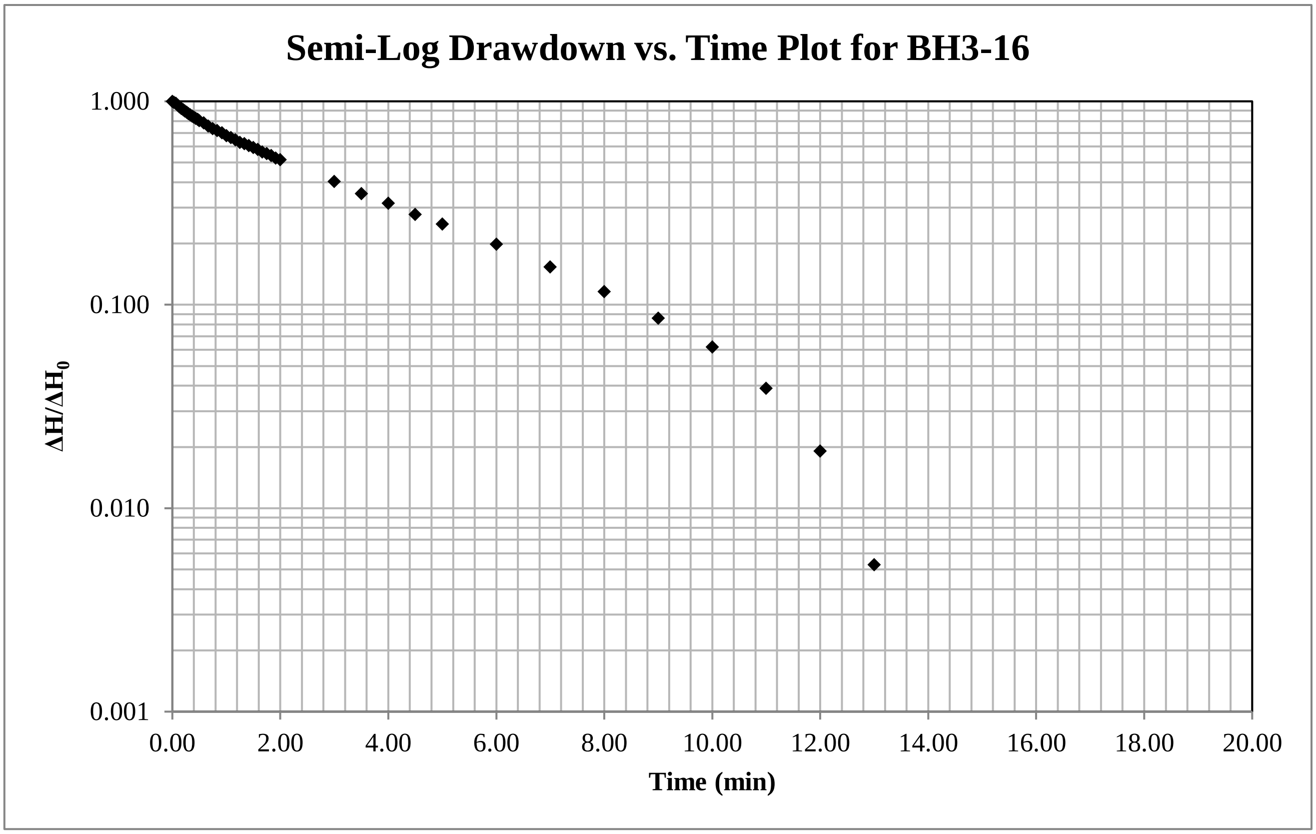
Data Points (from plot):

t*: 1.502 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 5.59E-06 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: PG3786
Test hole: BH3-16
Test: Rising Head Test 1
Date: May 4 - 2016



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 4.00025

Well Parameters:

L	3.048	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

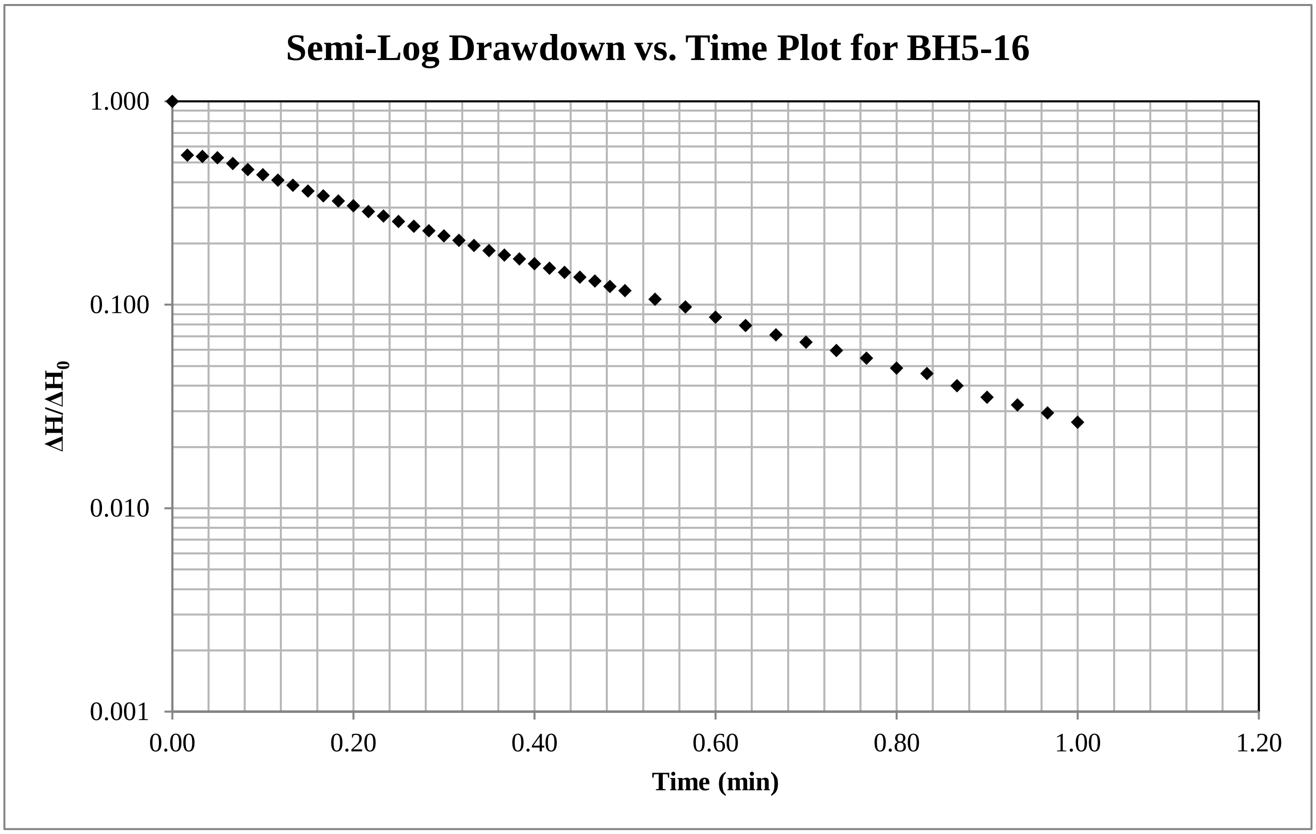
Data Points (from plot):

t^*: 3.329 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.52E-06 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: PG3786
Test hole: BH5-16
Test: Falling Head Test 1
Date: May 4 - 2016



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^{\,2}}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.33873

Well Parameters:

L	1.524	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

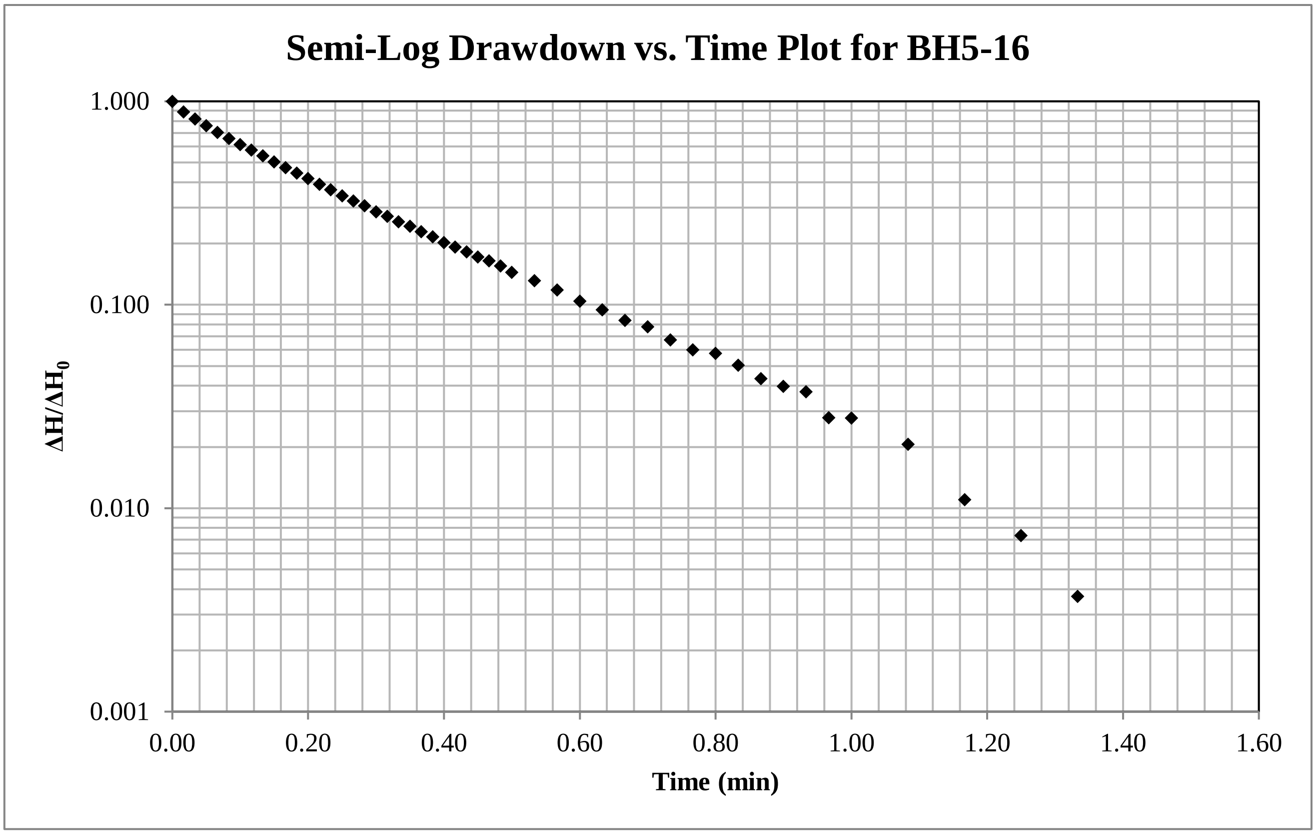
Data Points (from plot):

t*: 0.145 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 9.90E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project:	PG3786	
Test hole:	BH5-16	
Test:	Rising Head	Test 1
Date:	May 4 - 2016	



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^{\,2}}{F}\frac{1}{t*}\ln\left(\frac{\Delta H *}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.33873

Well Parameters:

L	1.524	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

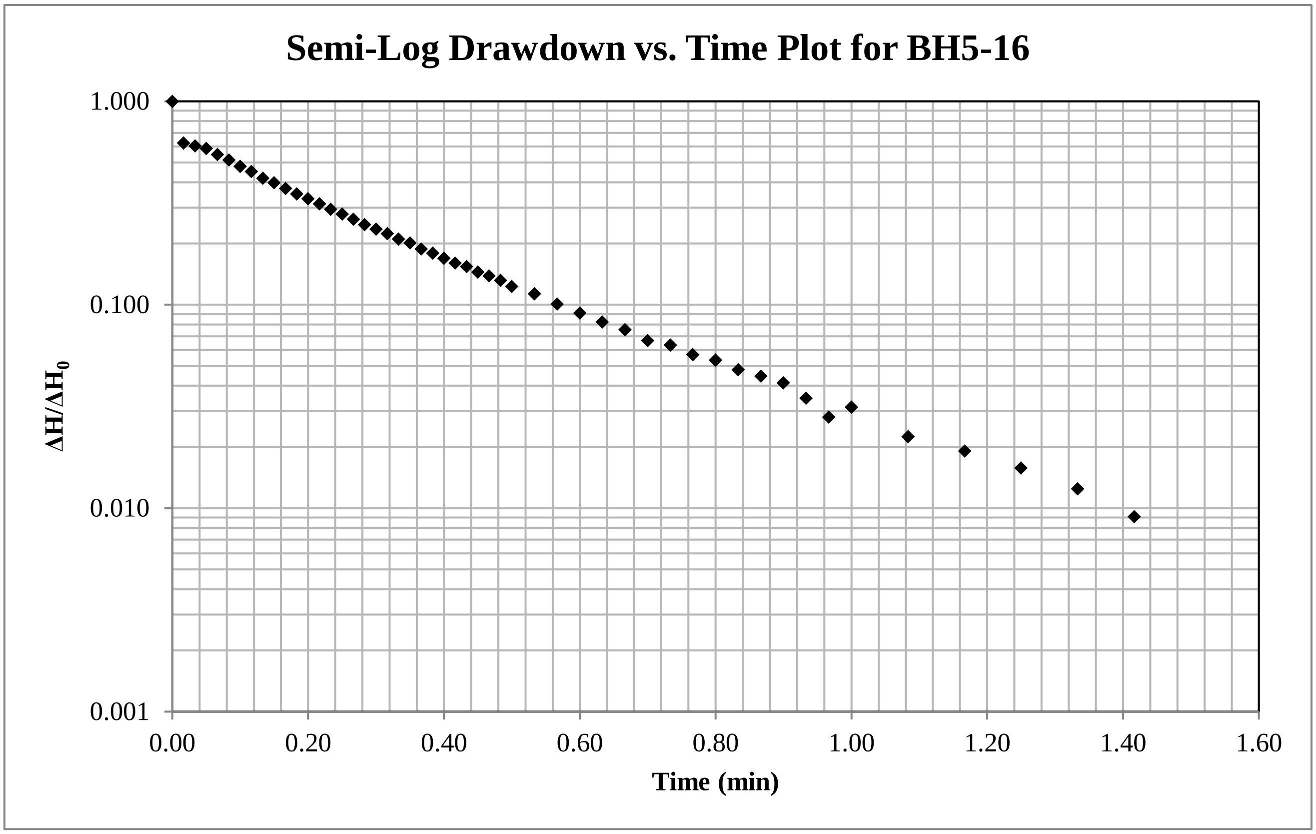
Data Points (from plot):

t*: 0.232 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 6.20E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: PG3786
Test hole: BH5-16
Test: Falling Head Test 2
Date: May 4 - 2016



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.33873

Well Parameters:

L	1.524	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

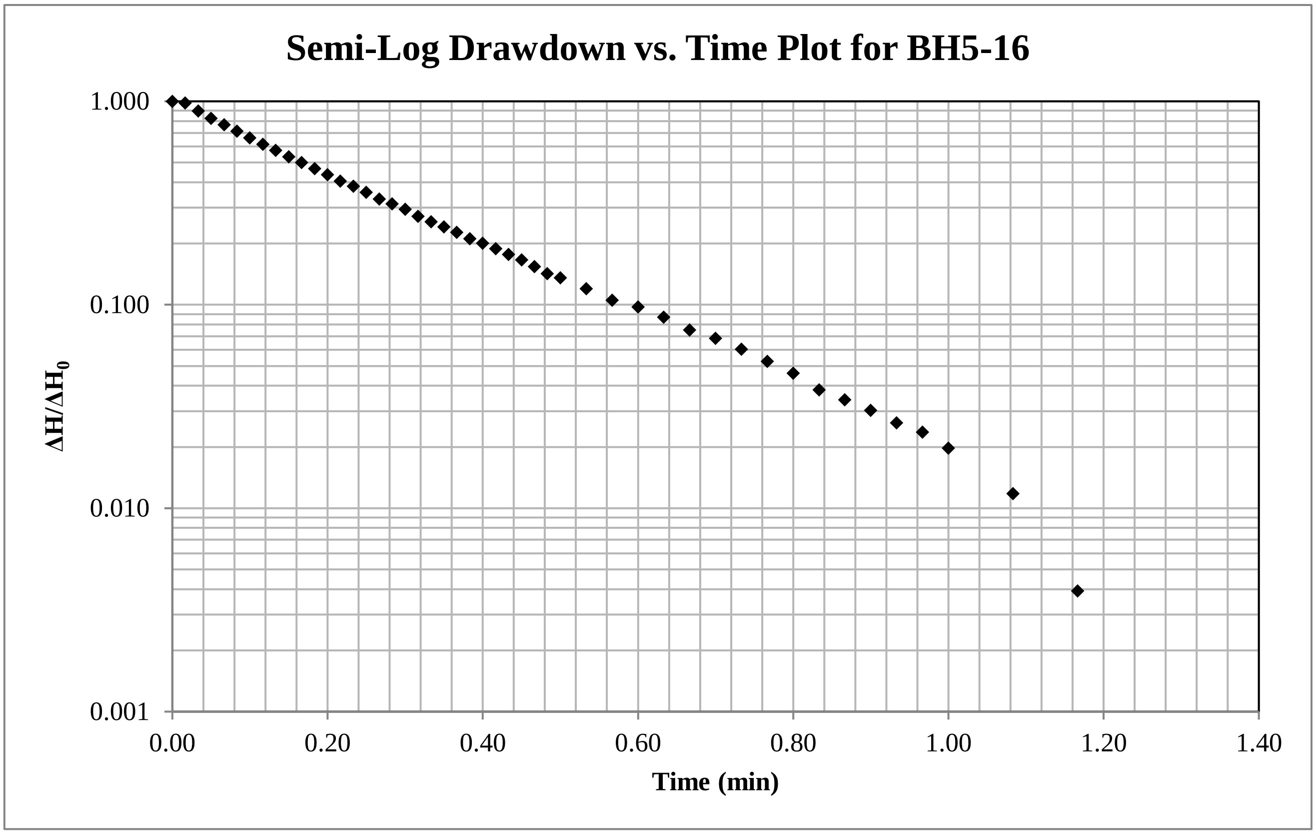
Data Points (from plot):

t*: 0.169 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 8.48E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project:	PG3786	
Test hole:	BH5-16	
Test:	Rising Head	Test 2
Date:	May 4 - 2016	



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t*} \ln\left(\frac{\Delta H *}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 2.33873

Well Parameters:

L	1.524	m	Saturated length of screen or open hole
D	0.0508	m	Diameter of well
r_c	0.0254	m	Radius of well

Data Points (from plot):

t*: 0.242 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 5.94E-05 m/sec

Table 9: Hydraulic Conductivity Test Results

Well ID	Depth of Well (mbgs)	Depth to Water Level (mbgs)	Test Type	Hydraulic Conductivity (m/sec)
BH1-16	3.1	0.4	Falling Head	4.02E-07
BH3-16	6.1	0.1	Falling Head	5.59E-06
			Rising Head	2.52E-06
BH5-16	4.5	0.6	Falling Head	9.90E-05
			Rising Head	6.20E-05
			Falling Head	8.48E-05
			Rising Head	5.94E-05
Maximum				**9.90E-05**
Minimum				**4.02E-07**
Geometric Mean				**1.51E-05**

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG3786-1 - TEST HOLE LOCATION PLAN

DRAWING PG3786-2 - PERMISSIBLE GRADE RAISE AREAS - HOUSING

DRAWING PG3786-3 - TREE PLANTING SETBACK RECOMMENDATIONS

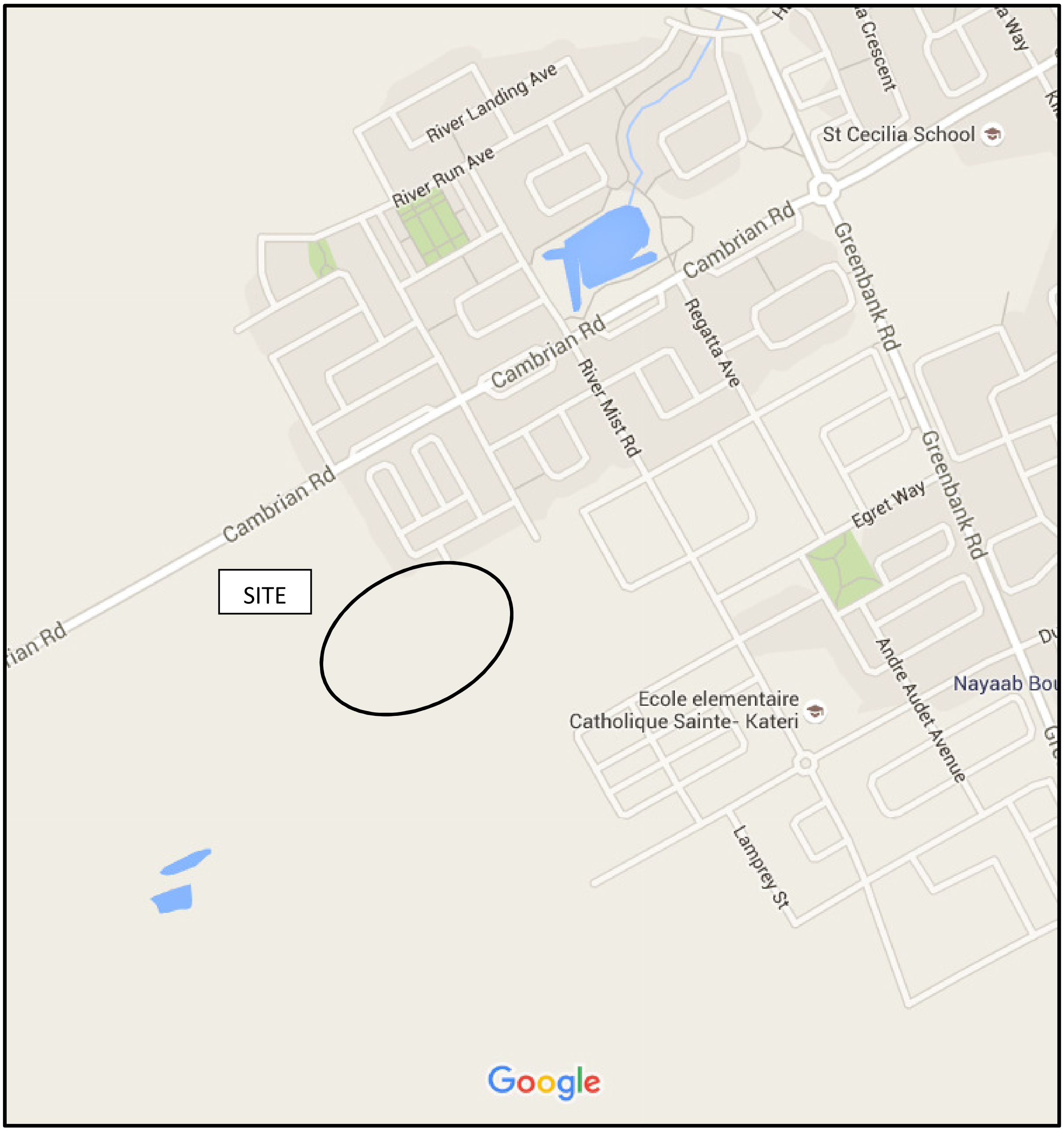


FIGURE 1

KEY PLAN

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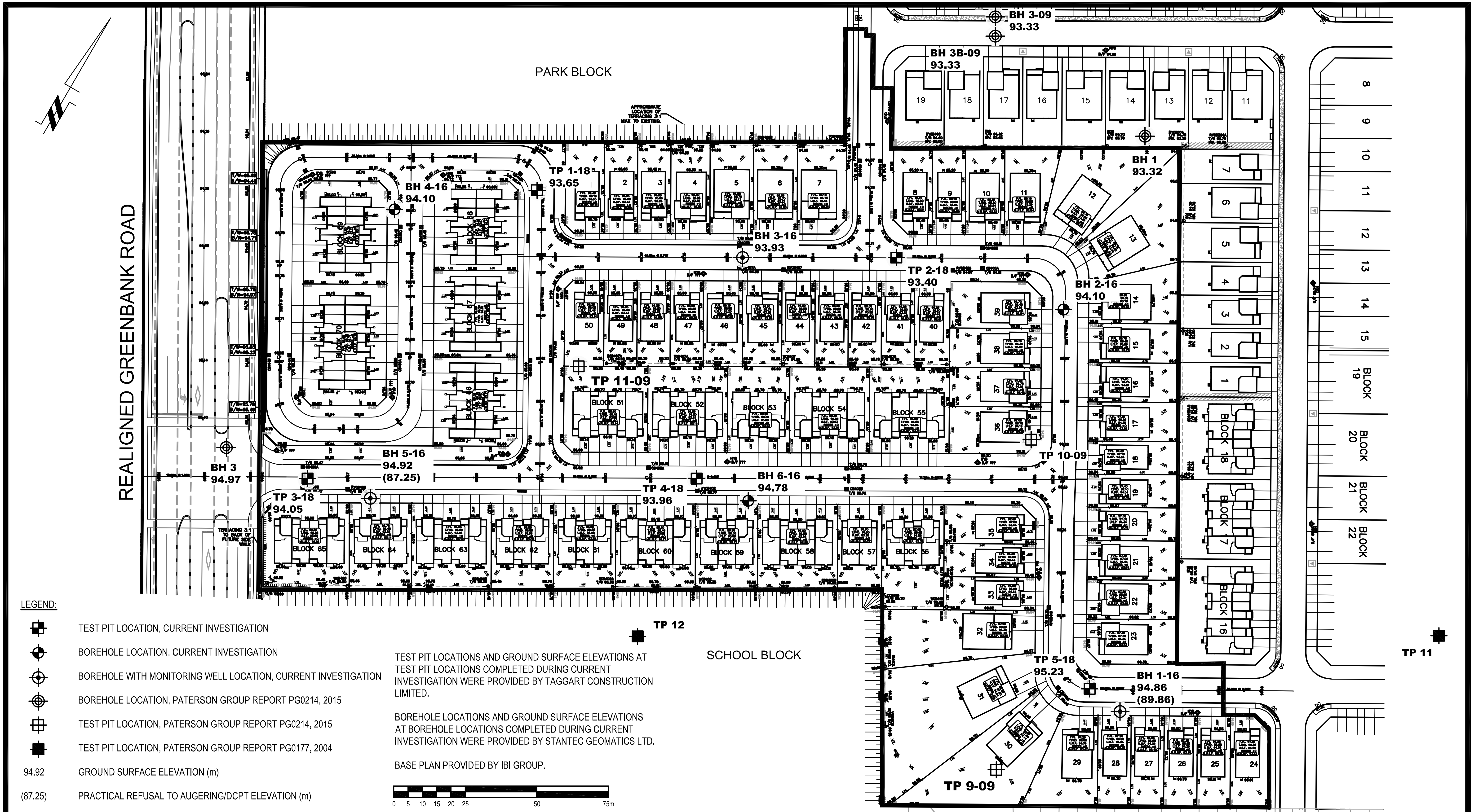

LEGEND:

⊠	TEST PIT LOCATION, CURRENT INVESTIGATION
◐	BOREHOLE LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION, CURRENT INVESTIGATION
◎	BOREHOLE LOCATION, PATERSON GROUP REPORT PG0214, 2015
⊞	TEST PIT LOCATION, PATERSON GROUP REPORT PG0214, 2015
■	TEST PIT LOCATION, PATERSON GROUP REPORT PG0177, 2004
94.92	GROUND SURFACE ELEVATION (m)
(87.25)	PRACTICAL REFUSAL TO AUGERING/DCPT ELEVATION (m)

TEST PIT LOCATIONS AND GROUND SURFACE ELEVATIONS AT TEST PIT LOCATIONS COMPLETED DURING CURRENT INVESTIGATION WERE PROVIDED BY TAGGART CONSTRUCTION LIMITED.

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS COMPLETED DURING CURRENT INVESTIGATION WERE PROVIDED BY STANTEC GEOMATICS LTD.

BASE PLAN PROVIDED BY IBI GROUP.

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
1	GRADING PLAN UPDATED AND 2018 TEST PITS ADDED	05/03/2018	RG

TAMARACK (NEPEAN) CORPORATION
GEOTECHNICAL INVESTIGATION
PROP. RESIDENTIAL DEVELOPMENT - THE MEADOWS PHASE 4
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: :1250	Date: 04/2016
Drawn by: MPG	Report No.: PG3786-1
Checked by: RG	Dwg. No.: PG3786-1
Approved by: DJG	Revision No.: 1

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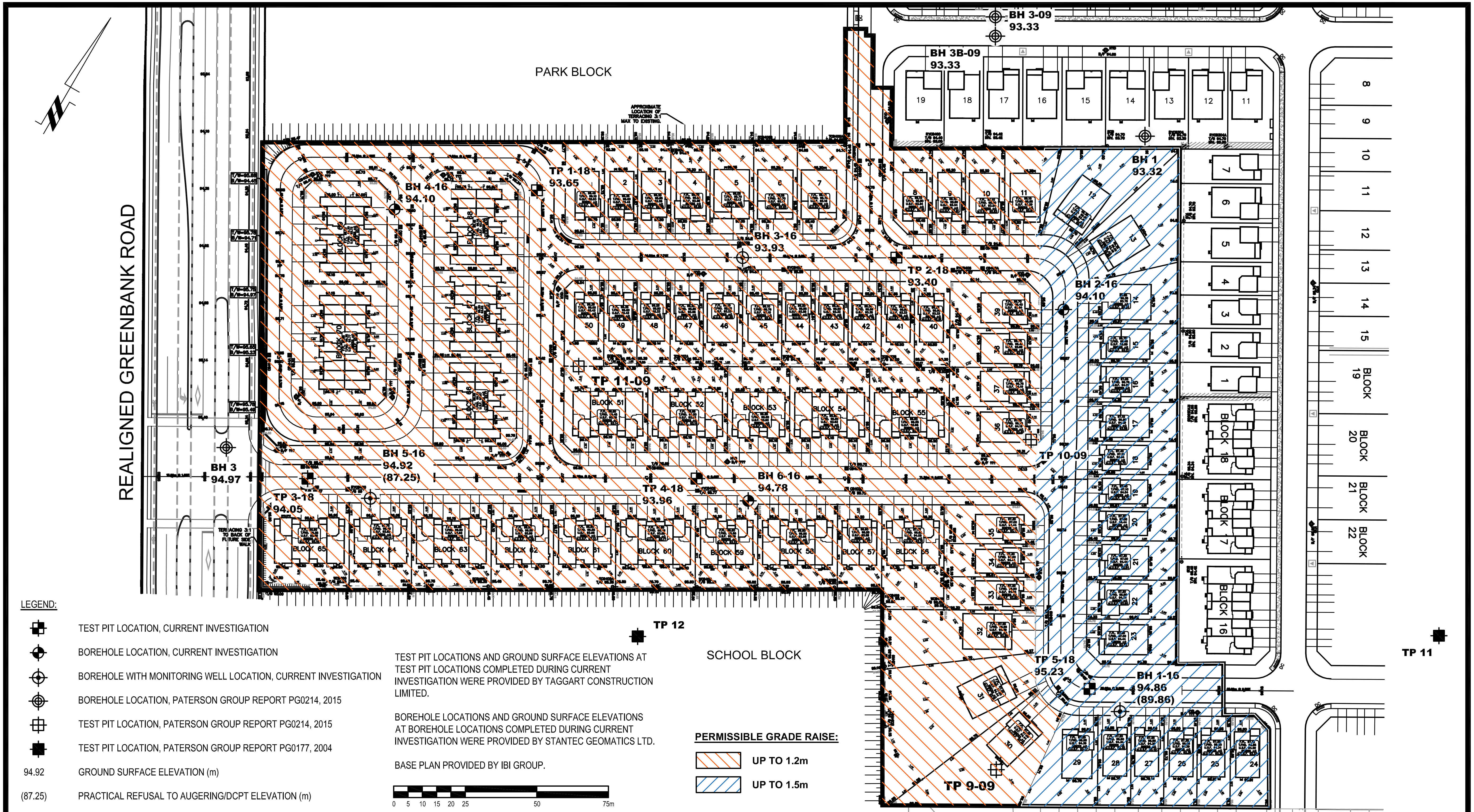

LEGEND:

■	TEST PIT LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE LOCATION, PATERSON GROUP REPORT PG0214, 2015
⊞	TEST PIT LOCATION, PATERSON GROUP REPORT PG0214, 2015
■	TEST PIT LOCATION, PATERSON GROUP REPORT PG0177, 2004
94.92	GROUND SURFACE ELEVATION (m)
(87.25)	PRACTICAL REFUSAL TO AUGERING/DCPT ELEVATION (m)

TEST PIT LOCATIONS AND GROUND SURFACE ELEVATIONS AT TEST PIT LOCATIONS COMPLETED DURING CURRENT INVESTIGATION WERE PROVIDED BY TAGGART CONSTRUCTION LIMITED.

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS COMPLETED DURING CURRENT INVESTIGATION WERE PROVIDED BY STANTEC GEOMATICS LTD.

BASE PLAN PROVIDED BY IBI GROUP.

PERMISSIBLE GRADE RAISE:

▨	UP TO 1.2m
▨	UP TO 1.5m

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consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
1	GRADING PLAN UPDATED AND 2018 TEST PITS ADDED	05/03/2018	RG

TAMARACK (NEPEAN) CORPORATION
GEOTECHNICAL INVESTIGATION
PROP. RESIDENTIAL DEVELOPMENT - THE MEADOWS PHASE 4
OTTAWA, ONTARIO

Title:
PERMISSIBLE GRADE RAISE PLAN - HOUSING

Scale: 1:1250
Date: 04/2016
Drawn by: MPG
Report No.: PG3786-1
Checked by: RG
Dwg. No.: PG3786-2
Approved by: DJG
Revision No.: 1

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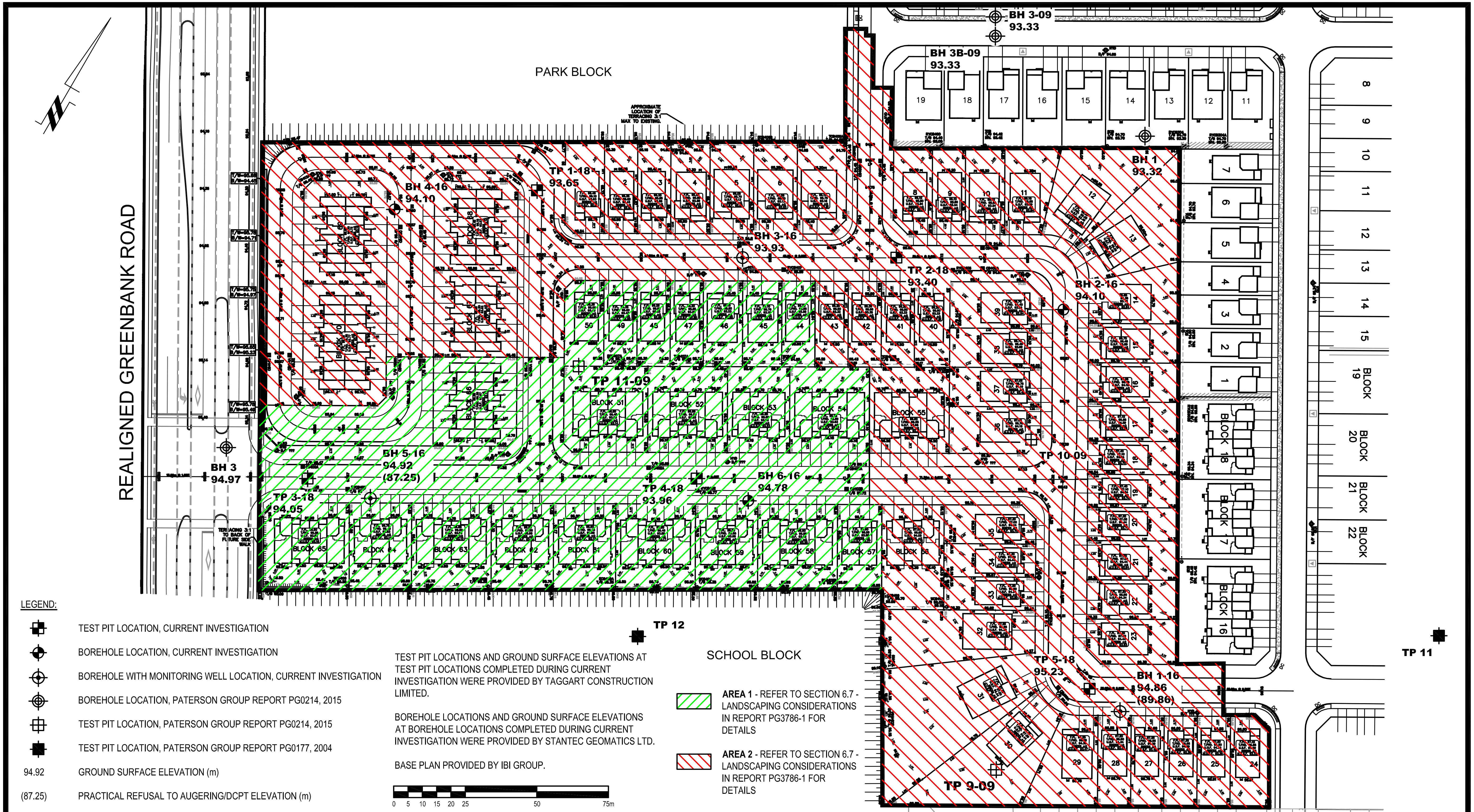

LEGEND:

■ TEST PIT LOCATION, CURRENT INVESTIGATION

● BOREHOLE LOCATION, CURRENT INVESTIGATION

⊕ BOREHOLE WITH MONITORING WELL LOCATION, CURRENT INVESTIGATION

⊕ BOREHOLE LOCATION, PATERSON GROUP REPORT PG0214, 2015

⊞ TEST PIT LOCATION, PATERSON GROUP REPORT PG0214, 2015

■ TEST PIT LOCATION, PATERSON GROUP REPORT PG0177, 2004

94.92 GROUND SURFACE ELEVATION (m)

(87.25) PRACTICAL REFUSAL TO AUGERING/DCPT ELEVATION (m)

TEST PIT LOCATIONS AND GROUND SURFACE ELEVATIONS AT TEST PIT LOCATIONS COMPLETED DURING CURRENT INVESTIGATION WERE PROVIDED BY TAGGART CONSTRUCTION LIMITED.

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS COMPLETED DURING CURRENT INVESTIGATION WERE PROVIDED BY STANTEC GEOMATICS LTD.

BASE PLAN PROVIDED BY IBI GROUP.

🟩 **AREA 1** - REFER TO SECTION 6.7 - LANDSCAPING CONSIDERATIONS IN REPORT PG3786-1 FOR DETAILS

🟥 **AREA 2** - REFER TO SECTION 6.7 - LANDSCAPING CONSIDERATIONS IN REPORT PG3786-1 FOR DETAILS

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

NO.	REVISIONS	DATE	INITIAL
1	GRADING PLAN UPDATED AND 2018 TEST PITS ADDED	05/03/2018	RG

TAMARACK (NEPEAN) CORPORATION
GEOTECHNICAL INVESTIGATION
PROP. RESIDENTIAL DEVELOPMENT - THE MEADOWS PHASE 4
OTTAWA, ONTARIO

Title:
TREE PLANTING SETBACK RECOMMENDATIONS

Scale: :1250
Date: 04/2016
Drawn by: MPG
Report No.: PG3786-1
Checked by: RG
Dwg. No.: PG3786-3
Approved by: DJG
Revision No.: 1

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