PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

6208 Renaud Road
Ottawa, Ontario

Prepared for TM (262615) HOLDING INC., and Marissa & Mathieu Brisebois.

Report PG6640 – 1 Revision 1 dated January 8, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by TM (262615) HOLDING INC., and Marissa & Mathieu Brisebois to conduct a geotechnical investigation for the proposed residential development to be located at 6208 Renaud Road in Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of test holes.

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

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

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

2.0 Proposed Development

Based on our review of available information, it is anticipated the proposed project would include a zoning amendment to an R4 zone for future potential of development such as lightweight framing construction (wooden) low rise apartment and stacked dwelling with a maximum building height of 11m and matching existing neighboring residential buildings

It is anticipated that the site will be municipally serviced by future water, storm and sanitary services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on May 8, 2023, and consisted of a total of two (2) boreholes sampled to a maximum depth of 6.7 m below ground surface throughout the subject site.

The test hole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG6640-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split spoon (SS) sample. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags.

All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils.

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

Groundwater

Flexible standpipe piezometers were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS referenced to a geodetic datum. The locations of the test holes, and the ground surface elevation at each test hole location, are presented on Drawing PG6640-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were collected from the subject site during the investigation and were visually examined in our laboratory to review the results of the field logging. All samples were submitted for moisture content testing. The test results are included on the Soil Profile and Test Data sheets presented in Appendix 1.

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a residential dwelling with a detached garage, asphaltic driveway, and landscaped areas. Large trees are located within the north of the site and along the west property limits.

The subject site is bordered to the north by Renaud Road, to the east by a single story residential dwelling and to the west and south by three-storey residential buildings. The ground surface across the site is relatively flat at an approximate geodetic elevation of 86.6 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of 0.3 m of topsoil underlain by a loose silty sand fill with traces of gravel and crushed stone to a depth of 0.6 m, underlain by compact sandy silt deposit extending to depths of 1.7 m to 2.0 m, below the existing grade. The sandy silt deposit encountered within the boreholes was observed to change from brown to grey in color, with traces of grey silty clay, below a depth 1.4 m from the existing ground surface.

The grey silty sand was underlain by a firm grey silty clay, extending to the end of the test holes. Shear strength ranging from 39 kPa to 24 kPa were measured within the silty clay deposit. These values are indicative of a firm to soft consistency. Remould values were not obtained due to the vane apparatus sinking after the initial shear strength testing, which is common for sensitive silty clays. Once the vane apparatus sinks, a remould value cannot be obtained. Based on the one remould value taken, the sensitivity value of 2.8 is noted. Based on nearby investigations, a sensitivity value ranging from 8 to 10 is expected within the weaker clay layer. Therefore the silty clay deposit at the site could be designated as an extra sensitive clay and the sensitivity of the silty clay has been accounted for in our recommendations.

Specific details of the soil profile at each test hole location are presented Appendix 1.

Atterberg Limit Testing Results

Atterberg limits testing was completed on nearby silty clay samples. The results of the Atterberg limits test are presented in Table 1 and in Appendix 1. The results of the moisture content test are presented on the Soil Profile and Test Data Sheet in Appendix 1.

The tested silty clay sample classify as inorganic clays of high plasticity (CH) in accordance with the Unified Soil Classification System.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	LI	Classification
PG3154 - BH 2 – TW 5	3.45	82	26	55	93.3	1.20	CH
PG1605 – BH 6 – TW 4	3.48	61	27	34	101.6	2.19	CH
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content; LI: Liquidity Index CH: Inorganic Clay of High Plasticity							

The silty clay samples at a depth of 4.0 m and below presented moisture contents greater than 80%. A high moisture content is indicative of sensitive clay as noted by the remold shear strength value. Furthermore, the liquidity index was measured to be greater than 1.0 which further indicates an extra sensitive silty clay deposit.

Bedrock

Based on geological mapping, the overburden drift thickness ranges between 30 and 50 m and is underlain by interbedded limestone and shale bedrock of the Carlsbad formation and shale of the Billings Formation.

4.3 Groundwater

Groundwater level readings were measured on May 19, 2023, and are presented in Table 2 below, and on the Soil Profile and Test Data sheets in Appendix 1.

Table 2 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-23	86.64	1.85	84.79	May 19, 2023
BH 2-23	86.61	2.02	84.59	May 19, 2023
Note: - The ground surface elevations are referenced to a geodetic datum.				

It should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples.

Based on these observations, the long-term groundwater level is anticipated to be at a depth ranging between 1.5 to 2.0 m throughout the subject site. However, groundwater levels are subject to seasonal fluctuations and could vary during the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Design Considerations

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed buildings be founded on conventional spread footings bearing on the undisturbed loose to compact silty sand, stiff silty clay and/or approved engineered fill.

Due to the presence of a silty clay deposit at the site, permissible grade raise restrictions have been provided.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant organic materials, should be stripped from under any buildings and other settlement sensitive structures. The existing fill material, where free of organic materials, should be reviewed by Paterson personnel at the time of construction to determine if the existing fill can be left in place below paved areas and below the slab granular fill layers.

Fill Placement

Fill placed for grading beneath the 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 imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building areas should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 98% of their respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geo-composite drainage membrane connected to a perimeter drainage system.

Protection of Subgrade and Bearing Surfaces

It is expected that site grading and preparation will consist of stripping of soil 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. Typically, exposed subgrade surfaces should be protected using a sufficient thickness of select subgrade material, engineered fill or lean concrete mud slab that can sustain vehicle traffic based on the weather conditions at the time of construction.

5.3 Foundation Design

Conventional Spread Footings

It is expected that the proposed building will be founded on conventional spread footings placed on undisturbed, loose to compact silty sand or firm to stiff silty clay. Using continuously applied loads, footings for the proposed buildings can be designed using the bearing resistance values presented in Table 3.

Table 3 – Bearing Resistance Values		
Bearing Surface	**Bearing Resistance Value at SLS (kPa)**	**Factored Bearing Resistance Value at ULS (kPa)**
Loose to Compact Silty Sand	60	90
Firm to Stiff Silty Clay	50	75
Note: Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed on an undisturbed, firm to stiff silty clay or on engineered fill, which is placed and compacted directly over the strata, can be designed using the above bearing resistance values. *Site review should ensure a minimum of 300 mm of silty sand is present under the footings for the bearing resistance proved to be applicable.		

A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance values at ULS. 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 a surface from which all topsoil and deleterious materials, such as loose, frozen or undisturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

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

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. Adequate lateral support is provided to a soil bearing medium 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.

Permissible 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. One (1) consolidation test from the adjacent site is included within this report. The results of the consolidation test from nearby investigation is presented in Table 4 and in Appendix 1.

The value for p' is the preconsolidation pressure and p' 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 4 – Summary of Consolidation Test Results

Borehole No.	Sample	Depth (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c	Q (*)
PG3154 – BH 2	TW 5	3.45	54.5	41.5	0.036	2.982	A
PG3154 – BH 3A	TW 1	5.03	51	45	0.032	2.551	P
PG1605 – BH 6	TW 4	3.48	77	34	0.039	5.171	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 0.5 m was assumed.

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

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

A permissible grade raise restriction of **0.3 m** is recommended for the subject site where silty clay is encountered below underside of footing level.

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.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class E** for the foundations considered as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2020. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2020 for a full discussion of the earthquake design requirements.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill from within the footprints of the proposed buildings, the existing fill, and/or native soil will be considered an acceptable subgrade surface on which to commence backfilling for floor slab construction.

For structures with basement slabs, it is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone.

For structures with slab-on-grade construction, the upper 300 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprints of the proposed buildings should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of its SPMDD.

5.6 Pavement Structure

For design purposes, the following pavement structures, presented below, are recommended for the design of the car parking areas and local roadways.

Table 5 - 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 6 - Recommended Pavement Structure – Local Residential Roadways	
Thickness (mm)	**Material Description**
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil, bedrock or fill.	

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

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

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping 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 its 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

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for any proposed buildings with below-grade space. The system, where considered, should consist of a 150 mm diameter perforated and corrugated plastic pipe, surrounded on all-sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

For proposed buildings with below-grade space, 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 Miradrain G100N, Delta Drain 6000 or equivalent) connected to a drainage system is provided.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. Generally, a minimum of 1.5 m thick soil cover (or an equivalent combination of soil cover and foundation insulation) should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by temporary shoring systems from the start of the excavation until the structure is backfilled. It is anticipated that sufficient space will be available for the great part of the excavations to be undertaken by open-cut methods (i.e., unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m, should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below ground water level. The subsoil at this site appeared to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time. It is expected that a deep excavation will be required to complete the connection to the storm trunk sewer in the right of way. The deep excavation should be fully supported by trench boxes or temporary shoring during the entirety of the time where the excavation is completed.

Temporary Shoring

Where space restrictions exist, such as the excavation to complete the connection to the storm trunk sewer in the right of way, temporary shoring may be required. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor.

It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system is recommended to consist of a soldier pile and lagging system which could be cantilevered, anchored or braced.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 7 – Soil Parameters	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At Rest Earth Pressure Coefficient (K_0)	0.5
Unit Weight (y), kN/m^3	21
Submerged Unit Weight (y), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Excavation Base Stability

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

- ❑ Shear failure within the ground caused by inadequate resistance to loads imposed by grade differences 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 are 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:

$$\mathrm{FS}_b = \mathrm{N}_b s_u / \sigma_z$$

where,

N_b - Stability factor dependent upon geometry of the excavation and give in Figure 1
S_u - Undrained shear strength of the soil below the base level
σ_z - Total overburden and surcharge pressure at the bottom of the excavation

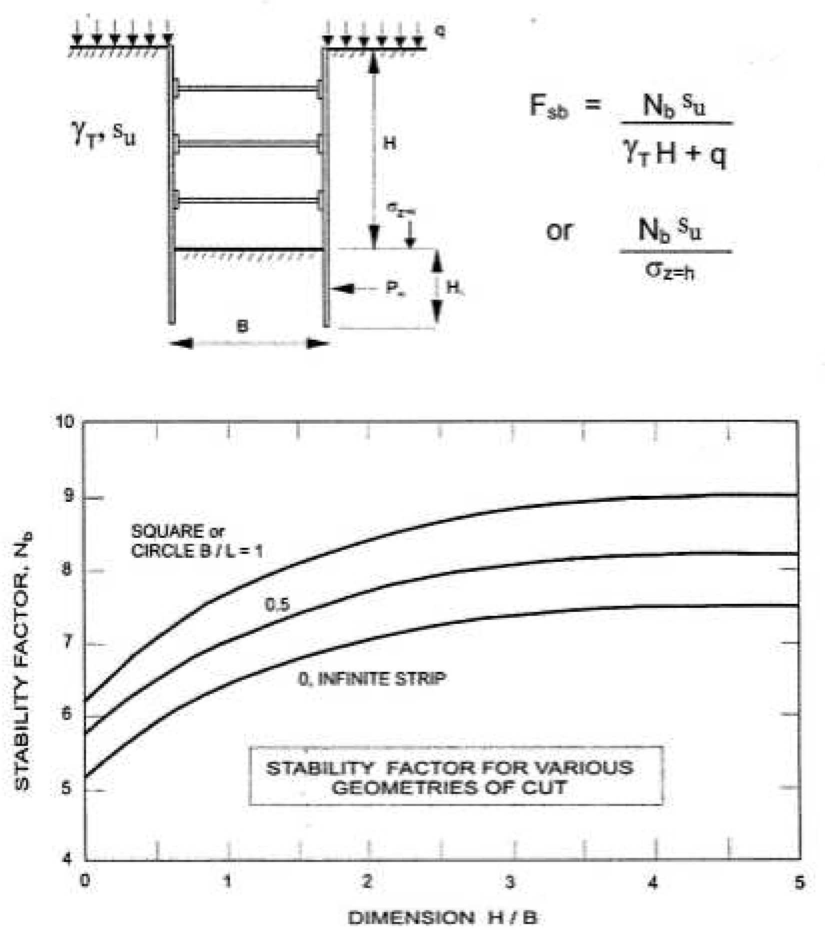


Figure 1 – Stability Factor for Various Geometries of Cut

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

Where the factor of safety is smaller than the required value, it is recommended that means to increase the factor of safety be provided such as stress relief cut at top of excavation and/or deepening the shoring system.

Based on shear values obtained from site, deep excavations of 4 m or more would require special consideration for shoring and trenching to avoid basal heave. Sheet piles should be used to avoid deep lateral forces from disturbing the bearing surfaces. Furthermore, the loading from heavy equipment should be considered during the shoring design and selection of the protection methods. Typically, setbacks of 3 to 4 m are used for equipment and should be specified by the shoring designer.

Excavation Precautions

For open cut situations, due to the subsoil conditions at the depth of the proposed services and the size and weight of the construction equipment and proposed pipe lengths, the following precautions should be taken while working in proximity to any open cut excavations:

- Where an engineered cut-off wall is to be used for shoring the deep excavations, the wall should be embedded at a sufficient depth to reduce the risk of failure at the base of the excavation.
- Trench boxes, where used, should be in direct contact with the trench wall soils. Periodic inspection by Paterson to verify the soil to wall contact is recommended.
- Deep excavations should be backfilled within 24 hours to mitigate risk of basal heaving and deformation of the trench walls (slope failure).
- Stockpiled soils are to be kept away from any deep excavations.
- Where possible, heavy construction traffic should be limited in proximity to the edge of excavation to limit vibration of the sensitive clay soil deposit.
- The roadway should be closed to residential/commercial vehicle traffic entirely, allowing for sufficient space for the installation of the service pipe.

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A crushed stone should be used for pipe bedding for sewer and water pipes. However, the bedding thickness should be increased to 300 mm and placed over a woven geotextile for areas where the services are bearing on firm grey silty clay. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm.

The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to 99% of the material's standard Proctor maximum dry density. Non-woven geotextile should be used to place the granular material on the soft silty clay bearing surface. The use of geogrid might also be required.

It is expected that the excavation to connect to the deep sewer will be in wet conditions and that the backfill material around the sewer will be saturated. Special field review during construction should be completed by Paterson.

It should generally be possible to re-use the moist (not wet) site-generated silty sand or clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated silty clay fill or silty sand will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period. If the silty clay is saturated, an extensive drying period is required to re-use the excavated silty clay or a subgrade improvement design will be required to allow vehicle traffic over the saturated silty clay during construction. Once in the trench, the saturated silty clay typically exhibits an increase in strength over time due to a reduction in excess pore water pressure. Typically, backfill materials similar to the soils along the sidewall are recommended to ensure that differential frost heave effects are limited.

Should the wet silty clay material be selected for re-use above the cover material, a drying period will be required such as the water content is reduced to complete proper compaction of the soil.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in the longest direction should be segregated from re-use as trench backfill.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. 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, sub-bedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 300 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.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. A perched groundwater condition may be encountered within the silty sand deposit which may produce significant temporary groundwater infiltration levels. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. Where deeper excavations are required, such as the excavation to complete the connection to the storm trunk sewer in the right of way, additional pumping could be required. Additional pumps should be used within the open sumps to maintain a dry excavation.

Groundwater Control for Building Construction

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) will be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase.
At least 4 to 5 months should be allowed for completion of the application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

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

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

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Considerations

Due to the presence of the aforementioned clay deposit, the location of street trees will be governed by the potential for soil volume change where trees and houses are located above a clay deposit.

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 the silty clay deposit along nearby adjacent sites. The results of the testing are presented in Table 1 in Subsection 4.2.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be near or greater than 40% in all the tested clay samples. Based on this, the clay is considered to be a clay of high potential for soil volume change.

Based on this, the setbacks would consist of 7.5 m for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m), provided the conditions noted below are met at the time of landscape design:

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

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

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

Based on the current guidelines, large trees (mature height over 14 m) can be planted within the subject site provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g., in a park or other green space).

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e., Manitoba Maples) and, as such, they should not be considered in the landscaping design.

7.0 Recommendations

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

- Review preliminary and detailed grading, servicing and structural plan(s) from a geotechnical perspective.

For the foundation design data provided herein to be applicable, a material testing and observation services program is required to be completed. The following aspects be performed by Paterson:

- ❑ Observation of all bearing surfaces prior to the placement of concrete.\
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Observation of the placement of the foundation insulation, if applicable.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by Paterson.

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

8.0 Statement of Limitations

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than TM (262615) HOLDING INC., and Marissa & Mathieu Brisebois, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Jan 8, 2024
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Nicolas Seguin

Nicolas Seguin, EIT, CPI | Joey R Villeneuve, M.A.Sc., P.Eng., ing.

Report Distribution:

- ❑ TM (262615) HOLDING INC., and Marissa & Mathieu Brisebois. (e-mail copy)
- ❑ Paterson Group Inc (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA SHEETS FROM NEARBY SITE

ATTERBERG TESTING RESULTS FROM NEARBY SITE

CONSOLIDATION TESTING RESULTS FROM NEARBY SITE

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6208 Renaud Road
Ottawa, Ontario

DATUM Geodetic

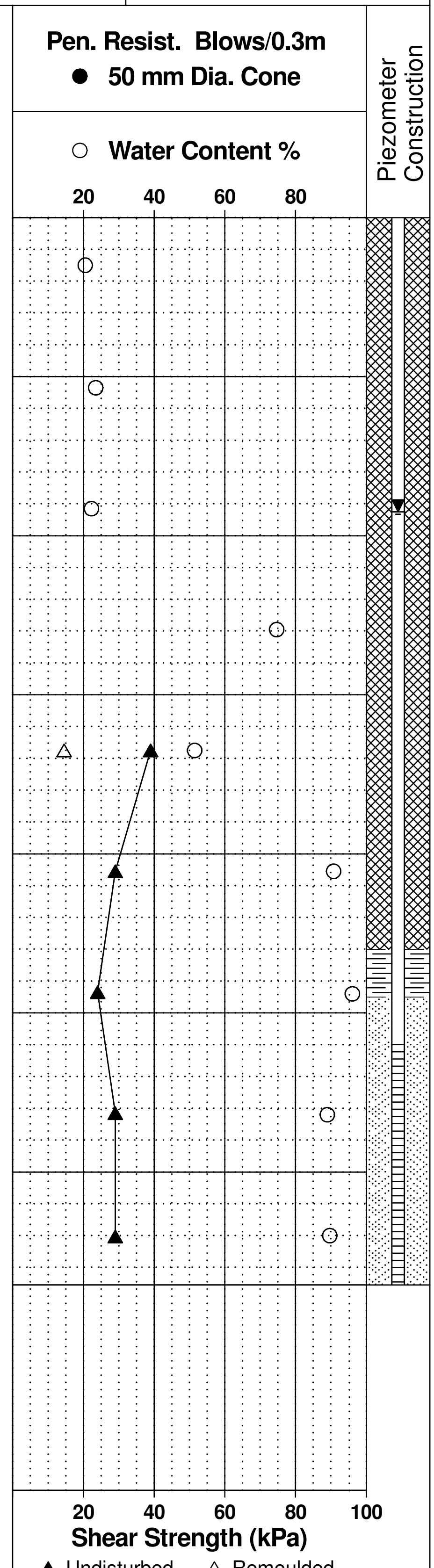
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** May 8, 2023

FILE NO. PG6640

HOLE NO. BH 1-23

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	86.64
TOPSOIL 0.23						
FILL: Brown silty sand, trace gravel and crushed stone 0.60	SS	1	25	3		
Compact, brown **SANDY SILT,** trace clay 1.37	SS	2	46	16	1	85.64
Compact, grey **SANDY SILT,** trace clay 1.68	SS	3	46	11	2	84.64
Firm, grey **SILTY CLAY,** trace sand	SS	4	100	1		
	SS	5	100	P	3	83.64
	SS	6	100	P	4	82.64
	SS	7	100	P	5	81.64
	SS	8	100	P	6	80.64
6.71	SS	9	100	P		
End of Borehole (GWL @ 1.85m depth on April 20, 2023)						



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 6208 Renaud Road
Ottawa, Ontario

DATUM Geodetic

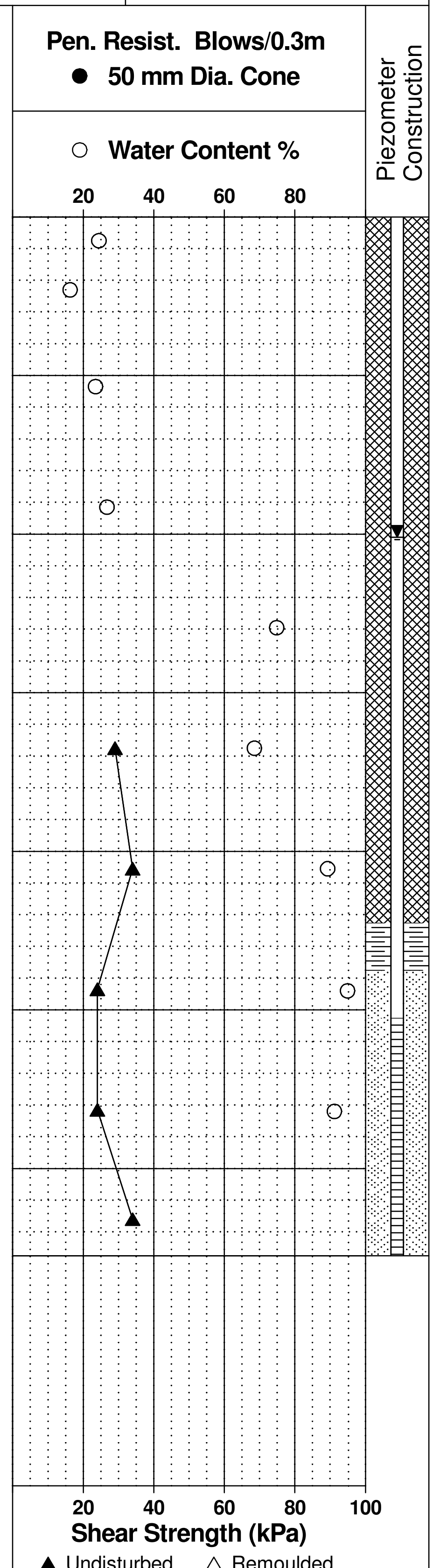
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** May 8, 2023

FILE NO. PG6640

HOLE NO. BH 2-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	86.61
TOPSOIL 0.25							
FILL: Brown silty sand 0.60		SS	1	83	2		
Compact, brown **SANDY SILT,** trace clay 1.37		SS	2	75	13	1	85.61
Compact, grey **SANDY SILT,** trace clay 1.98		SS	3	67	11	2	84.61
Firm, grey **SILTY CLAY,** trace sand		SS	4	92	P	3	83.61
		SS	5	92	P	4	82.61
		SS	6	100	P	5	81.61
		SS	7	100	P	6	80.61
6.55		SS	8	100	P		
End of Borehole (GWL @ 2.02m depth on April 20, 2023)							



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.
P	-	Pushed split spoon sampler through disturbed soil

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

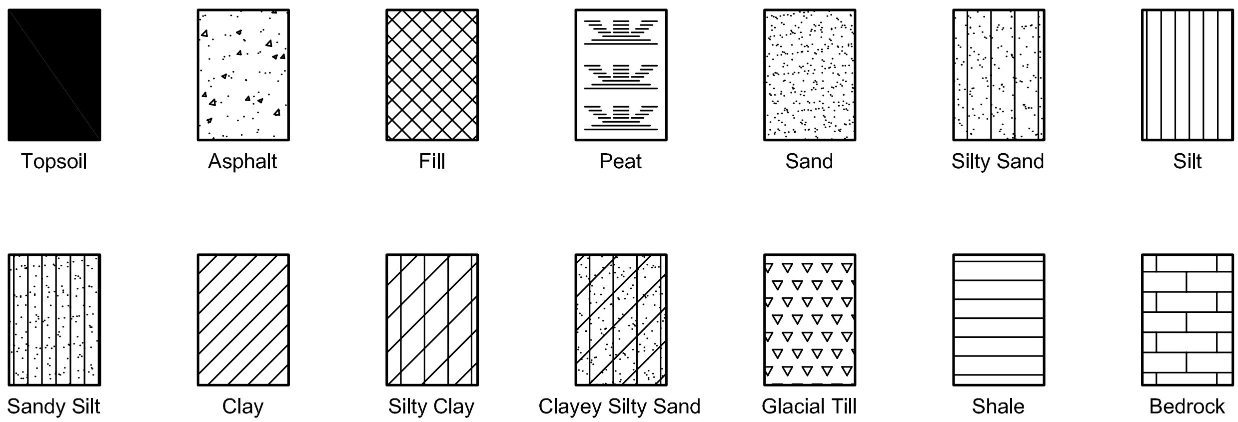
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

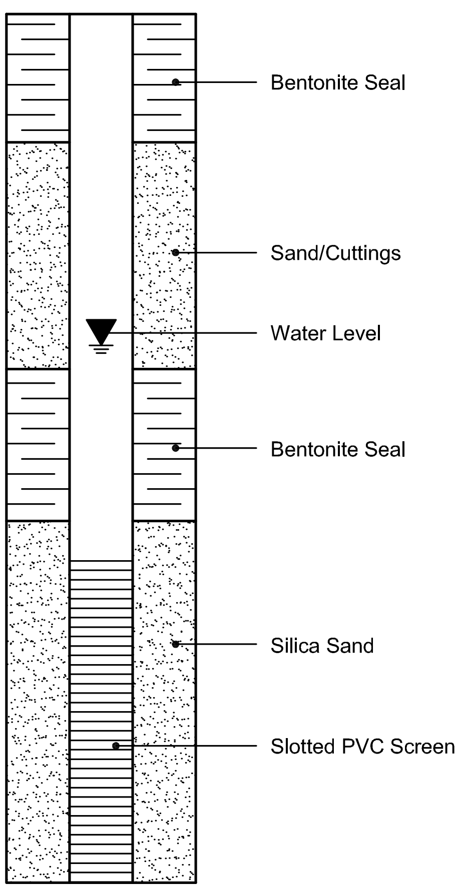
SYMBOLS AND TERMS (continued)

STRATA PLOT

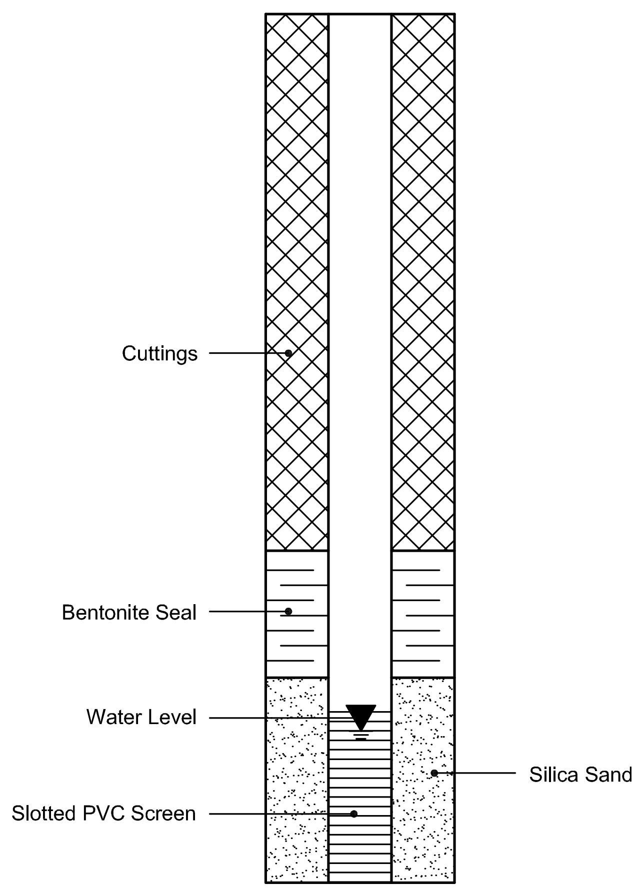


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 57546

Report Date: 26-May-2023

Order Date: 19-May-2023

Project Description: PG6640

	Client ID:	BH2-23-SS6	-	-	-
	Sample Date:	08-May-23 09:00	-	-	-
	Sample ID:	2320501-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	53.7	-	-	-
General Inorganics					
pH	0.05 pH Units	8.21	-	-	-
Resistivity	0.1 Ohm.m	31.4	-	-	-
Anions					
Chloride	10 ug/g dry	44	-	-	-
Sulphate	10 ug/g dry	42	-	-	-

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-Renaud Road
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME 55 Power Auger

DATE 11 Feb 08

FILE NO. PG1605

HOLE NO. BH 6

SOIL DESCRIPTION

GROUND SURFACE

TOPSOIL — 0.20

Stiff, brown **SILTY CLAY**, some sand

- firm to soft and grey by 1.4m depth

- very soft by 8.3m depth

11.74 — End of Borehole

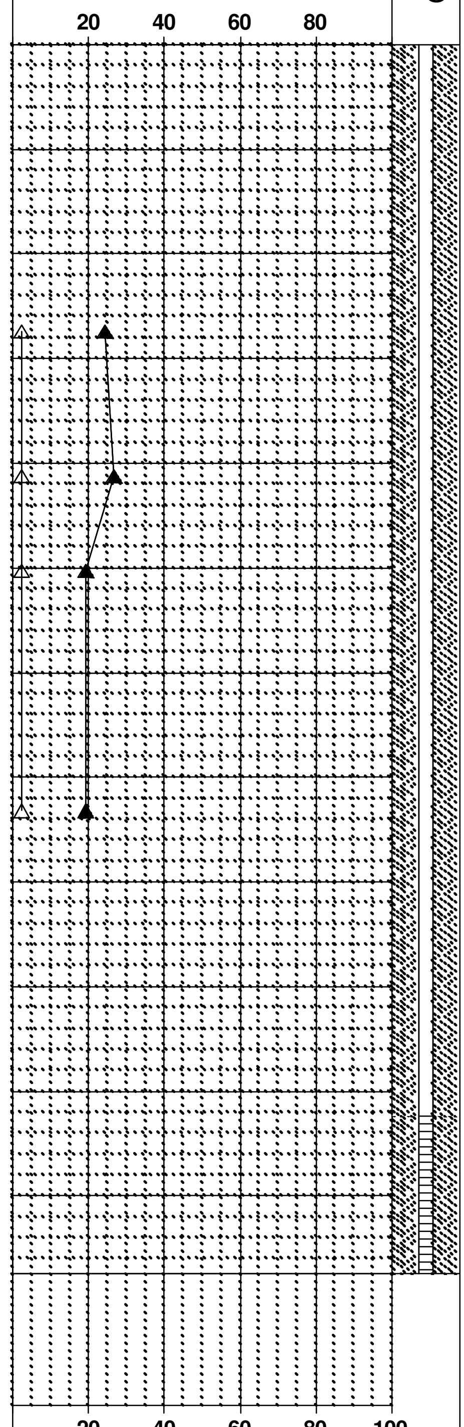
(Piezometer frozen - Feb. 20/08)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	0	4
SS	3	83	2
TW	4	100	
TW	5	100	
TW	6	100	

DEPTH (m)	ELEV. (m)
0	86.17
1	85.17
2	84.17
3	83.17
4	82.17
5	81.17
6	80.17
7	79.17
8	78.17
9	77.17
10	76.17
11	75.17

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

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Piezometer Construction

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-Renaud Road
Ottawa, Ontario

DATUM

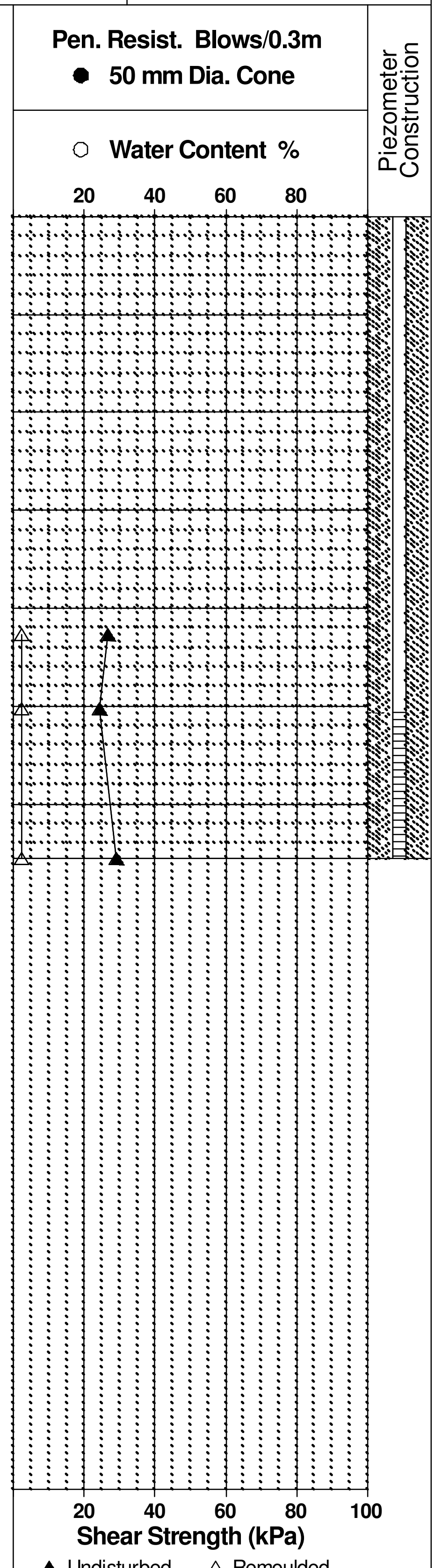
REMARKS

BORINGS BY CME 55 Power Auger **DATE** 11 Feb 08

FILE NO. PG1605

HOLE NO. BH 7

SOIL DESCRIPTION	Depth of base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.53
FILL: Sand and gravel	0.46	AU	1				
Compact to loose, brown **SILTY SAND**	2.44	AU	2				
		SS	3	58	11	1	85.53
		SS	4	50	11	2	84.53
Firm, grey **SILTY CLAY**	6.56	SS	5	25	6		
		SS	6	8	3	3	83.53
						4	82.53
						5	81.53
		SS	7	100	1	6	80.53
End of Borehole							
(Piezometer frozen -Feb. 20/08)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 3143 Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

FILE NO. **PG3154**

REMARKS 18T 459913E; 5030937N

HOLE NO. **BH 2**

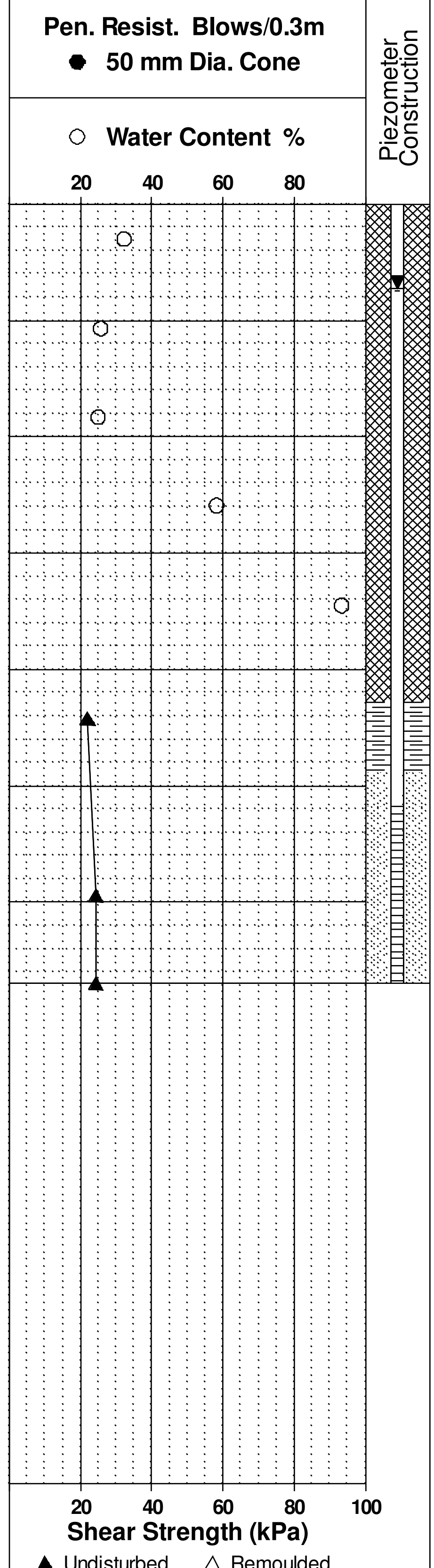
BORINGS BY CME 45 Power Auger **DATE** January 8, 2014

SOIL DESCRIPTION	Bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.15	AU	1		
Compact, brown **SILTY SAND**	2.44	SS	2	50	16
		SS	3	75	12
Soft, grey **SILTY CLAY**	6.70	SS	4	75	3
		TW	5	100	
		TW	6	100	

DEPTH (m)	ELEV. (m)
0	86.13
1	85.13
2	84.13
3	83.13
4	82.13
5	81.13
6	80.13

End of Borehole

(GWL @ 0.72m-Jan. 22, 2014)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 3143 Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

REMARKS 18T 459859E; 5030980N

BORINGS BY CME 45 Power Auger

DATE January 8, 2014

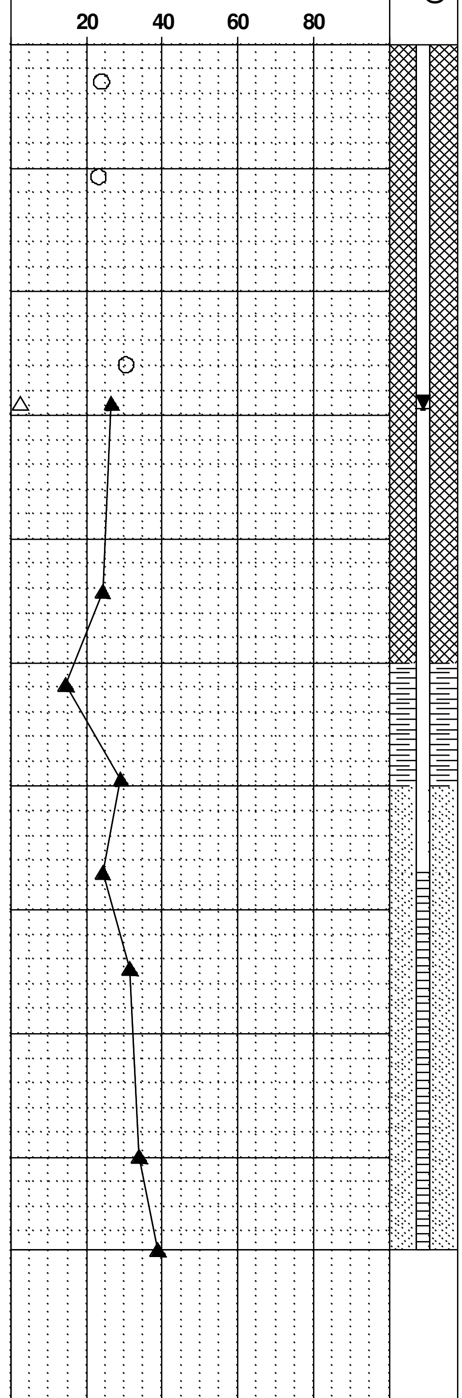
FILE NO. PG3154

HOLE NO. BH 3

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	85.98
TOPSOIL 0.15	AU	1				
Compact, brown **SILTY SAND** 1.45	SS	2	50	12	1	84.98
Soft to firm, grey **SILTY CLAY**	SS	3	83	2	2	83.98
	TW	4	100		3	82.98
					4	81.98
					5	80.98
					6	79.98
					7	78.98
	TW	5	100		8	77.98
9.75					9	76.98
End of Borehole						
(GWL @ 2.94m-Jan. 22, 2014)						

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

○ Water Content %



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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 3143 Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

FILE NO. PG3154

REMARKS

HOLE NO. BH 3A

BORINGS BY CME 45 Power Auger

DATE January 8, 2014

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	85.98
TOPSOIL 0.15							
Compact, brown **SILTY SAND** 1.45						1	84.98
Soft to firm, grey **SILTY CLAY**						2	83.98
						3	82.98
						4	81.98
		TW	1	75		5	80.98
5.49 End of Borehole							

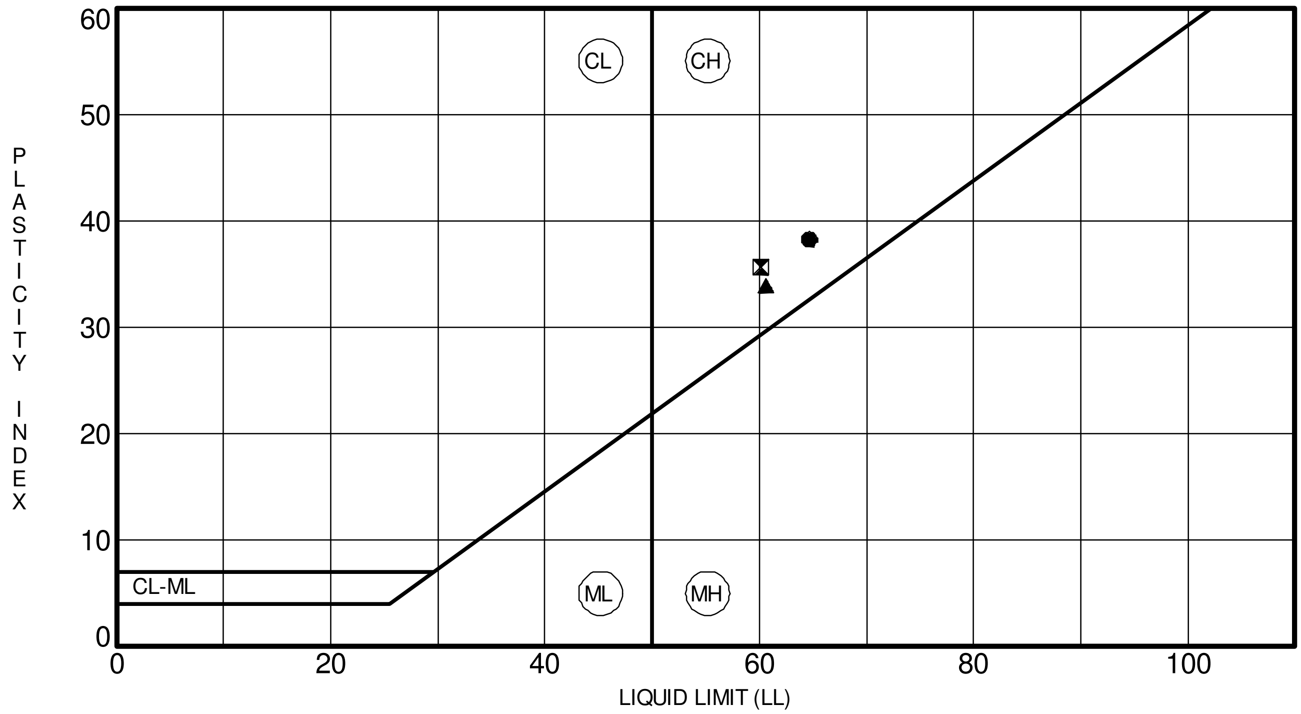
Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

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



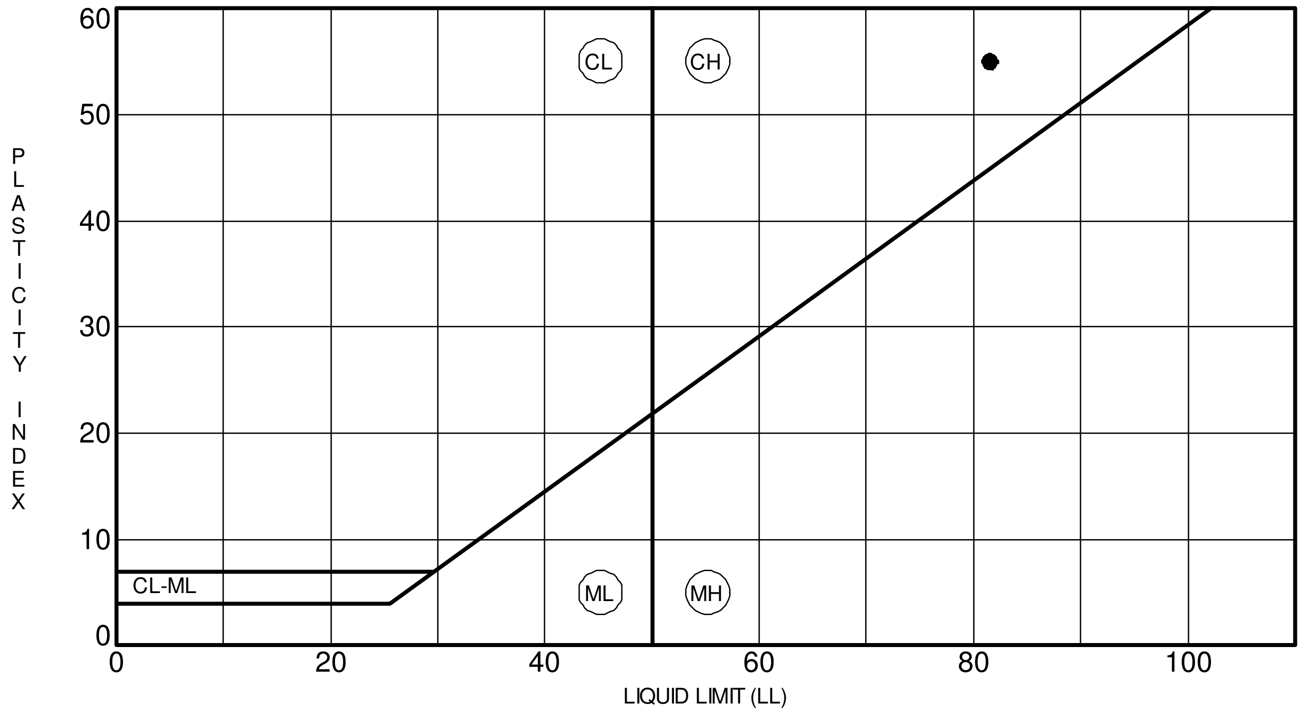
	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 2**	**65**	**26**	**38**		**CH - Inorganic clay of high plasticity**
⊠	**BH 3**	**60**	**24**	**36**		**CH - Inorganic clay of high plasticity**
▲	**BH 6**	**61**	**27**	**34**		**CH - Inorganic clay of high plasticity**

CLIENT **Minto Communities Inc.**
FILE NO. **PG1605**
PROJECT **Geotechnical Investigation - Proposed Residential Development-Renaud Road**
DATE **11 Feb 08**

patersongroup **Consulting Engineers**

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

ATTERBERG LIMITS' RESULTS



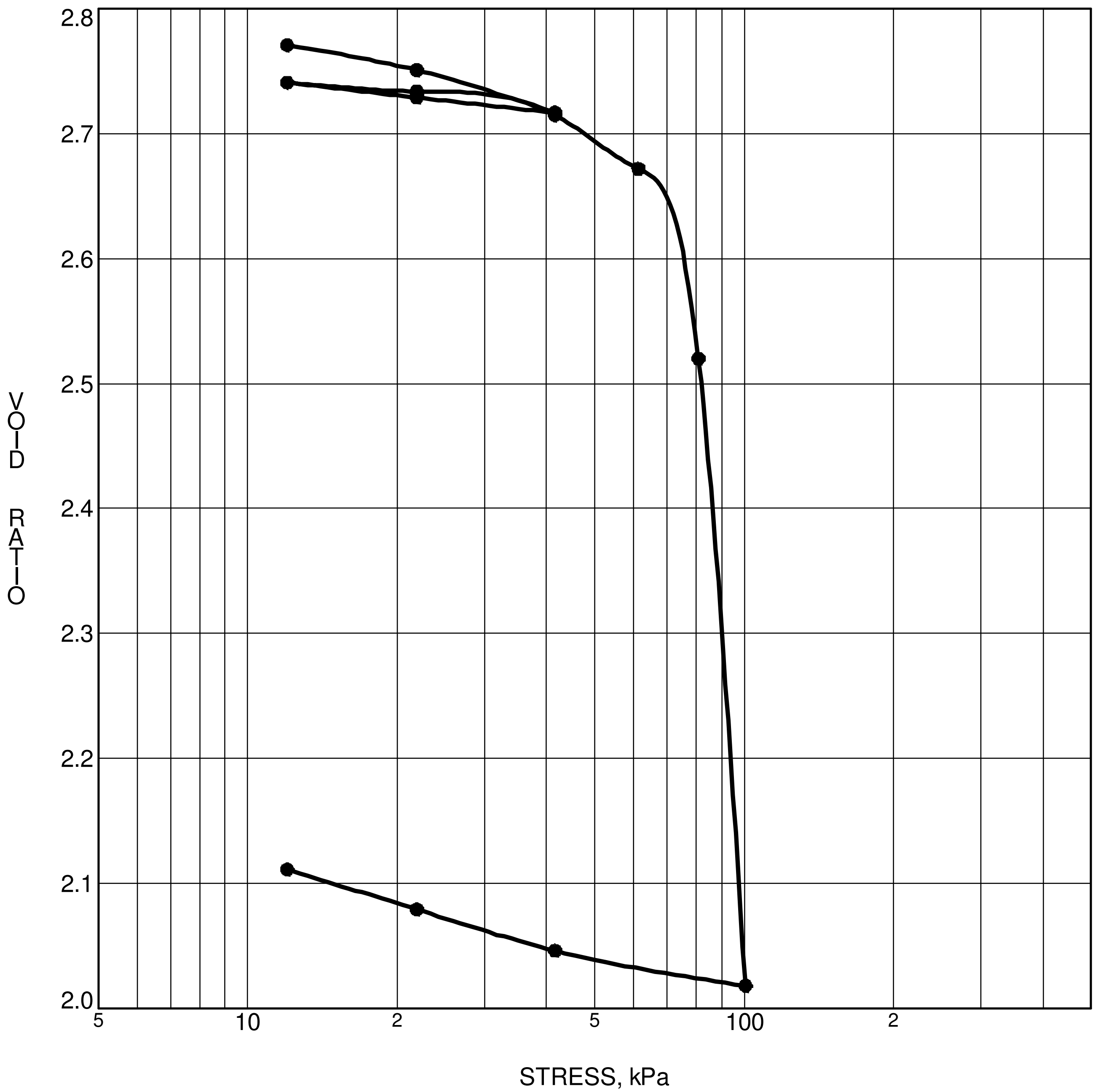
	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 2**	**TW 5**	**82**	**26**	**55**		**CH - Inorganic clays of high plasticity**

CLIENT	**Domicile Developments Inc.**	FILE NO.	**PG3154**
PROJECT	**Geotechnical Investigation - Prop. Residential Development - 3143 Navan Road**	DATE	**8 Jan 14**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



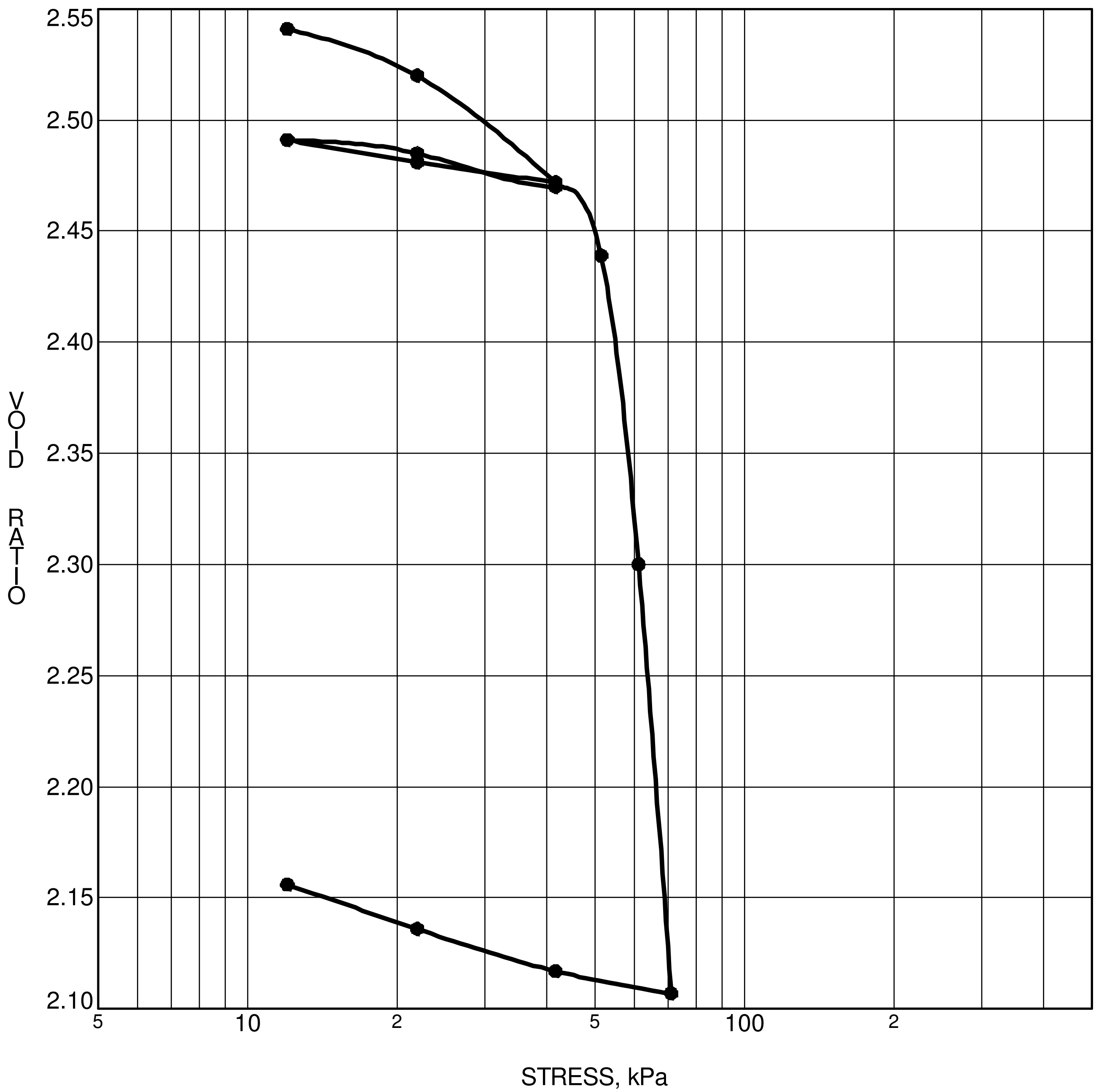
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 6**	p'$_o$	**34** kPa	Ccr	**0.039**
Sample No.	**TW 4**	p'$_c$	**77** kPa	Cc	**5.171**
Sample Depth	**3.48** m	OC Ratio	**2.3**	Wo	**101.6** %
Sample Elev.	**82.69** m	Void Ratio	**2.793**	Unit Wt.	**14.2** kN/m^3

CLIENT	**Minto Communities Inc.**	FILE NO.	**PG1605**
PROJECT	**Geotechnical Investigation - Proposed Residential Development-Renaud Road**	DATE	**02/20/08**

patersongroup **Consulting Engineers**

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

CONSOLIDATION TEST



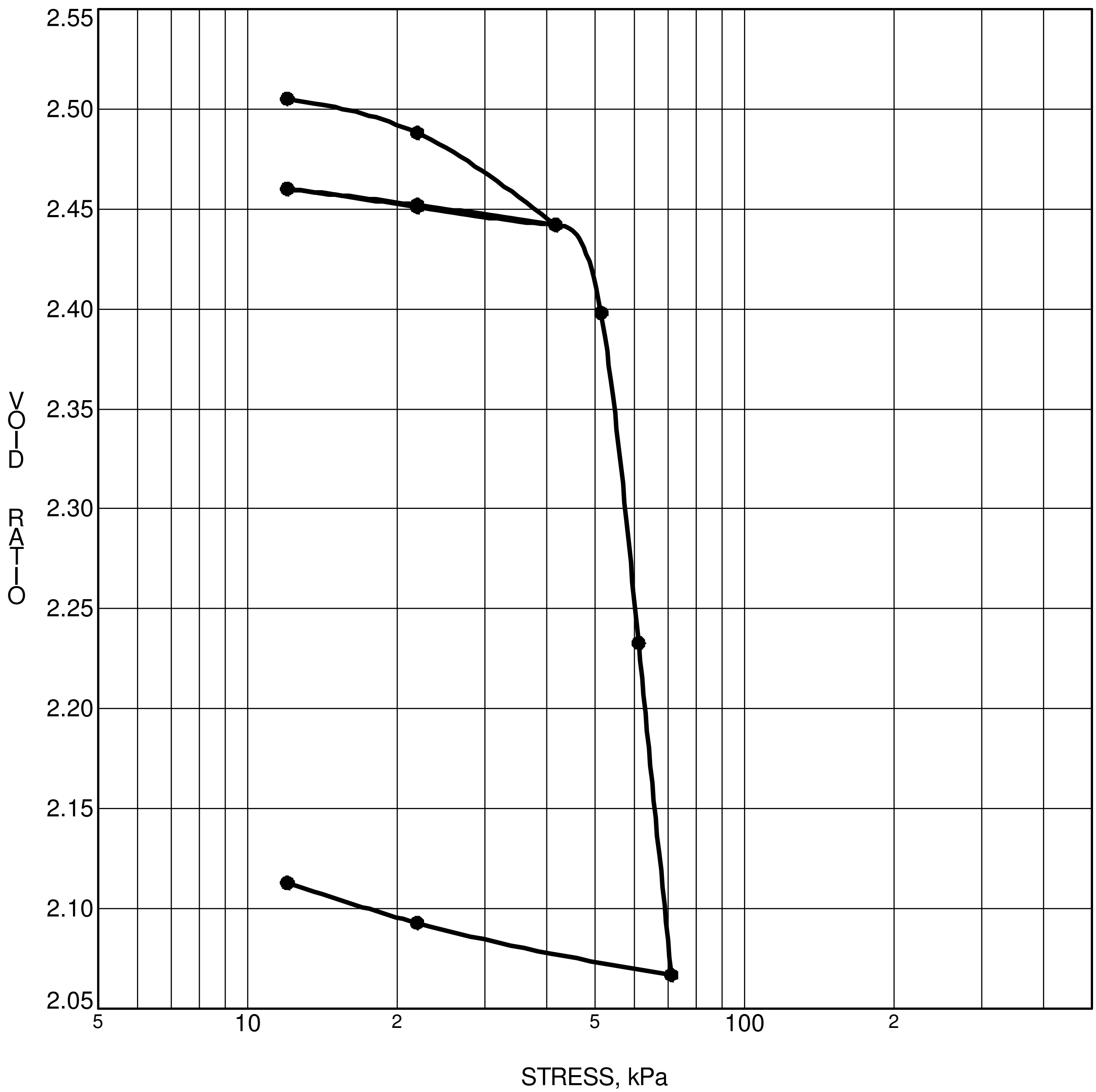
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 2**	p'_o	**41.5** kPa	Ccr	**0.036**
Sample No.	**TW 5**	p'_c	**54.5** kPa	Cc	**2.982**
Sample Depth	**3.45** m	OC Ratio	**1.3**	Wo	**93.3** %
Sample Elev.	**82.68** m	Void Ratio	**2.566**	Unit Wt.	**14.6** kN/m^3

CLIENT	**Domicile Developments Inc.**	FILE NO.	**PG3154**
PROJECT	**Geotechnical Investigation - Prop. Residential Development - 3143 Navan Road**	DATE	**17/01/2014**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 3A**	p'o	**45** kPa	Ccr	**0.032**
Sample No.	**TW 1**	p'c	**51** kPa	Cc	**2.551**
Sample Depth	**5.03** m	OC Ratio	**1.1**	Wo	**92.3** %
Sample Elev.	**80.95** m	Void Ratio	**2.539**	Unit Wt.	**14.7** kN/m^3

CLIENT **Domicile Developments Inc.** FILE NO. **PG3154**

PROJECT **Geotechnical Investigation - Prop. Residential Development - 3143 Navan Road** DATE **17/01/2014**

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST

APPENDIX 2

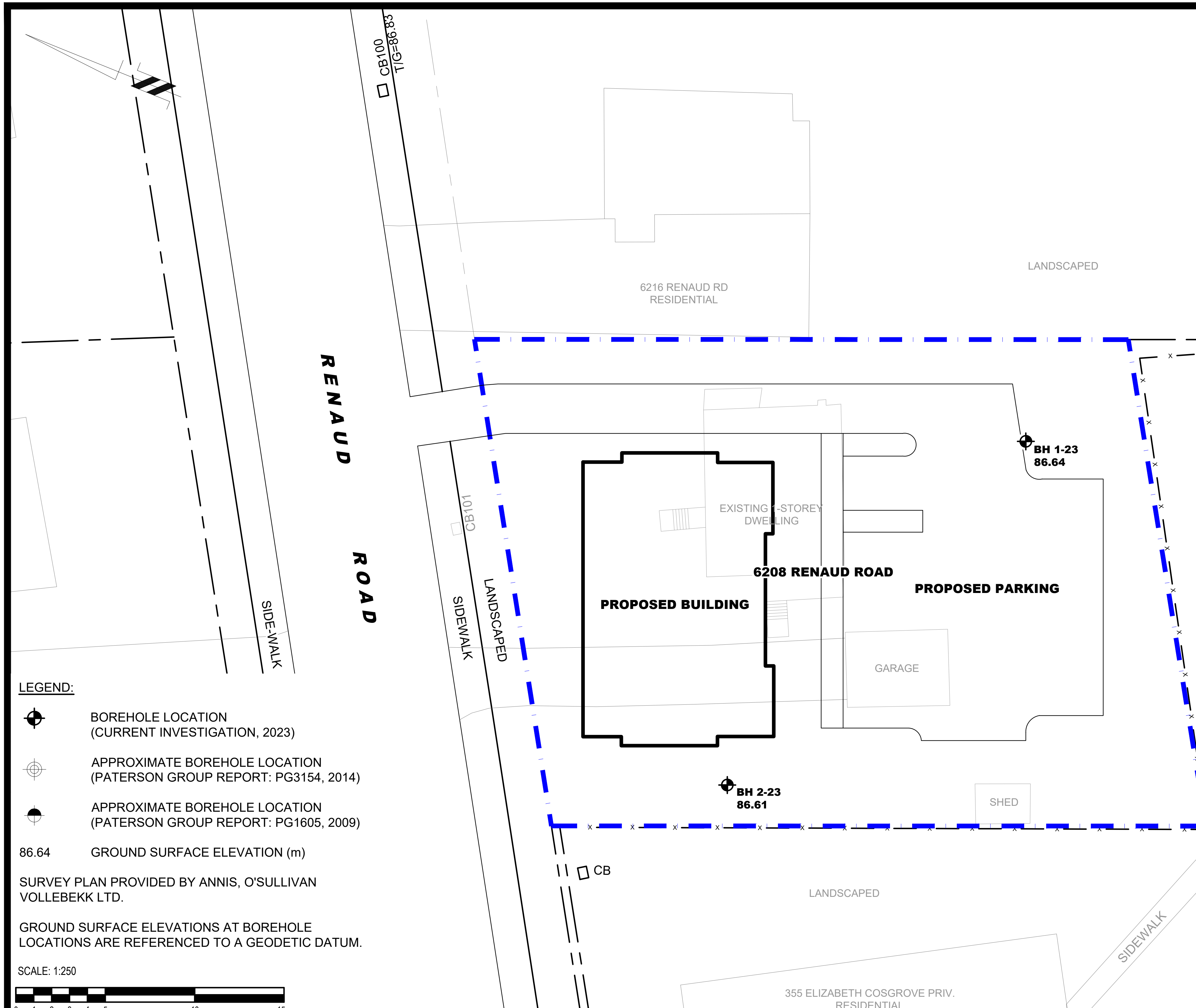
FIGURE 1 – KEY PLAN

DRAWING PG6640-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

BOREHOLE LOCATION (CURRENT INVESTIGATION, 2023)

APPROXIMATE BOREHOLE LOCATION (PATERSON GROUP REPORT: PG3154, 2014)

APPROXIMATE BOREHOLE LOCATION (PATERSON GROUP REPORT: PG1605, 2009)

86.64 GROUND SURFACE ELEVATION (m)

SURVEY PLAN PROVIDED BY ANNIS, O'SULLIVAN VOLLEBEKK LTD.

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

SCALE: 1:250

0 1 2 3 4 5 10 15m

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO NEW CONCEPTUAL PLAN	21/12/2023	NS

TM (262615) HOLDING INC., AND MARISSA & MATHIEU BRISEBOIS

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
6208 RENAUD ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 05/2023
Drawn by: GK	Report No.: PG6640-1
Checked by: NS	Dwg. No.: PG6640-1
Approved by: JV	Revision No.:

11x17

p:\autocad drawings\geotechnical\pg66xx\pg6640\pg6640-1-test hole location plan (rev.01).dwg