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Geotechnical Engineering

Environmental Engineering

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Noise and Vibration Studies

Geotechnical Investigation

Proposed Mixed-Use Development
1131-1151 Teron Road
Ottawa, Ontario

Prepared For

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11073656 Canada Inc.

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Revision 3

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by 11021028 Canada Inc. and 11073656 Canada Inc. to conduct a geotechnical investigation for the proposed new development to be located at 1131-1151 Teron Road, in the City of Ottawa, Ontario (Refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current geotechnical review were:

❑ to determine the subsurface soil and groundwater conditions based on the soils information.

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

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

2.0 Proposed Development

Based on the available information, it is understood that the proposed development will consist of two multi-storey mix-use buildings sharing one underground level, and located within the southern portion of the site. The easternmost building will have 3 floors and the other building will have nine floors. It is further understood that the proposed development will include associated at grade car parking areas, access lanes and landscaped areas. It is anticipated that the site will be municipally serviced. It is expected that the existing dwelling situated on site will be demolished as part of this project.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was conducted on March 13 and 16, 2020. At that time, a total of 6 boreholes were advanced to a maximum depth of 9.6 m. A previous geotechnical investigation was conducted on May 17 , 2012. At the time 3 boreholes were advanced to a maximum depth of 11.3 m. The locations of the test holes are shown on Drawing PG5283-1 - 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. All fieldwork was conducted under the full-time supervision of Paterson personnel from our geotechnical division under the direction of a senior engineer. The testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using a 50 mm diameter split spoon sampler. Soil samples were also recovered along the sidewalls of the test pits by hand during excavation.

All soil samples were visually inspected and initially classified on site. The split spoon samples were placed in sealed plastic bags. All samples were transported to our laboratory for examination and classification. The depths at which the split spoon and auger samples were recovered from the test holes are shown as SS and AU 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 carried out at regular depth intervals in cohesive soils using a field vane apparatus.

Overburden thickness was evaluated during the course of the site investigations by dynamic cone penetration testing (DCPT) at several borehole locations. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

Flexible standpipes were installed in all boreholes to monitor the groundwater levels subsequent to the completion of the sampling program. Groundwater infiltration levels were noted at the time of excavation at the borehole locations.

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground services. The location and ground surface elevation at each borehole location were also provided by Paterson. The elevations are referenced to the geodetic datum. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG5283-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One soil sample was submitted to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are discussed in Subsection 6.7 and shown in Appendix 1.

4.0 Observations

4.1 Surface Conditions

The subject site consist of 2 properties, 1131 Teron Road and 1151 Terron Road. The north property, 1151 Terron Road, is currently undeveloped land with a grass surface. Several trees were noted along the borders of the site with March Road and at the south portion. The site is generally at grade with respect to the adjacent roads bordering to the north (March Road) and to the southeast/south (Teron Road). It is further noted that two high voltage transmission lines cross the subject site from the northwest to the southeast corner.

The property to the south, 1131 Teron Road, is currently occupied by a residential dwelling with associated paved/gravel surfaced areas used for parking and access lanes. The property is generally flat and at grade with Teron Road. A small drainage ditch was noted on the north property line with 1151 Teron Road. Residential developments are situated to the east and south of the property.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consisted of a layer of topsoil, followed by a loose brown silty sand with clay. A very stiff brown silty clay crust was encountered underlying the silty sand. A very stiff to firm grey silty clay deposit was encountered below the brown clay crust. Practical refusal to DCPT was encountered at a depth of 16.6 m at BH 3-20 and 16.43 at BH 2. 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.

Silty Clay

Silty clay was encountered immediately beneath the loose silty sand layer at all test hole locations. The upper portion of the silty clay has been weathered to a very stiff brown crust. The crust extends to depths varying between 3.0 and 3.7 m. In situ shear vane field testing carried out within the silty clay layer in the lower portion of the weathered crust yielded undrained shear strength values ranging from approximately 110 to 154 kPa. These values are indicative of a very stiff consistency. Horizontal sand seams were noted in the brown silty clay crust at some test home locationsIn situ shear vane field testing carried out within the grey silty clay yielded undrained shear strengths ranging from approximately 33 to 110 kPa. These values are indicative of a firm to very stiff consistency.

Bedrock

Based on available geological mapping, the local bedrock consists of migmatic rocks of the Paragneiss formation with an anticipated overburden thickness of 15 to 25 m.

4.3 Groundwater

The groundwater levels were recorded by Paterson personnel and the readings are presented in Table 1.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH1-20	88.9	3.88	85.02	March 23, 2020
BH2-20	88.98	0.15	88.83	March 23, 2020
BH3-20	89.71	0.9	88.81	March 23, 2020
BH4-20	90.17	1.29	88.88	March 23, 2020
BH5-20	89.99	1.26	88.73	March 23, 2020
BH6-20	89.29	1.94	87.35	March 23, 2020

Note: The ground surface elevations at the borehole locations were surveyed by Paterson personnel.

It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole, which can lead to higher than typical groundwater levels. The long-term groundwater level can also be estimated based on moisture levels and colouring of the recovered soil samples. Based on these observations at the borehole locations, the long-term groundwater level is expected at a 3 to 4 m below ground surface.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered adequate for the proposed development. It is expected that the proposed structure on the south property can be founded over conventional shallow foundations placed on undisturbed, stiff to very stiff silty clay bearing surface. It is expected that the proposed multi-storey building on the north property can be founded on a raft foundation or a deep foundation system.

Due to the presence of the silty clay deposit, the subject site will be subject to permissible grade raise restrictions.

To complete the construction of the underground level, temporary shoring will be needed to support the neighboring buildings and roads. The design of the temporary shoring system needs to adequately support the existing low-rise buildings along the east and south side of the site, which are in close proximity with the proposed excavation.

The above and other consideration are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, containing significant amounts of deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the subgrade level during site preparation activities.

It is anticipated that the existing fill, free of deleterious material and significant amounts of organics, can be left in place below the proposed car parking areas and access lanes. However, it is recommended that the existing fill layer be proof-rolled using heavy vibratory equipment and approved by Paterson at the time of construction. Any poor performing areas noted during the proof-rolling operations should be removed and replaced with appropriate fill material.

Fill Placement

Fill used 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 material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings should be compacted to at least 98% of its 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 stiff brown silty clay, free of organics and deleterious materials, is to be used to build up the subgrade level for areas to be paved, the silty clay, under dry conditions, 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 use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

To protect the native silty clay bearing surface, it is expected that a granular working pad will be required and will have a minimum thickness of 600 mm. Alternatively, a 50 to 75 mm thick mudslab consisting of a minimum 15 MPA lean concrete can be used to cover the subgrade.

Proof Rolling

For the proposed parking areas and access lanes, proof rolling will be required in areas where the loose silty sand, free of deleterious materials, and approved by Paterson personnel at the time of construction is encountered at subgrade level. The purpose of the proof rolling is to induce some of the initial settlements to reduce long term total settlements. It is recommended that the subgrade surface be proof-rolled **under dry conditions** by an adequately sized roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by the geotechnical consultant at the time of construction.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, founded on an undisturbed, stiff to very stiff silty clay can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**.

Strip and pad footings founded on an undisturbed, loose silty sand can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **200 kPa**. It is recommended to proof roll the existing silty sand bearing surface prior to placement of footings. (See subsection).

A geotechnical resistance factor of 0.5 was incorporated into the above noted bearing resistance values at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed, in the dry, prior to the placement of concrete for footings.

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided herein will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to an undisturbed soil bearing surface above the groundwater table when a plane extending horizontally and vertically from the bottom edge of the footing at a minimum of 1.5H:1V passes through in situ soil of the same or higher capacity as the bearing medium soil.

Raft Foundation

Consideration can be given to a raft foundation to found the proposed multi-storey building. The factored bearing resistance (contact pressure) at ultimate limit states (ULS) can be taken to be **250 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The design should allow for the use of no more than 80% of the overconsolidation of the silty clay and account for a potential 0.5 m post-development groundwater lowering. A bearing resistance value at serviceability limit states (SLS) (contact pressure) of **160 kPa** can be used. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

The modulus of subgrade reaction was calculated to be **5.0 MPa/m** for a contact pressure of **160 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, sensitive silty clay is not elastic and limits have to be placed on the stress ranges of a particular modulus. This value can be re-evaluated once detail of the structural design becomes available.

Based on the above assumptions for the raft foundation, the proposed building can be designed using the above parameters and a total and differential settlement of 25 and 15 mm, respectively.

Deep Foundation

For support of the proposed multi-storey building consideration could be given to using concrete filled steel pipe piles driven to refusal on the bedrock surface.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. As a minimum, the pipe piles should be equipped with a base plate having a thickness of at least 20 mm to minimize damage to the pile tip during driving. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	910	1090	10	29
245	11	1050	1260	10	35
245	13	1250	1500	10	41

As a minimum, the pipe piles should be equipped with a base plate having a thickness of at least 20 mm to reduce potential damage to the pile tip during driving.

Provision should be made for restriking all of the piles at least once, 48 hours after the initial driving, to confirm the design set and/or the permanence of the set, and to check for upward displacement due to driving adjacent piles. It is recommended that a pile load test or dynamic monitoring and capacity testing be carried out at an early stage during the piling operations to verify the transferred energy from the piledriving equipment and determine the load carrying capacity of the piles. The recommended number of tests is dependent on the number of piles and pile sizes; as a guideline a minimum of 2 tests per pile size should be carried out. It is also recommended that the tested pile locations be spread out across the proposed building footprints.

The post construction settlement of structural elements which derive their support from piles bearing on bedrock should be negligible.

If piles are to be left exposed during winter months, some form of frost protection will be required to prevent frost adhesion and jacking of the piles. Further guidelines can be provided on these measures at the time of construction, if required.

Permissible Grade Raise Recommendations

A **permissible grade raise restriction of 2.0 m** can be used for design purposes. If greater 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.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the foundations considered at this site.The soils underlying the proposed shallow 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 fill, containing deleterious or organic materials, the native soil or engineered fill approved by the geotechnical consultant at the time of excavation will be considered to be an acceptable subgrade surface on which to commence backfilling for basement floor slab or slab on grade construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

It is recommended that the upper 200 mm of sub-floor fill for basement slabs consist of 19 mm clear crushed stone. The upper 300 mm of sub-floor fill below slabs on grade should consist of OPSS Granular A crushed stone. All backfill materials within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight. However, if a full drainage system is being implemented and approved by Paterson at the time of construction, hydrostatic pressure can be omitted in the structural design.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.28 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using

$P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

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

Table 3 - Recommended Pavement Structure - Car Only Parking Areas/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 in situ soils or OPSS Granular B Type I or II material placed over in situ soil or fill.	

Table 4 - Recommended Pavement Structure - Heavy Duty and Access Lanes

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
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

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

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 I or II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable 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

A perimeter foundation drainage system is recommended to be provided for the proposed structure. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The clear stone should be wrapped in a non-woven geotextile. The pipes should have a positive outlet, such as a gravity connection to a sump pit located in the lowest basement level or storm sewer.

Underfloor Drainage

Underfloor drainage system will be required to control water infiltration for the lower basement area. For preliminary design purposes, it is recommended that 150 mm perforated pipes should be placed in each bay. The spacing of the underfloor drainage system should be confirmed by the geotechnical consultant once design details for the proposed development are finalized and reviewed for suitability at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings, Pile Caps and Grade Beam Against Frost Action

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

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

The parking garage may require protection against frost action depending on the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

At this site, temporary shoring may be required to complete the required excavations. However, it is recommended that where sufficient room is available open cut excavation in combination with temporary shoring can be used.

Excavation Side Slopes

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

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

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

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

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

Temporary Shoring

If a temporary shoring system is considered, the design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner's representative prior to implementation.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced.

The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 5 - Soil Parameters for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	18
Submerged Unit Weight (γ), kN/m^3	11

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

6.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the brown silty clay or below, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 98% of its SPMDD. The bedding material should extent 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 225 mm thick lifts and compacted to a minimum of 98% of its SPMDD.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay 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 98% of the material's SPMDD.

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 and should extend from trench wall to trench wall. Generally, 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 compatible brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's 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

Groundwater Control for Building Construction

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. It is anticipated that groundwater infiltration into the excavations should be low and controllable using conventional open sumps.

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

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

Long-term Groundwater Control

The recommendations for the proposed building long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building perimeter or sub-slab drainage system will be directed to the proposed building cistern/sump pit. It is expected that groundwater flow will be low (i.e.- less than 30,000 L/day per building) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The groundwater flow should be controllable using conventional open sumps.

Impacts on Neighbouring Structures

Based on observations, the long term groundwater level is anticipated at depths below 3 - 4 m. A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The extent of any significant groundwater lowering should occur within a limited range of the subject site due to the minimal temporary groundwater lowering.

The neighbouring structures are expected to be founded within the brown silty clay crust bearing surface. No issues are expected, with respect to groundwater lowering, that would cause long term damage to adjacent structures surrounding the proposed building.

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 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 results of the chloride content, pH and resistively indicate the presence of a moderate to aggressive environment for exposed ferrous metals at this site.

6.8 Landscaping Considerations

Tree Planting Restrictions

According to the City of Ottawa Guidelines for tree planting, where a sensitive silty clay deposit is present within the vicinity of the site, tree planting restrictions should be determined. However, for this site, based on the founding medium of the underground level which will occupy the majority of the site where buildings are located, tree planting restrictions are not required from a geotechnical perspective.

6.9 Storm Water Detention Cistern and Bioswales

Storm Water Detention Cistern

Based on the available site servicing drawings, it is understood that storm water detention cistern is proposed along the western corner of the 9-storey building. The cistern will have an approximate volume of 66 m^3 and it will be attached to the proposed building. The top of the cistern will be at approximate geodetic elevation 89.3 m and the bottom of the cistern will be at approximate geodetic elevation 88.5m. The finish floor level for the basement of the adjacent 9 storey building will be at geodetic elevation of 86.7m. The finish grade level at the location of the cistern will be at approximate geodetic elevation of 90.8m. Therefore, the cistern will be fully buried with a soil cover of approximately 2.3 m above the top of the cistern. Furthermore, due to the founding depth and the depth of the long-term groundwater level, frost protection and waterproofing will not be required for the proposed storm water cistern.

Based on the founding level of the cistern and the finish floor level of the adjacent 9 storey building, the minimum vertical separation between the bottom of the cistern and the USF of the adjacent basement wall is anticipated to be approximately 1.8 m. The loads resulting from the cistern shall be taken into account in the design of the basement wall of the building in contact with the proposed storm water cistern. The cistern will exert a lateral hydrostatic pressure on the portion of the wall above the founding level of the cistern.
On the other hand, the lateral component of the cistern surcharge shall be added to the lateral earth pressure acting on the basement wall for the portion of the wall below the founding level of the cistern. Further details on the design of the basement wall are discussed in section 5.6.

Due to the difference in elevation between the founding depth of the cistern and the 9 storey building, it is recommended that the cistern be founded on OPSS Granular A or Granular B Type II extending to the founding level of the adjacent basement wall and compacted to a minimum 98% of the material’s SPMDD.

Bioswales

It is understood that bioswales are being considered for this development. According to the available drawings, the shallow bioswales will drain water into the proposed catch basins that are to be located at the center of each infiltration trench. It is further understood that the bottom of the proposed bioswales will be at a minimum geodetic elevation of 88.66 m. According to the groundwater level observations during the current geotechnical investigation, the long-term groundwater level is expected to be at 3 to 4m below existing ground surface, which is at approximate geodetic elevation of 85.5 to 84.5m.

Furthermore, based on available geological mapping and on refusal to DCPT during the previous geotechnical investigation, the subsurface conditions within 1m below the bottom of the bioswales consists of silty sand and/or silty clay deposit. Therefore, the proposed bioswale is considered to have a minimum separation of 1.0m above the long-term groundwater table and the bedrock surface. The bioswales are considered suitable for the proposed development from a geotechnical perspective

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation services program including the following aspects be performed by the geotechnical consultant.

☐ Review of the final grading plan from a geotechnical perspective, once available.

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

☐ Sampling and testing of the concrete and fill materials used.

☐ 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.

☐ Observation of the placement of foundation insulation, if applicable.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test hole locations, we request 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 our 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 11021028 Canada Inc. and 11073656 Canada Inc. 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
Dec 20, 2021
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Joey R. Villeneuve, M.A.Sc, P.Eng.

David J. Gilbert, P.Eng

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1131 and 111 Teron Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2020 March 16

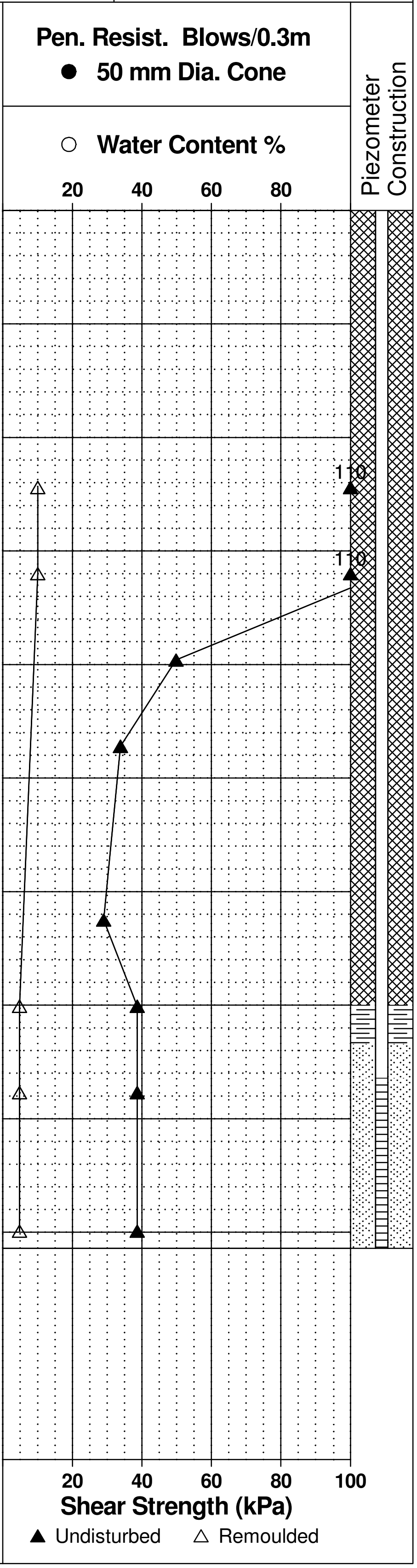
FILE NO. PG5283

HOLE NO. BH 1-20

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.13
Very loose, brown **SILTY SAND** with clay, trace organics	1.52
Very stiff, brown **SILTY CLAY,** trace sand - grey by 3.0m depth - stiff to firm by 3.8m depth	9.14
End of Borehole	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	92	3
SS	3	100	4
SS	4	100	W

DEPTH (m)	ELEV. (m)
0	88.91
1	87.91
2	86.91
3	85.91
4	84.91
5	83.91
6	82.91
7	81.91
8	80.91
9	79.91



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1131 and 111 Teron Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

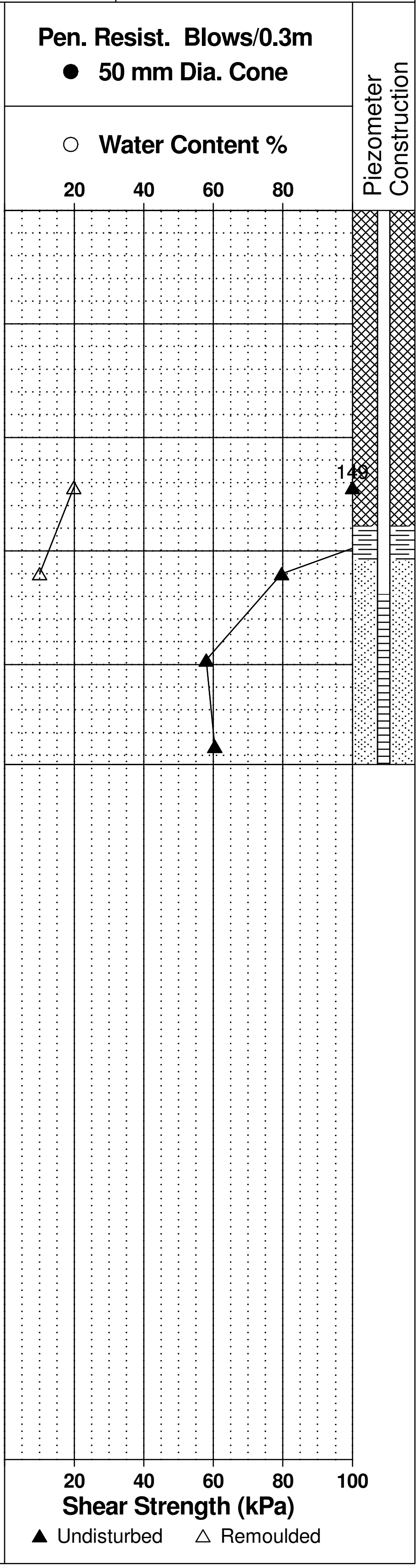
BORINGS BY CME 55 Power Auger

DATE 2020 March 16

FILE NO. PG5283

HOLE NO. BH 2-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	88.98
TOPSOIL 0.15		AU	1				
Loose, brown **SILTY SAND** with clay, trace organics 1.52		SS	2	75	5	1	87.98
Very stiff to stiff, brown **SILTY CLAY,** trace sand - grey by 3.0m depth 4.88		SS	3	100	4	2	86.98
						3	85.98
						4	84.98
End of Borehole							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1131 and 111 Teron Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2020 March 16

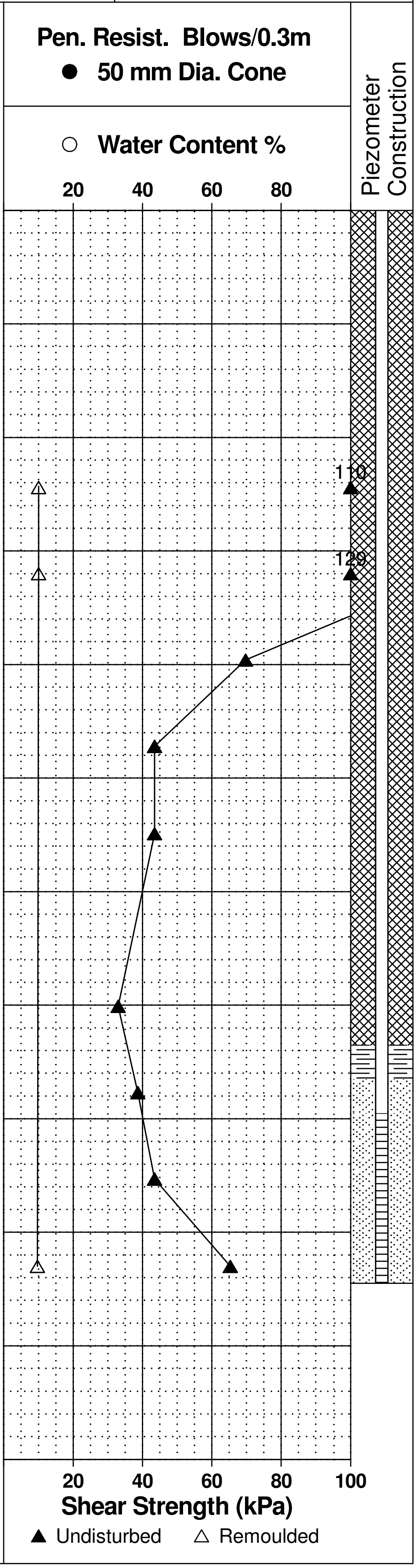
FILE NO. PG5283

HOLE NO. BH 3-20

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOI	0.10
Loose, brown **SILTY SAND** with clay, trace organics	1.52
Very stiff to stiff, brown **SILTY CLAY,** trace sand - grey by 3.0m depth - firm to stiff by 4.6m depth	9.45
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 16.2m depth. Inferred **SILTY CLAY**	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	4
SS	3	100	3
SS	4	100	W

DEPTH (m)	ELEV. (m)
0	89.71
1	88.71
2	87.71
3	86.71
4	85.71
5	84.71
6	83.71
7	82.71
8	81.71
9	80.71
10	79.71
11	78.71



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1131 and 111 Teron Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

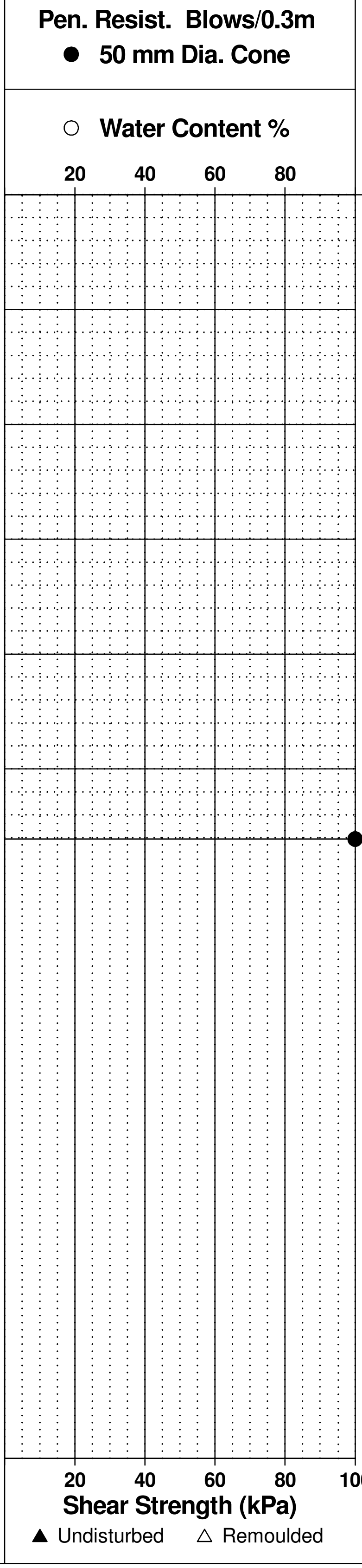
BORINGS BY CME 55 Power Auger

DATE 2020 March 16

FILE NO. PG5283

HOLE NO. BH 3-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						11	78.71	
						12	77.71	
						13	76.71	
Inferred **SILTY CLAY**						14	75.71	
						15	74.71	
						16	73.71	
16.61								
End of Borehole Practical DCPT refusal at 161.61m depth.								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1131 and 111 Teron Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5283**

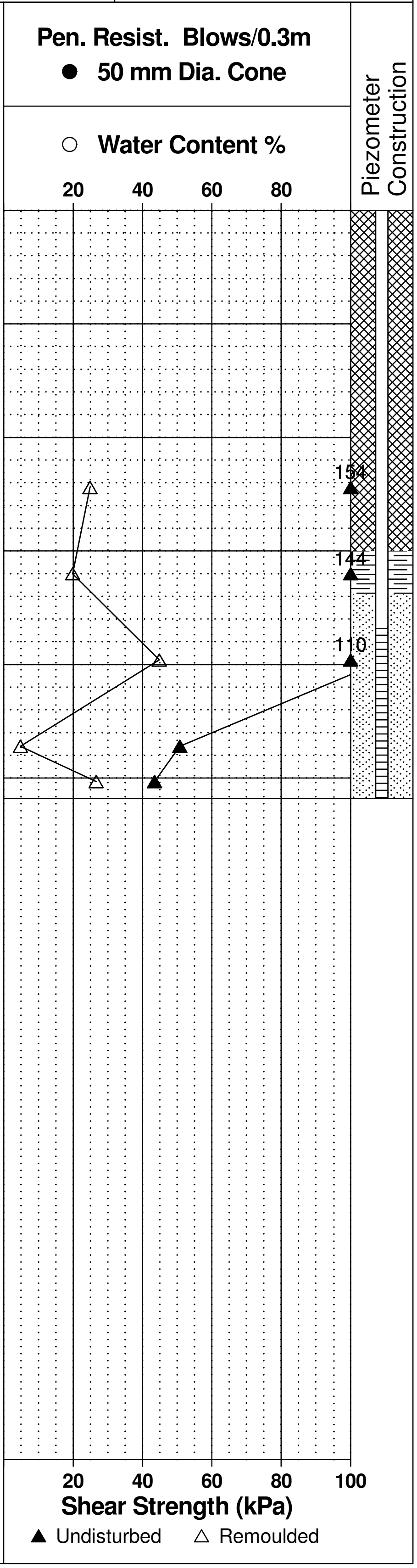
REMARKS

HOLE NO. **BH 4-20**

BORINGS BY CME 55 Power Auger

DATE 2020 March 13

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	90.17
TOPSOIL 0.05		AU	1				
Asphaltic concrete 0.10							
FILL: Brown sand with crushed stone 0.76							
		SS	2	67	3	1	89.17
Brown **SANDY SILT,** some clay 1.83							
		SS	3	92	3	2	88.17
Very stiff, brown **SILTY CLAY**						3	87.17
- very stiff to firm and grey by 3.7m depth						4	86.17
5.18						5	85.17
End of Borehole							



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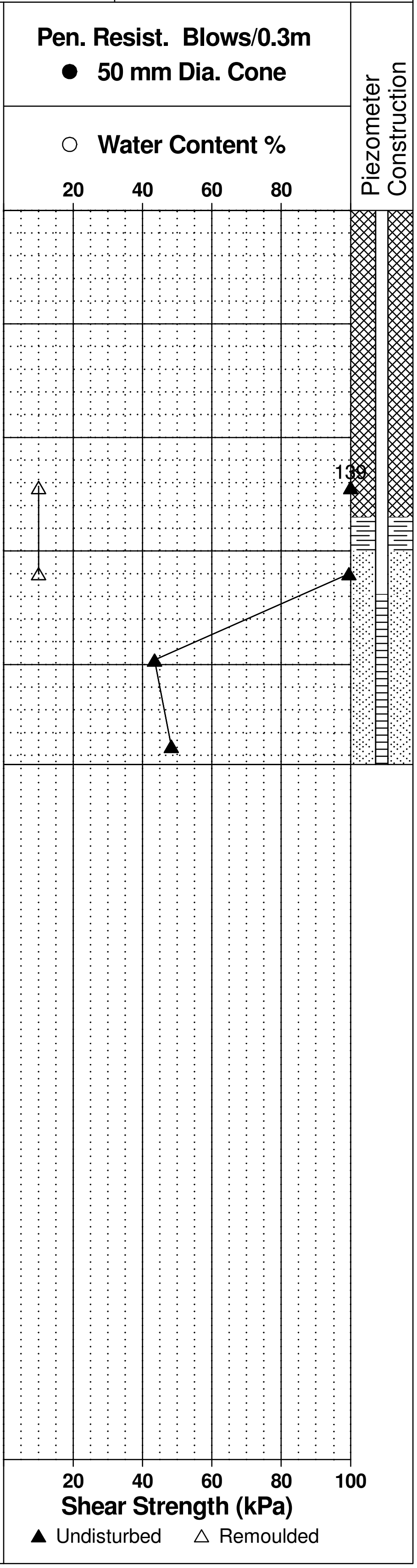
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1131 and 111 Teron Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME 55 Power Auger
DATE 2020 March 16
FILE NO. PG5283
HOLE NO. BH 5-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.29
TOPSOIL	0.10						
FILL: Brown silty sand with clay, trace gravel	0.60	AU	1				
Brown **SILTY SAND** with clay	1.52	SS	2	58	4	1	88.29
Very stiff to stiff, brown **SILTY CLAY**		SS	3	88	5	2	87.29
						3	86.29
- firm and grey by 3.8m depth	4.88					4	85.29
End of Borehole							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1131 and 111 Teron Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5283**

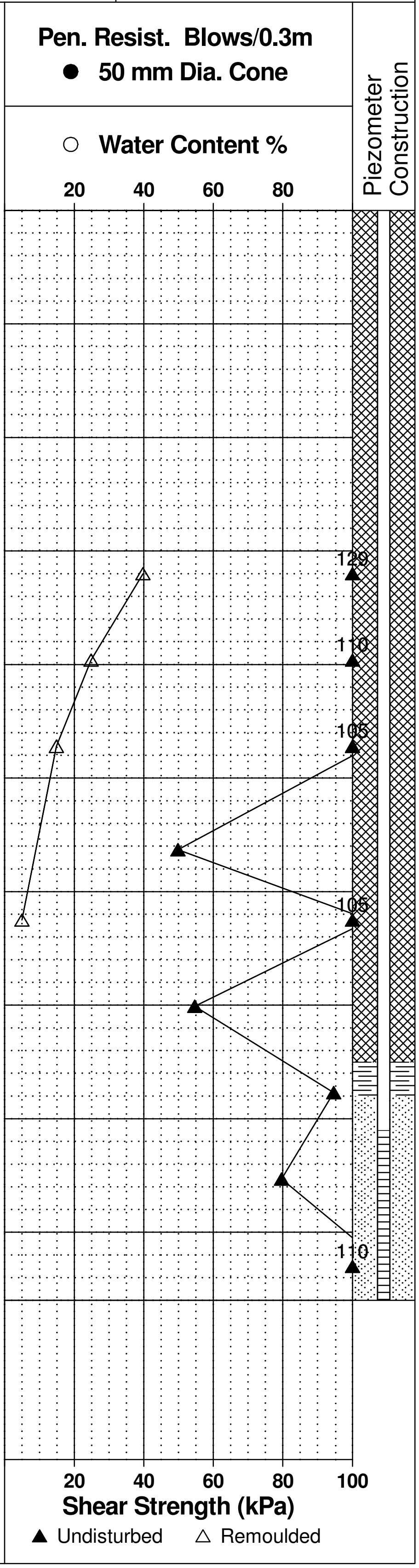
REMARKS

HOLE NO. **BH 6-20**

BORINGS BY CME 55 Power Auger

DATE 2020 March 13

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.99
TOPSOIL 0.08		AU	1				
Brown **SILTY SAND** with clay 1.32		SS	2	33	3	1	88.99
Brown **SILTY CLAY** with sand seams 2.21		SS	3	83	3	2	87.99
Very stiff to stiff, brown **SILTY CLAY** - grey by 3.7m depth - with sand seams by 5.3m depth		SS	4	100	3	3	86.99
						4	85.99
						5	84.99
		SS	5	83	W	6	83.99
						7	82.99
						8	81.99
9.60						9	80.99
End of Borehole							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - 1131 Teron Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site on Teron Road. Geodetic elevation = 91.265m.

REMARKS

BORINGS BY CME 55 Power Auger

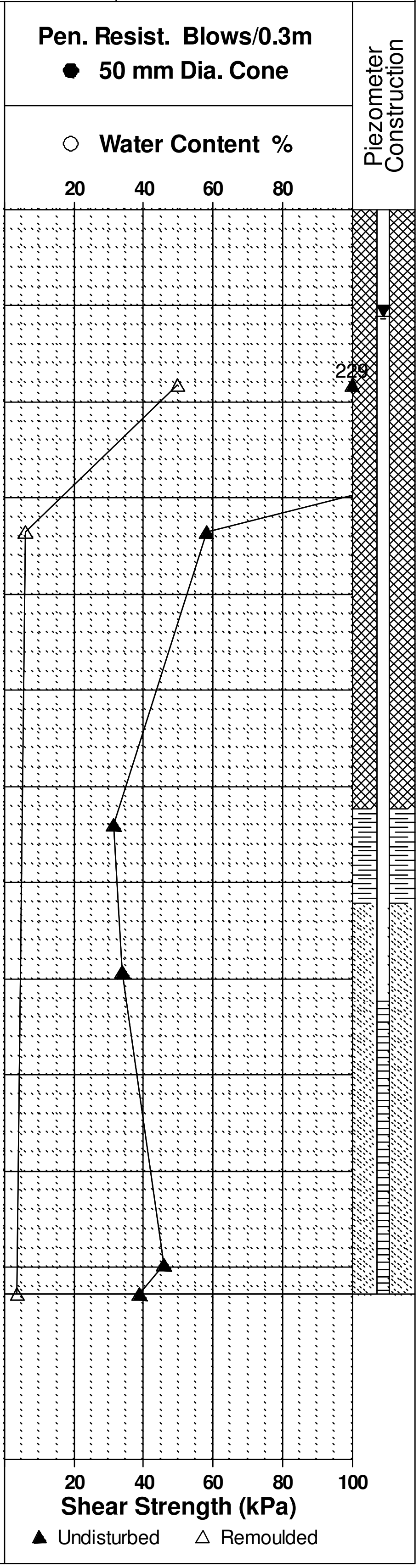
DATE May 17, 2012

FILE NO. PG2622

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.25		AU	1		
Brown **SILTY SAND** 0.69		AU	2		
Hard to very stiff, brown **SILTY CLAY** - stiff to firm and grey by 2.4m depth		SS	3	83	9
11.28					
End of Borehole (GWL @ 1.11m-May 23, 2012)					

DEPTH (m)	ELEV. (m)
0	89.53
1	88.53
2	87.53
3	86.53
4	85.53
5	84.53
6	83.53
7	82.53
8	81.53
9	80.53
10	79.53
11	78.53



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - 1131 Teron Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site on Teron Road. Geodetic elevation = 91.265m.

REMARKS

BORINGS BY CME 55 Power Auger

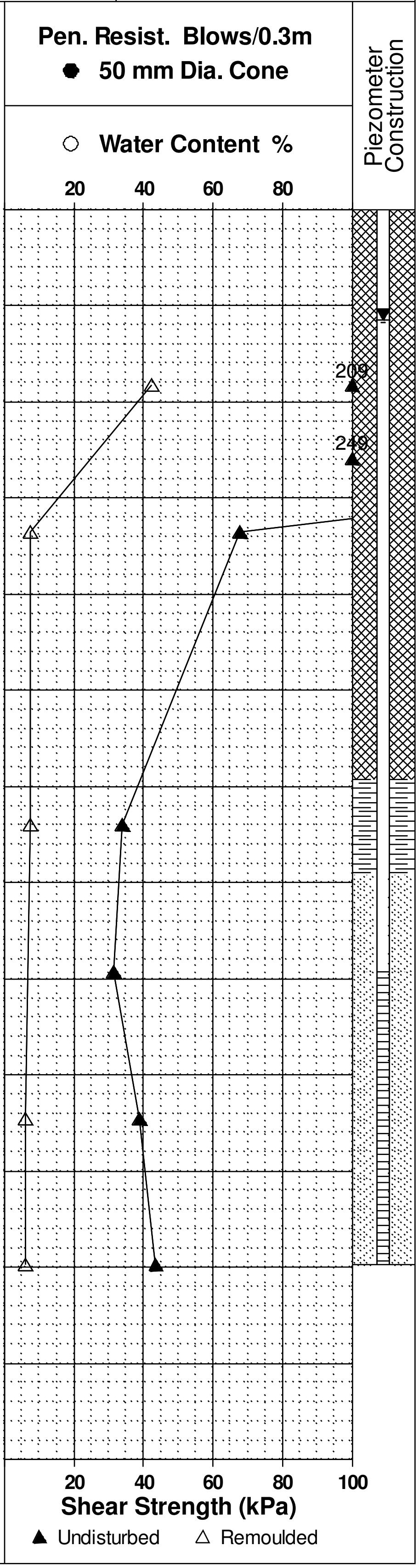
DATE May 17, 2012

FILE NO. PG2622

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.23					
Brown **SILTY SAND** 0.60		AU	1		
Brown **SILTY CLAY** with sand seams 1.45		SS	2	83	9
Hard to very stiff, brown **SILTY CLAY** - stiff to firm and grey by 2.8m depth 10.97					
Dynamic Cone Penetration Test commenced at 10.97m depth. Cone pushed to 16.2m depth.					

DEPTH (m)	ELEV. (m)
0	89.38
1	88.38
2	87.38
3	86.38
4	85.38
5	84.38
6	83.38
7	82.38
8	81.38
9	80.38
10	79.38
11	78.38
12	77.38
13	76.38



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - 1131 Teron Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site on Teron Road. Geodetic elevation = 91.265m.

REMARKS

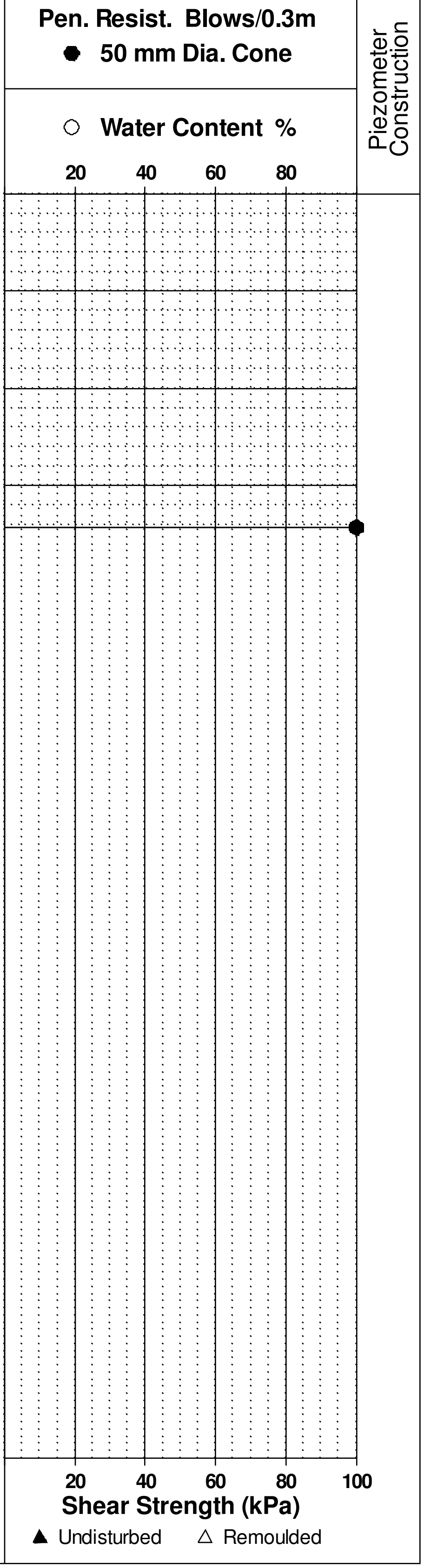
BORINGS BY CME 55 Power Auger

DATE May 17, 2012

FILE NO. PG2622

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						13	76.38
						14	75.38
						15	74.38
						16	73.38
16.43							
End of Borehole Practical DCPT refusal at 16.43m depth (GWL @ 1.15m-May 23, 2012)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - 1131 Teron Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site on Teron Road. Geodetic elevation = 91.265m.

REMARKS

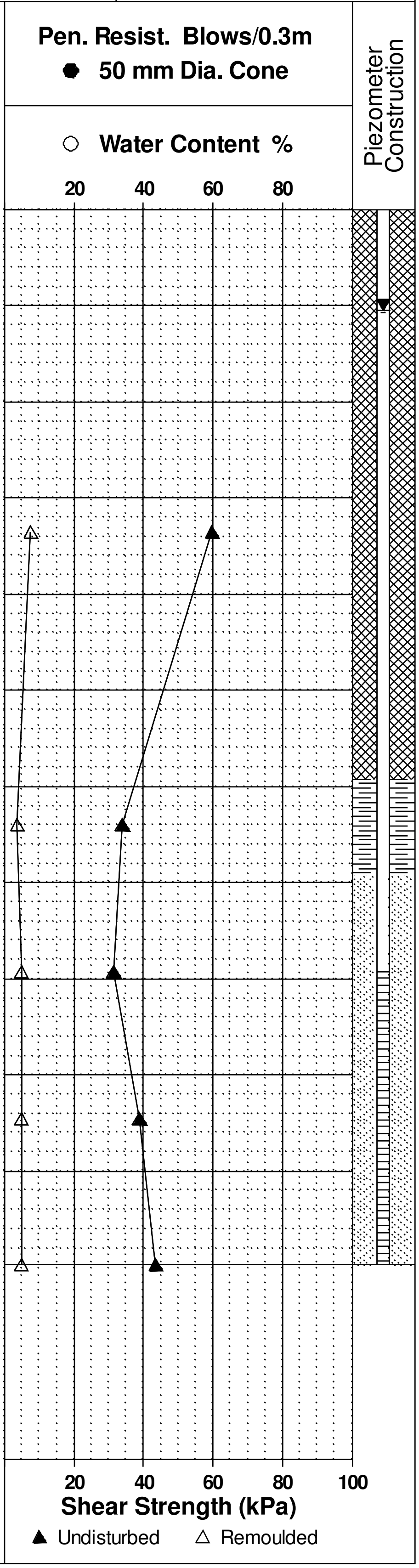
BORINGS BY CME 55 Power Auger

DATE May 17, 2012

FILE NO. PG2622

HOLE NO. BH 3

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.35
TOPSOIL	0.25						
Brown **SILTY SAND**		AU	1				
		SS	2	83	13	1	88.35
	1.45						
Very stiff to stiff, brown **SILTY CLAY**		SS	3		7	2	87.35
- stiff to firm and grey by 2.6m depth						3	86.35
						4	85.35
						5	84.35
						6	83.35
						7	82.35
						8	81.35
						9	80.35
						10	79.35
	10.97						
End of Borehole							
(GWL @ 1.05m-May 23, 2012)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

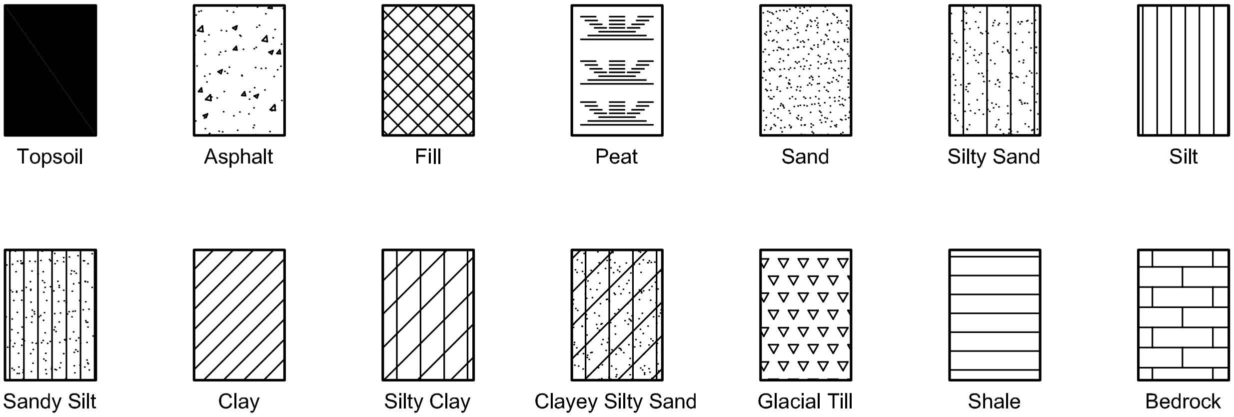
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

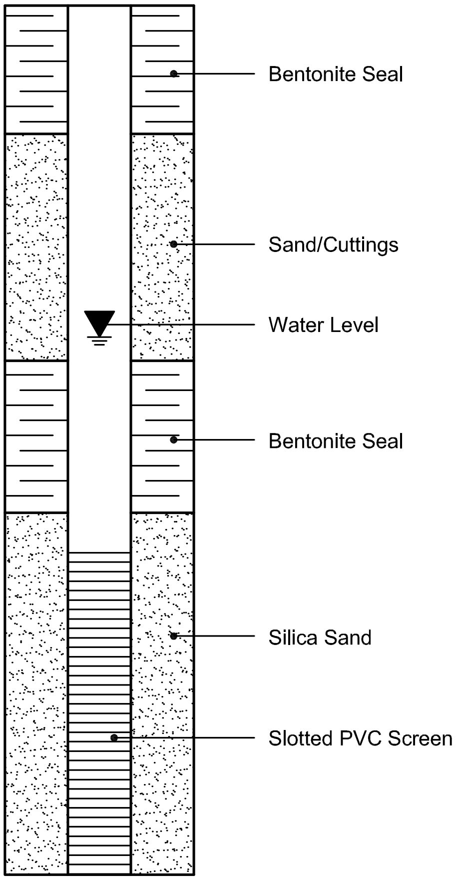
SYMBOLS AND TERMS (continued)

STRATA PLOT

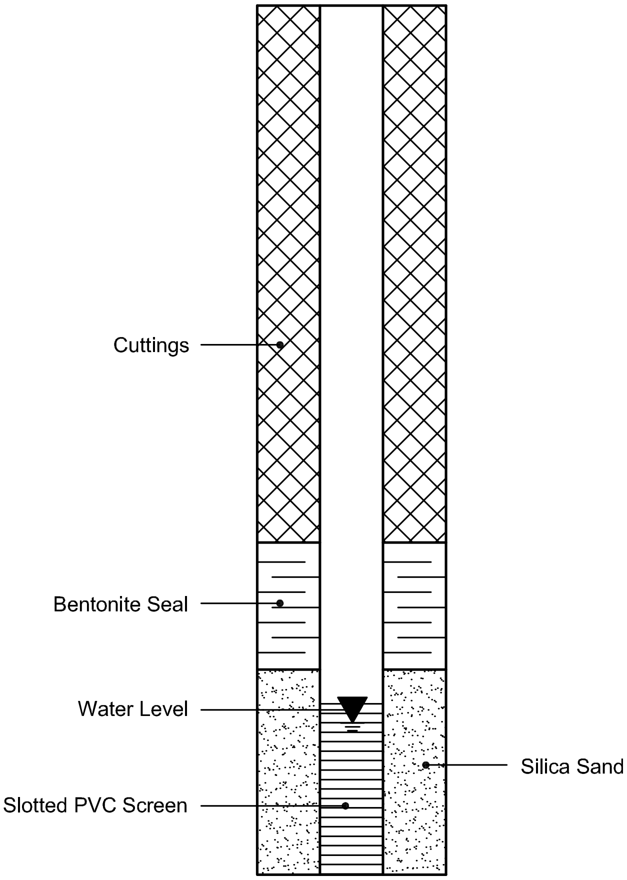


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29675

Report Date: 20-Mar-2020

Order Date: 16-Mar-2020

Project Description: PG5283

	Client ID:	BH 6-20 SS4	-	-	-
	Sample Date:	13-Mar-20 10:00	-	-	-
	Sample ID:	2012089-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	65.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.07	-	-	-
Resistivity	0.10 Ohm.m	39.4	-	-	-
Anions					
Chloride	5 ug/g dry	40	-	-	-
Sulphate	5 ug/g dry	132	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5283-1 - TEST HOLE LOCATION PLAN

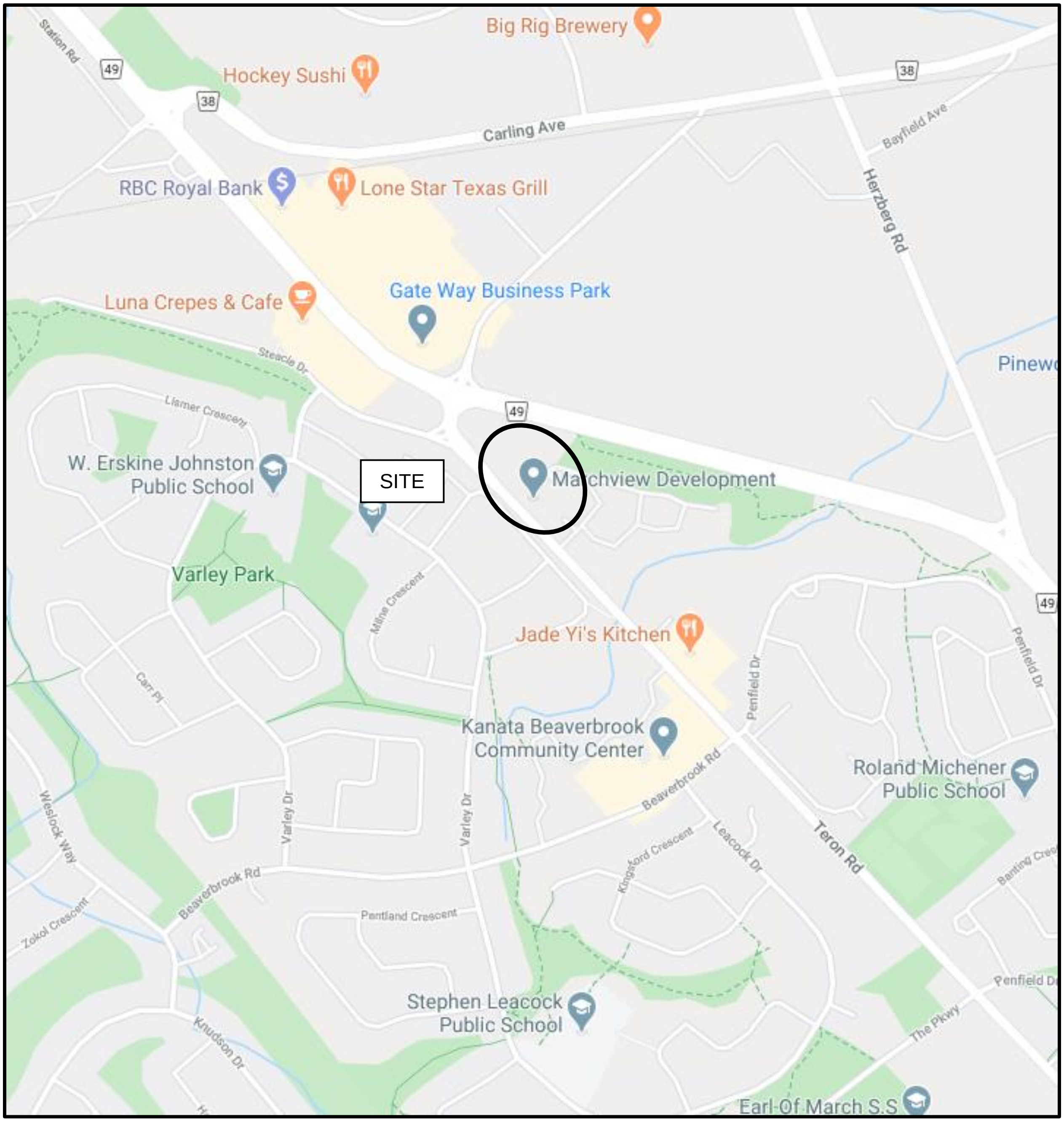
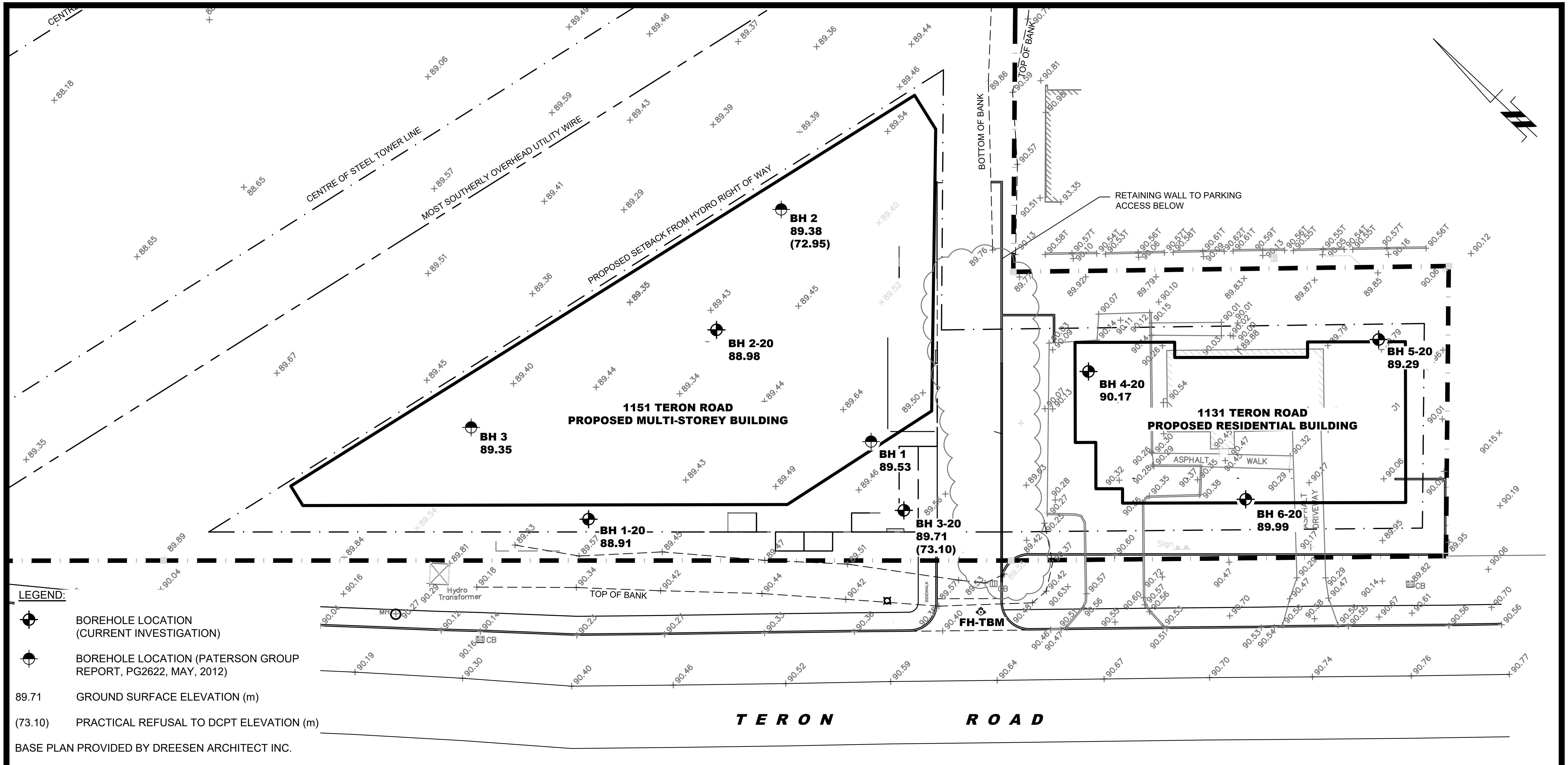


FIGURE 1

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION (CURRENT INVESTIGATION)
⊕	BOREHOLE LOCATION (PATERSON GROUP REPORT, PG2622, MAY, 2012)
89.71	GROUND SURFACE ELEVATION (m)
(73.10)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

BASE PLAN PROVIDED BY DREESEN ARCHITECT INC.

CONCEPTUAL PLAN PROVIDED BY NEUF ARCHITECTS

TBM- TOP SPINDLE OF FIRE HYDRANT.
GEODETIC ELEVATION= 91.27m

SCALE: 1:400

0 5 10 15 20 25m

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consulting engineers

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Ottawa, Ontario K2E 7J5
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NO.	REVISIONS	DATE	INITIAL

11021028 AND 11073656 CANADA INC.

GEOTECHNICAL INVESTIGATION
1131 AND 1151 TERON ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400
Drawn by: YA
Checked by: JV
Approved by: DJG
Date: 03/2020
Report No.: PG5283-1
Dwg. No.: PG5283-1
Revision No.:

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