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Geotechnical Engineering

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Geotechnical Investigation

Proposed Residential Development
Kanata Highlands - Phase 1
Terry Fox Drive
Ottawa, Ontario

Prepared For

Richcraft Homes

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September 24, 2013

Report: PG2971-1

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APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Richcraft Homes (Richcraft) to conduct a geotechnical investigation for Phase 1 of the proposed Kanata Highlands residential development located on Terry Fox Drive, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan, presented in Appendix 2).

The objectives of the investigation were to:

- ☐ determine the subsoil and groundwater conditions at this site by means of test holes; and
- ☐ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its 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

It is expected that the current phase of the development will consist predominately of residential buildings, parking areas, local roadways and landscaping areas. It is also anticipated that the proposed development will be serviced by future municipal services.

The subject property is located on the east side of Terry Fox Drive approximately 1.2 kilometers northwest of Richardson Side Road. The site is bordered to the west by Terry Fox Drive and to the east by a road allowance (not traveled).

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

The field program for the current geotechnical investigation was carried out on May 23 and 24, 2013. At that time, a total of nine (9) test pits were advanced to depths varying between 0.8 to 6.1 m below ground surface. Relevant historical boreholes (BH 12 and BH 13) completed during the previous geotechnical field investigation were completed on May 25, 2006. The test hole locations were distributed across the site in a manner to provide general coverage at the proposed development. The locations of the test holes and the ground surface elevation at each test hole location are shown in Drawing PG2971-1 - Test Hole Location Plan, included in Appendix 2.

The test pits were completed using a hydraulic shovel and the boreholes completed during our previous investigation were put down using a track-mounted auger drill rig operated by a crew of two. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The test pitting and drilling procedure consisted of excavating or augering to the required depths at the selected locations and sampling the overburden. Sampling and testing the overburden was completed in general accordance with ASTM D5434-12 - Guide for Field Logging of Subsurface Explorations of Soil and Rock.

Sampling and In Situ Testing

Soil samples from the test pits were recovered from the side walls of the open excavation. The soil samples recovered from the boreholes were recovered using a 50 mm diameter split-spoon sampler. All soil samples were classified on site, placed in sealed plastic bags and transported to our laboratory for further review. The depths at which the split spoon and grab samples were recovered from the test holes are shown as SS and G, 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. Undrained shear strength testing in test pits was completed using a handheld, portable vane apparatus (field inspection vane tester Roctest Model H-60). Undrained shear strength testing in boreholes was completed using a MTO field vane apparatus. This testing was done in general accordance with ASTM D2573-08 - Standard Test Method for Field Vane Shear Test in Cohesive Soil.

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

Groundwater

Groundwater infiltration levels were noted during the excavation of the test pits. Our groundwater infiltration observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson personnel to provide general coverage of the site. The test holes were located in the field and surveyed by Annis O'Sullivan Vollebekk Limited. It is understood that the ground surface elevations are referenced to a geodetic datum.

The locations of the test holes and the ground surface elevation at each test hole location are presented in Drawing PG2971-1 - Test Hole Location Plan included in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging. The subsurface soils were classified in general accordance with ASTM D2488-09a, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).

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 soil. The results are provided in Appendix 1, and are discussed further in Subsection 6.7.

Paracel Laboratories (Paracel), of Ottawa, performed the laboratory analysis of the soil sample submitted for analytical testing. Paracel is a member of the Standards Council of Canada/Canadian Association for Environmental Analytical Laboratories (SCC/CAEAL). Paracel is accredited and certified by SCC/CAEAL for specific tests registered with the association.

The following testing guidelines were utilized for the submitted soil sample. The anions were analyzed using EPA 300.1, the pH was analyzed using EPA 150.1, the resistivity was analyzed using EPA 120.1, and the percent solids was determined using gravimetrics.

4.0 OBSERVATIONS

4.1 Surface Conditions

The majority of the site is densely treed with some agricultural land located at the southwest portion of the site. Several mulch and fill piles were noted within the central portion of agricultural land. The ground surface across the site was undulating and slopes upward to the east. Bedrock outcrops are present within the north and east portions of the site and within several areas along the south portion of the subject site.

4.2 Subsurface Profile

Generally, the soil conditions encountered at the test hole locations consist of topsoil overlying a silty clay deposit, glacial till and/or bedrock. Bedrock was encountered at TP 4, TP 5, TP 6, TP 7, BH 12 and BH 13 at depths varying between 0.8 to 5.5 m below existing ground surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each test hole location.

Silty Clay

Silty clay was encountered below the topsoil at all test hole locations. The upper portion of the silty clay is weathered to a brown crust, which extends to a maximum depth of 4.9 m. In situ shear vane field testing carried out within the silty clay crust yielded peak undrained shear strength values ranging from approximately 100 to greater than 150 kPa, which are indicative of a very stiff consistency.

Grey silty clay was encountered below the weathered silty clay crust at TP 1, TP 2, TP 3, TP 8, TP 9 and BH 13. In situ shear vane tests conducted within the grey silty clay layer generally yielded undrained shear strength values ranging from 40 to 90 kPa. These values are indicative of a firm to stiff consistency.

Glacial Till

A compact to dense glacial till, consisting of a brown silty clay with sand, gravel, cobbles and some boulders, was encountered underlying the silty clay deposit at TP 4, TP 5, TP 7 and BH 13 at depths varying between 0.7 and 4.9 m below existing ground surface.

Bedrock

Bedrock was encountered at TP 4, TP 5, TP 6, TP 7, BH 12 and BH 13 at depths varying between 0.8 to 5.5 m below existing ground surface. Bedrock outcrops were observed within the north and east portions of the site and within several areas along the south portion of the subject site.

A limited survey of bedrock outcrop locations was completed by Annis, O'Sullivan, Vollebekk and a representative from our geotechnical department. The results of the survey are presented in Drawing PG2971-1 - Test Hole Location Plan in Appendix 2. Due to the existing tree canopy a significant portion of the site was not surveyed.

Based on available geological mapping, the depth of the bedrock surface is expected to range from ground surface to 15 m below ground surface at the site area. The bedrock at the site area consists of metamorphic and igneous bedrock.

4.3 Groundwater

The groundwater infiltration levels were measured in the open test pits upon completion of the sampling program during our field investigation. The observed groundwater levels are presented on the Soil Profile and Test Data sheets in Appendix 1. The long-term groundwater level can also be estimated based on the moisture content, observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater level is expected between 4 to 5 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed development. It is expected that the proposed buildings will be founded by conventional style footings placed over in situ soils, bedrock surface and/or engineered fill.

It is expected that bedrock removal will be required in areas across the site for building construction and service installation.

A permissible grade raise restriction is required where the silty clay deposit is present below the proposed buildings. If higher 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.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only a small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

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. 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 building areas 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 and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. 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.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. This material should be used structurally only to build up the subgrade for roads and pavements. Where the fill is open-graded, a blinding layer of finer granular fill or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be determined at the time of construction.

5.3 Foundation Design

Bearing resistance values are provided in Table 1 for footings placed on an undisturbed, silty clay, glacial till or clean, bedrock bearing surface. Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed over an undisturbed, stiff silty clay bearing surface can be designed using the values provided in Table 1.

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

Footings designed using the bearing resistance values at SLS provided in Table 1 will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings placed on clean, surface sounded bedrock will be subjected to negligible settlements.

Table 1 - Bearing Resistance Values

Bearing Surface	Bearing Resistance Values at SLS (kPa)	Factored Bearing Resistance Values at ULS (kPa)
Stiff silty Clay	125	175
Compact glacial till	150	225
Clean surface sounded bedrock	500	1000
Engineered Fill placed on in situ soil and/or bedrock	150	225

Notes:
- ❏ ULS - Ultimate Limit States
- ❏ SLS - Serviceability Limit States
- ❏ A geotechnical resistance factor of 0.5 was applied to the provided bearing resistance values at ULS

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

Where fill is required to raise the grade below the footing level, the fill located within the zone of influence of the footings should consist of engineered fill, as described under Subsection 5.2.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the insitu soils above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil. In sound unfractured bedrock, a 1H:6V slope may be used.

Settlement and Grade Raise Restrictions

Consideration must be given to potential settlements, which could occur due to the presence of the silty clay deposit, 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, consisting of the unfactored dead load (DL) and a portion of the unfactored live load (LL). Generally, it is recommended to consider about 50% of the live load.

A permissible grade raise restriction of **1.5 m** is recommended for the proposed buildings where silty clay is encountered at underside of footing elevation. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise restriction calculations.

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

If higher grade raises and/or higher loading conditions are required, post construction settlements can be reduced by several methods, such as:

❏ preloading and surcharging
❏ lightweight fill (LWF)

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the shallow foundations bearing over the silty clay deposit. It should also be noted that where footings are founded over a glacial till bearing surface or directly on the bedrock surface, Part 9 of the Ontario Building Code (2006) can be used for design purposes. The soils underlying the subject site are not susceptible to liquefaction.

Reference should be made to the latest revision of the 2006 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed buildings, the native soil surface or engineered fill will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction. Provision should be made for proof-rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any fill. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 150 to 200 mm of sub-slab fill consists of a 19 mm clear crushed stone.

5.6 Pavement Design

For design purposes, the pavement structure presented in the following tables could be used for the design of driveways, car parking areas and access lanes/local residential streets.

Table 2 - Recommended Pavement Structure - Driveways and Car Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** - HL 3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil, bedrock or fill.	

Table 3 - Recommended Pavement Structure - Local Residential Roadways

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

Table 4 - Recommended Pavement Structure - Roadways with Bus Traffic

Thickness mm	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** - Superpave 19.0 Asphaltic Concrete
50	**Lower Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
600	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either 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 I or II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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

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

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

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

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

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum 1.5 m thick soil cover (or equivalent) should be provided in this regard. For footings founded directly on sound bedrock where insufficient soil cover is available, the suggested insulation can be omitted.

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

Frost Susceptibility of Bedrock

When bedrock is encountered above the proposed founding depth and soil frost cover is less than the 1.5 m, the frost susceptibility of the bedrock should be determined. This can be accomplished as follows:

- ☐ Drill probeholes within the bedrock and assess its frost susceptibility.
- ☐ Examine service trench profiles extending in bedrock in the vicinity of the foundation to determine if weathering is extensive.

If the bedrock is considered to be **non-frost susceptible**, the footings can be poured directly on the bedrock without any further frost protective measures.

If the bedrock is considered to be **frost susceptible**, the following measures should be implemented for frost protection:

☐ Option A - Subexcavate the weathered bedrock to sound bedrock or to the required frost cover depth. Pour footings at the lower level.

☐ Option B - Use insulation to protect footings. It is preferable to pour footings on the insulation overlying weathered bedrock. However, due to potential undulating bedrock surface, consideration may have to be given to adopting an insulation detail that allows the footing to be poured directly on the weathered bedrock.

6.3 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.

In bedrock, almost vertical side slopes can be used provided that all loose rock and blocks with unfavourable weak planes are removed or stabilized.

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

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

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

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. 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. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

It should generally be possible to re-use the organic free, moist (not wet) overburden material above the cover material. Wet materials will be difficult to re-use as their high water contents make compacting these materials impractical without an extensive drying period. All stones greater than 300 mm in their longest dimension and other deleterious materials should be removed prior to reusing these materials.

Well fractured bedrock should be acceptable as backfill provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones 300 mm or larger in their longest dimension are removed. Where blast rock is used a blinding layer (OPSS Granular A crushed stone) or a geotextile may be required above the blast rock to reduce the loss of fine particles within the voids of the rock fill.

Based on the soil profile encountered, the subgrade for the underground municipal services will be placed in both bedrock and in overburden soils. It is recommended that the subgrade medium be inspected in the field to determine how steeply the bedrock surface, where encountered, drops off. A transition treatment should be provided where the bedrock slopes at more than 3H:1V. At these locations, the bedrock should be excavated and extra bedding be placed to provide a 3H:1V (or flatter) transition from the bedrock subgrade towards the soil subgrade. This treatment reduces the propensity for bending stress to occur in the service pipes.

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

6.5 Groundwater Control

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. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

A temporary MOE permit to take water (PTTW) will be required for this project if more than 50,000 L/day are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MOE.

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.

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.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Restrictions

The proposed residential dwellings are located in a low to moderate sensitivity area with respect to tree planting in areas which are underlain by a silty clay deposit. It is recommended that trees placed within 4 m of the foundation wall should consist of low water demanding trees with shallow roots systems that extend less than 1.5 m below ground surface. Trees placed greater than 4 m from the foundation wall may consist of typical street trees, which are typically moderate water demand species with roots extending to a maximum depth of 2 m below ground surface.

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.

Swimming Pools

The in-situ soils are considered to be acceptable for in-ground swimming pools. Above ground swimming pools must be placed at least 3 m away from the residence foundation and neighbouring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer`s requirements.

Hot Tub and Exterior Structure

The in-situ soils are considered acceptable for hot tub and exterior structures, such as a deck addition, provided the finished grading respects our permissible grade raise restrictions. Otherwise, hot tub installation and exterior structure construction are considered routine, and can be constructed with the manufacturer’s requirements.

7.0 RECOMMENDATIONS

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

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

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

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to placing backfilling materials.

☐ Field density tests to ensure that the specified level of compaction has been achieved.

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

☐ Suggest foundation alternatives based on the potential long term settlements.

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 material testing and observation program by the geotechnical consultant.

8.0 STATEMENT OF LIMITATIONS

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

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Richcraft Homes 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.

Richard Groniger, C. Tech.

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Report Distribution:

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APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata Highlands - Phase 1 - Terry Fox Drive
Ottawa, Ontario

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

FILE NO. PG2971

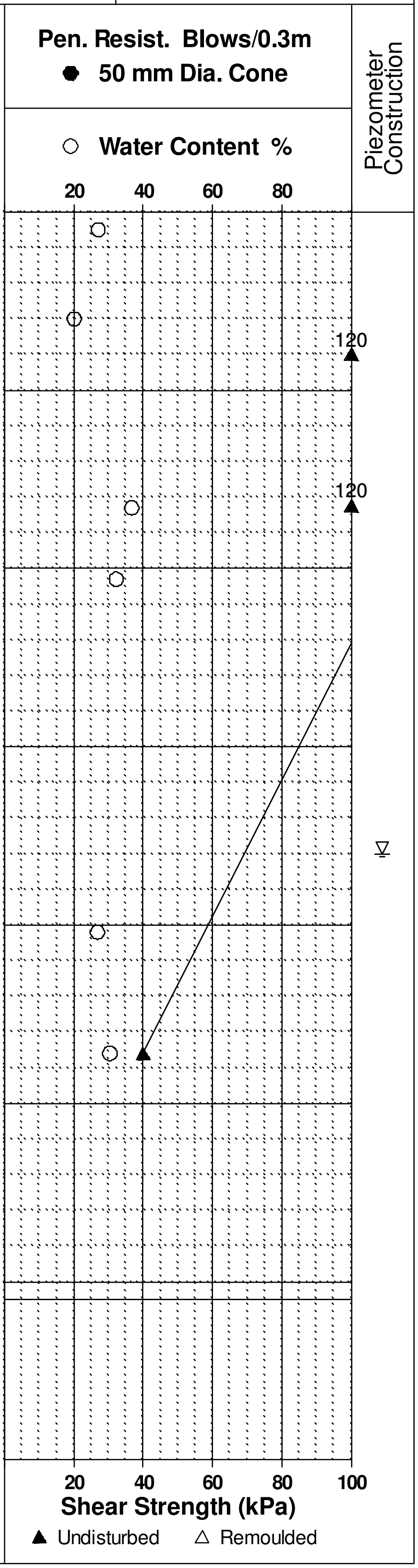
REMARKS

HOLE NO. TP 1

BORINGS BY Hydraulic Shovel

DATE May 23, 2013

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.81
TOPSOIL 0.20		G	1				
		G	2				
						1	104.81
		G	3				
						2	103.81
		G	4				
Very stiff to stiff, brown **SILTY CLAY**							
						3	102.81
						4	101.81
		G	5				
- firm to stiff and grey by 4.2m depth							
		G					
						5	100.81
						6	99.81
6.10							
End of Test Pit (GWL @ 3.6m depth based on field observations)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata Highlands - Phase 1 - Terry Fox Drive
Ottawa, Ontario

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

FILE NO. **PG2971**

REMARKS

HOLE NO. **TP 2**

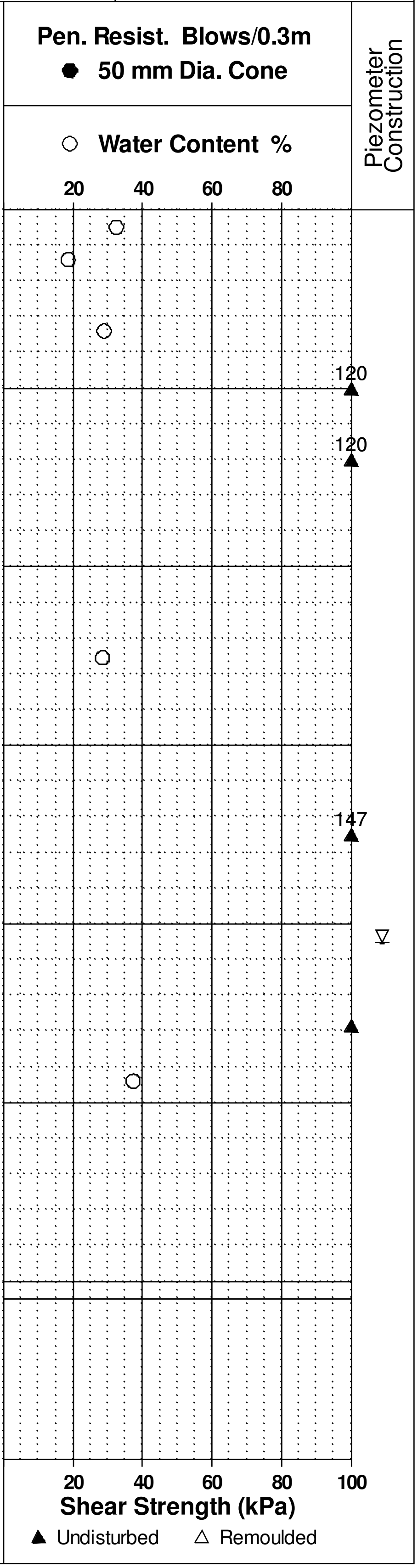
BORINGS BY Hydraulic Shovel

DATE May 23, 2013

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.20
Brown **SILTY SAND,** trace clay, gravel and cobbles	0.56
Very stiff, brown **SILTY CLAY** - stiff and grey by 4.6m depth	6.10
End of Test Pit (GWL @ 4.1m depth based on field observations)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
G	1		
G	2		
G	3		
G	4		

DEPTH (m)	ELEV. (m)
0	107.17
1	106.17
2	105.17
3	104.17
4	103.17
5	102.17
6	101.17



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata Highlands - Phase 1 - Terry Fox Drive
Ottawa, Ontario

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

REMARKS

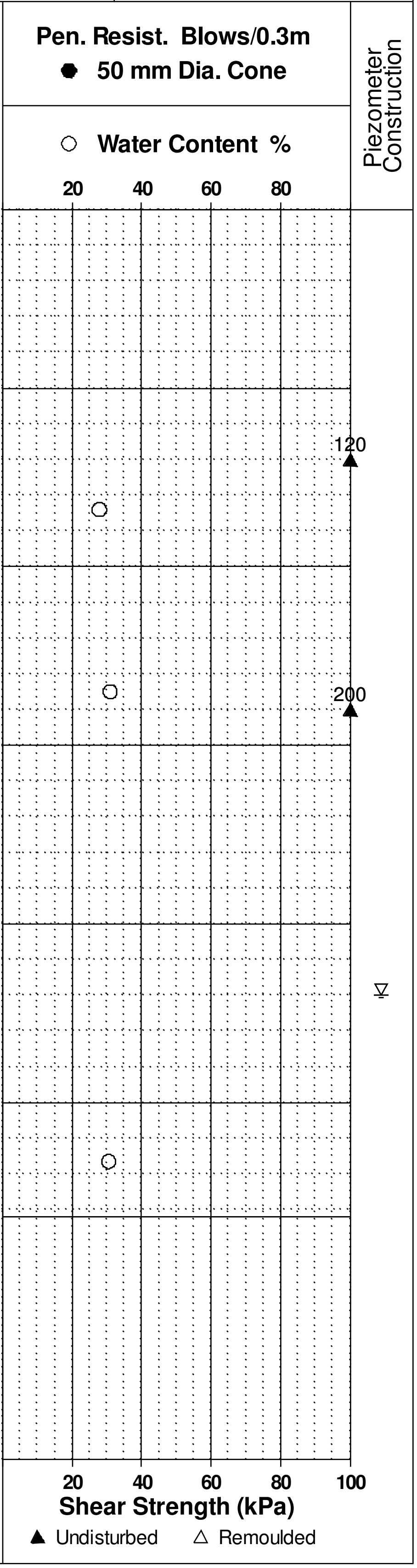
BORINGS BY Hydraulic Shovel

DATE May 23, 2013

FILE NO. PG2971

HOLE NO. TP 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	108.05
TOPSOIL 0.20						
					1	107.05
	G	1				
					2	106.05
Very stiff to stiff, brown **SILTY CLAY**						
					3	105.05
					4	104.05
	G	2				
- stiff and grey by 4.9m depth					5	103.05
	G	3				
5.64						
End of Test Pit (GWL @ 4.4m depth based on field observations)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata Highlands - Phase 1 - Terry Fox Drive
Ottawa, Ontario

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

FILE NO. **PG2971**

REMARKS

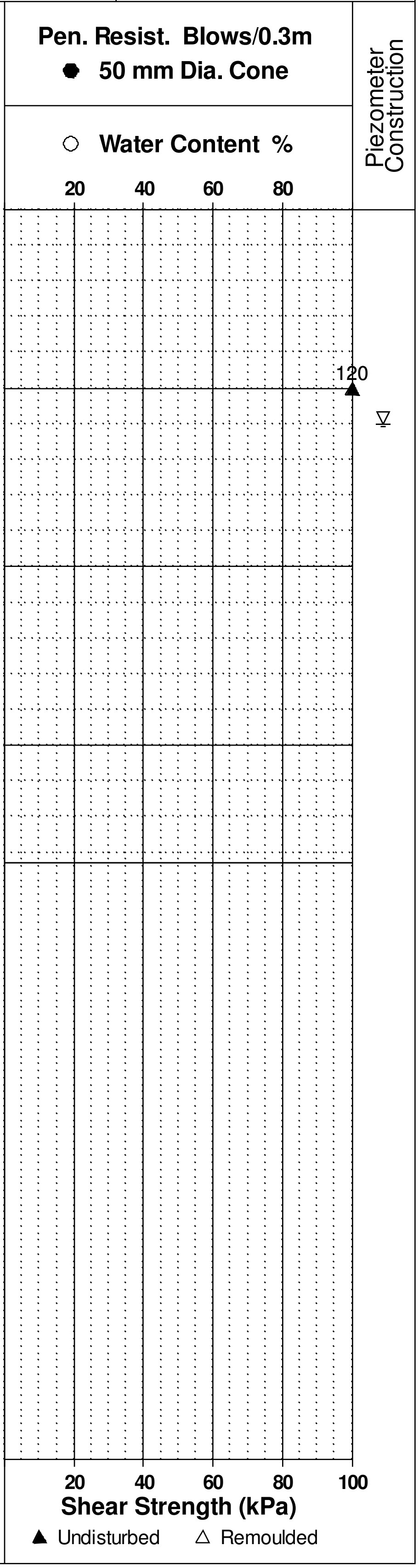
HOLE NO. **TP 4**

BORINGS BY Hydraulic Shovel

DATE May 23, 2013

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.30					
Very stiff to stiff, brown **SILTY CLAY** 2.13					
GLACIAL TILL: Dense, brown silty clay with sand, gravel, cobbles and boulders 3.66					
End of Test Pit (GWL @ 1.2m depth based on field observations)					

DEPTH (m)	ELEV. (m)
0	108.21
1	107.21
2	106.21
3	105.21



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata Highlands - Phase 1 - Terry Fox Drive
Ottawa, Ontario

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

FILE NO. PG2971

REMARKS

HOLE NO. TP 5

BORINGS BY Hydraulic Shovel

DATE May 23, 2013

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
TOPSOIL (to 0.30)	■					0	108.42	
Brown **SILTY CLAY** with sand (0.30 to 0.74)	▨							▽
GLACIAL TILL: Dense, brown silty clay with sand, gravel, cobbles and boulders (0.74 to 2.80)	^ ^ ^					1	107.42	
						2	106.42	
End of Test Pit Practical refusal to excavation at 2.80m depth (GWL @ 0.6m depth based on field observations)								

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

○ Water Content %
20 40 60 80

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

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SOIL PROFILE AND TEST DATA

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Ottawa, Ontario

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

FILE NO. **PG2971**

REMARKS

HOLE NO. **TP 6**

BORINGS BY Hydraulic Shovel

DATE May 23, 2013

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
GROUND SURFACE						0	108.34		
TOPSOIL 0.20									
Very stiff, brown **SILTY CLAY** 0.76									▽
End of Test Pit Practical refusal to excavation at 0.76m depth (GWL @ 0.6m depth based on field observations)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata Highlands - Phase 1 - Terry Fox Drive
Ottawa, Ontario

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

FILE NO. PG2971

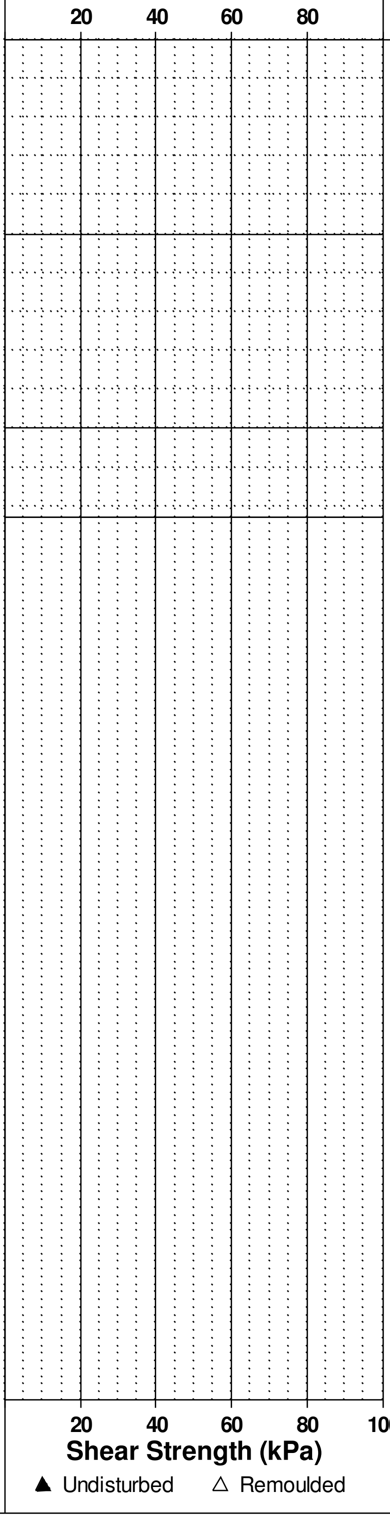
REMARKS

HOLE NO. TP 7

BORINGS BY Hydraulic Shovel

DATE May 23, 2013

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL	■					0	108.22
0.25							
Stiff, brown **SILTY CLAY** with sand	▨						
0.97						1	107.22
GLACIAL TILL: Dense, brown silty clay with sand, gravel, cobbles and boulders	^^^					2	106.22
2.46							
End of Test Pit Practical refusal to excavation at 2.46m depth (TP dry upon completion)							



Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata Highlands - Phase 1 - Terry Fox Drive
Ottawa, Ontario

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REMARKS

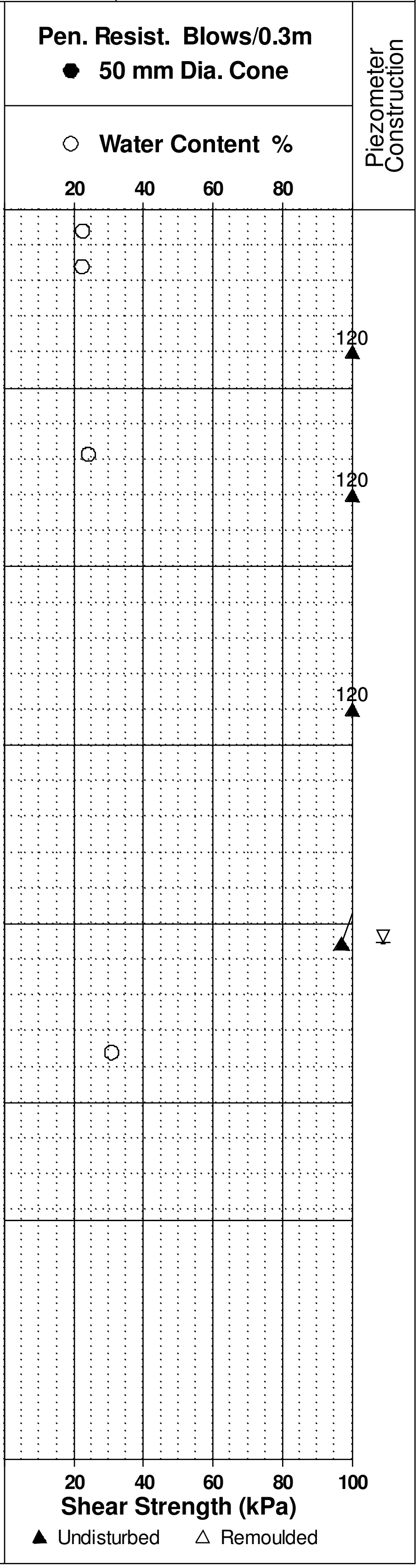
BORINGS BY Hydraulic Shovel

DATE May 24, 2013

FILE NO. PG2971

HOLE NO. TP 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	107.00
TOPSOIL 0.25		G	1				
		G	2				
						1	106.00
		G	3				
						2	105.00
Very stiff to stiff, brown **SILTY CLAY**							
						3	104.00
						4	103.00
- stiff and grey by 4.6m depth		G	4				
						5	102.00
5.66							
End of Test Pit (GWL @ 4.1m depth based on field observations)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata Highlands - Phase 1 - Terry Fox Drive
Ottawa, Ontario

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

FILE NO. PG2971

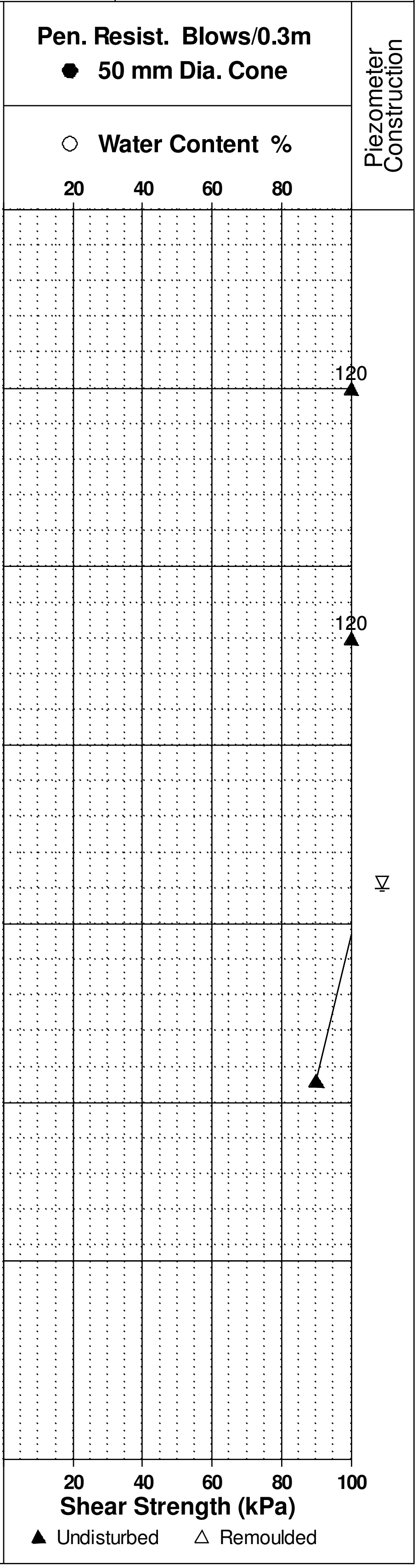
REMARKS

HOLE NO. TP 9

BORINGS BY Hydraulic Shovel

DATE May 24, 2013

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.16
TOPSOIL 0.23							
						1	104.16
						2	103.16
Very stiff to stiff, brown **SILTY CLAY**							
						3	102.16
						4	101.16
- stiff and grey by 4.3m depth							
						5	100.16
5.89							
End of Test Pit (GWL @ 3.8m depth based on field observations)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development, Huntmar Road
Ottawa, Ontario

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

REMARKS

BORINGS BY CME 75 Power Auger

DATE May 25, 06

FILE NO. PG0816

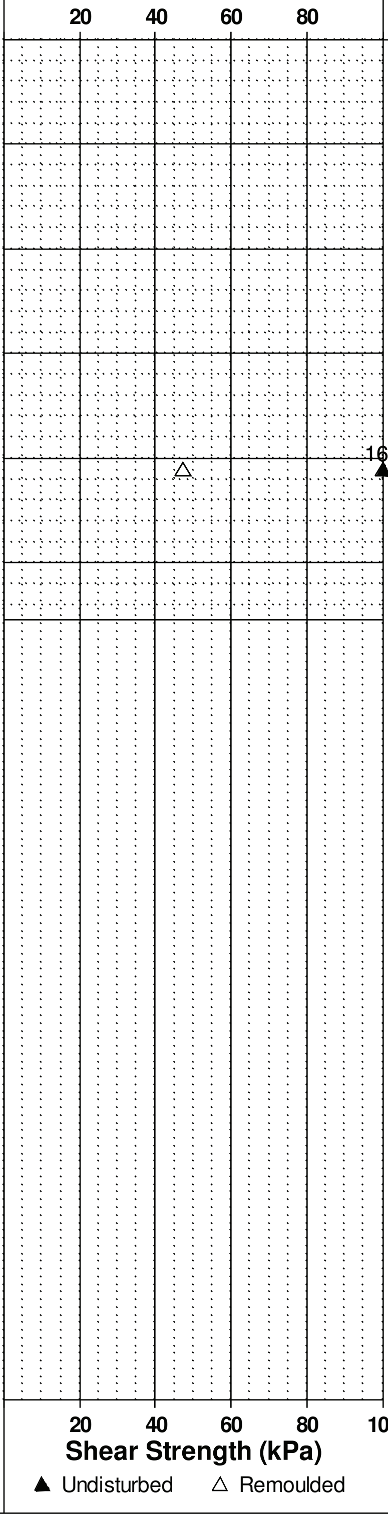
HOLE NO. BH12

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.40
TOPSOIL 0.18							
		SS	1	83	15	1	99.40
		SS	2	92	12		
						2	98.40
Very stiff, brown **SILTY CLAY**, occasional sand seams or gravel		SS	3	100	11		
						3	97.40
		SS	4	100	8		
						4	96.40
		SS	5	100	5	5	95.40
5.54							
End of Borehole Practical refusal to augering @ 5.54m depth (GWL @ 0.67-Jul 24/06)							

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

○ **Water Content %**

Piezometer Construction



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development, Huntmar Road
Ottawa, Ontario

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

FILE NO. **PG0816**

REMARKS

HOLE NO. **BH13**

BORINGS BY CME 75 Power Auger

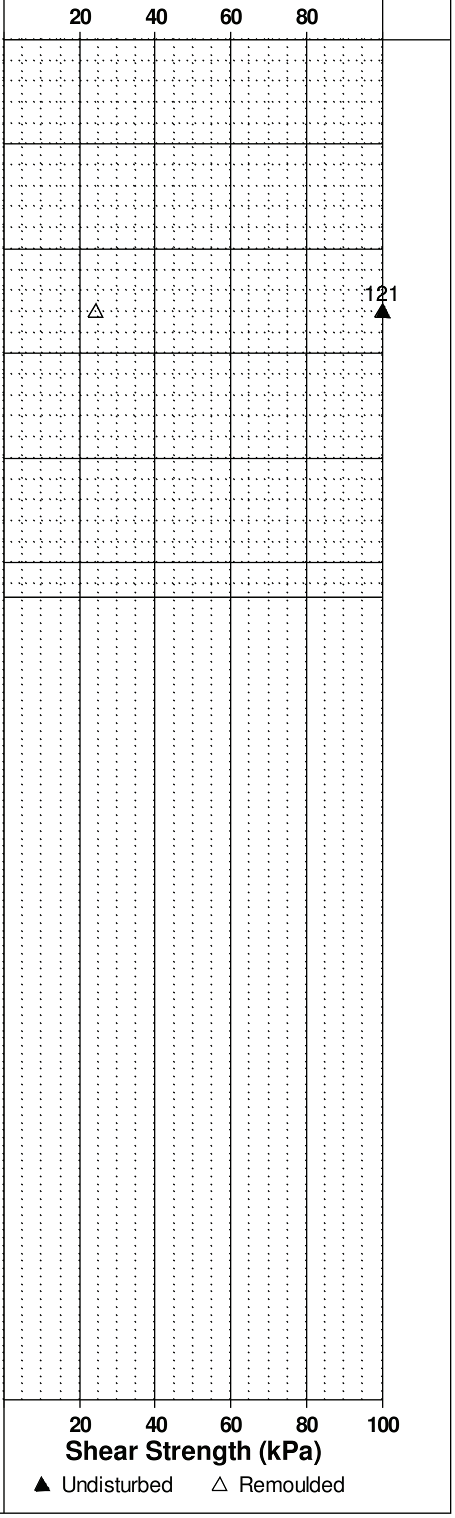
DATE May 25, 06

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	104.42
TOPSOIL 0.25							
		SS	1	100	13	1	103.42
		SS	2	100	5		
Very stiff, brown **SILTY CLAY**						2	102.42
						3	101.42
		SS	3	100	4		
- grey by 3.7m depth						4	100.42
		SS	4	100	4		
4.88							
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders 5.33		TW	5	75		5	99.42
End of Borehole Practical refusal to augering @ 5.33m depth							

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

○ Water Content %

Piezometer Construction



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

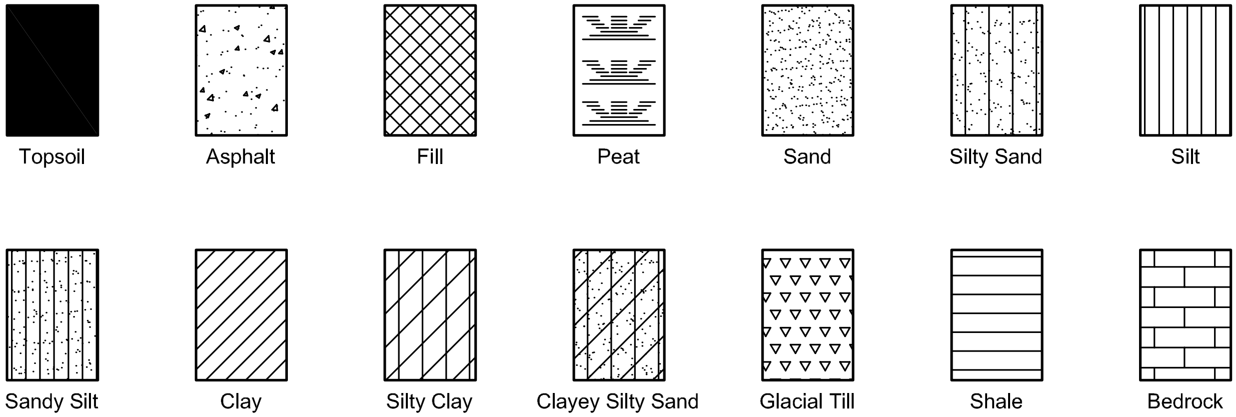
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

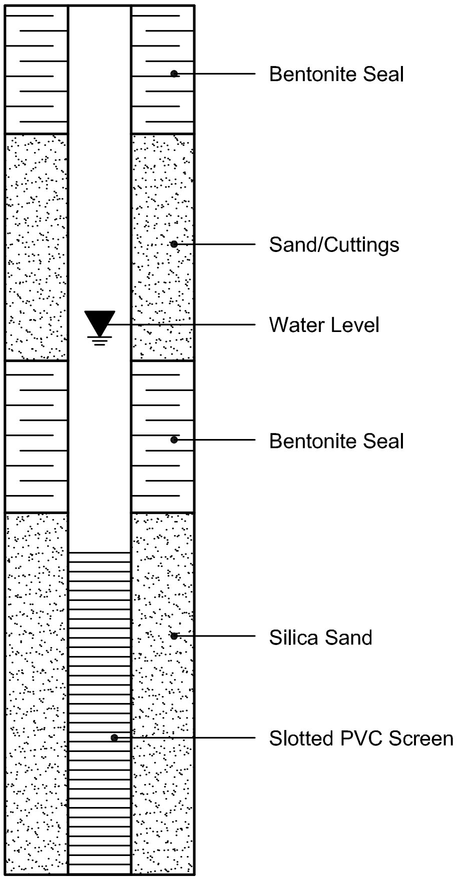
SYMBOLS AND TERMS (continued)

STRATA PLOT

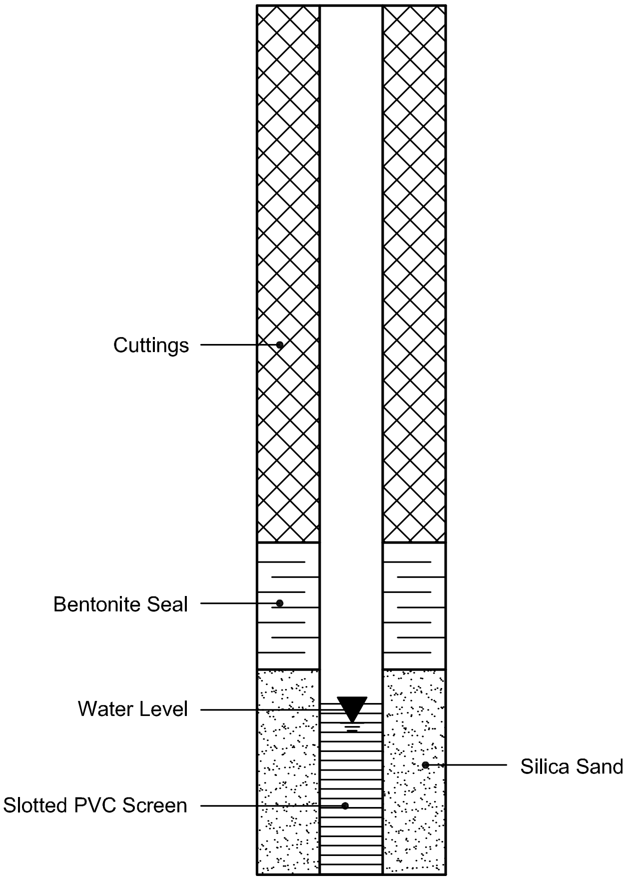


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 03-Jun-2013
Order Date:28-May-2013

Client: **Paterson Group Consulting Engineers**
Client PO: 13971
Project Description: PG2971

	Client ID:	TP#2-G4	-	-	-
	Sample Date:	24-May-13	-	-	-
	Sample ID:	1322083-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	69.1	-	-	-
General Inorganics					
pH	0.05 pH Units	7.42	-	-	-
Resistivity	0.10 Ohm.m	63.9	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	12	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG2971-1 - TEST HOLE LOCATION PLAN

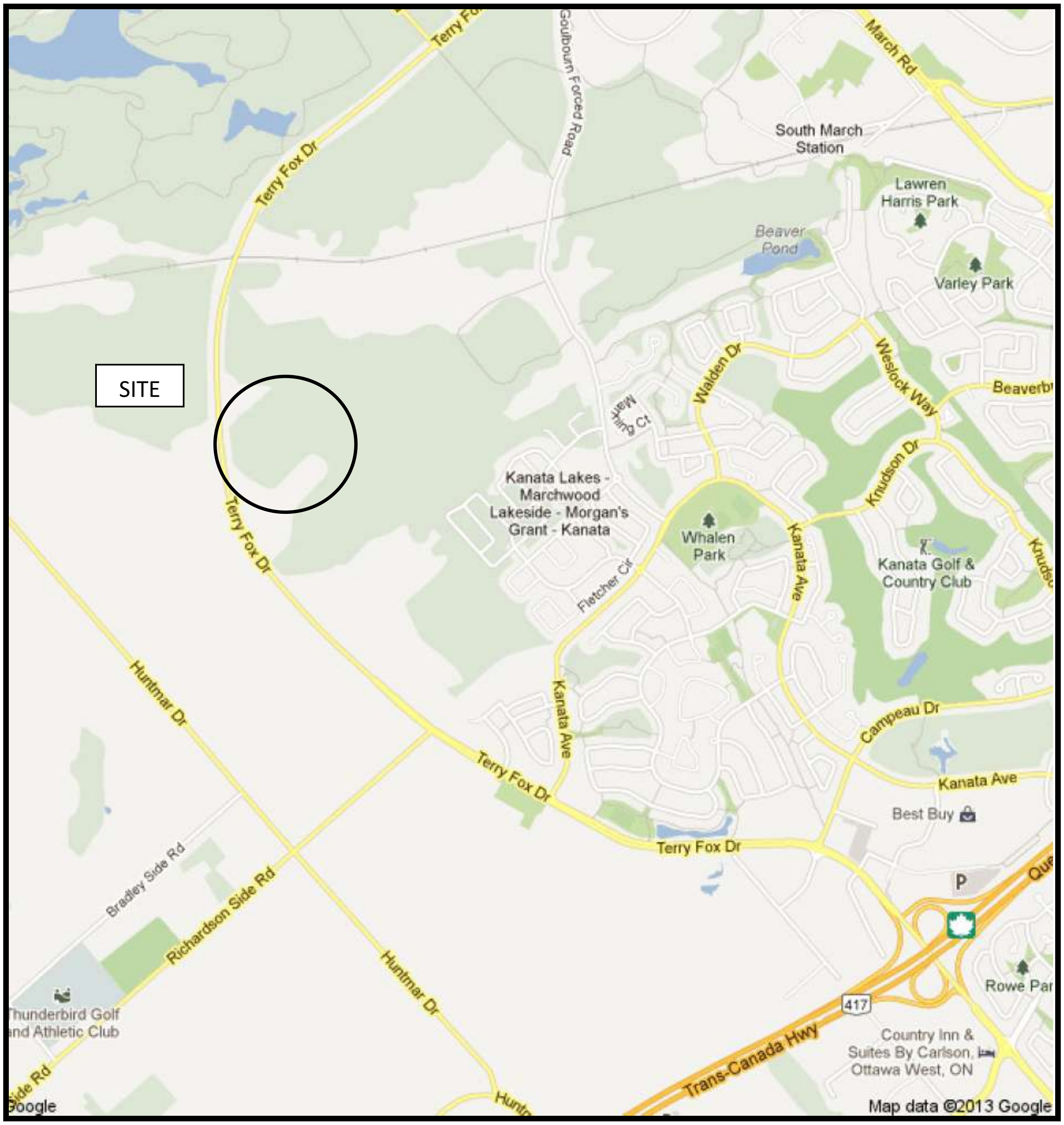


FIGURE 1

KEY PLAN

LEGEND:

▣	TEST PIT LOCATION
●	BOREHOLE LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP PG0816
108.22	GROUND SURFACE ELEVATION (m)
(105.76)	PRACTICAL REFUSAL TO EXCAVATION ELEVATION (m)
x 106.66	BEDROCK OUTCROP ELEVATIONS

BASE PLAN, TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS AT TEST HOLE LOCATIONS PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

SCALE - 1:2500

0 50 100 150m

paterson group
consulting engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

Scale:	1:2500
Des.:	RG
Dwn:	MPG
Chkd:	DG

RICHCRAFT GROUP OF COMPANIES

GEOTECHNICAL INVESTIGATION
KANATA HIGHLANDS - PHASE 1 - TERRY FOX DRIVE
OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No.	PG2971-1
Report No.:	PG2971-1
Date:	06/2013