patersongroup

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July 25, 2016
File: PG3836-LET.01

CLV Group
485 Bank Street, Suite 200
Ottawa, Ontario
K2P 1Z2

Attention: **Mr. Mike Kelly**

Subject: **Preliminary Geotechnical Investigation**
Proposed Residential Development
530 Tremblay Road - Ottawa

Dear Sir,

Paterson Group (Paterson) was commissioned by CLV Group to conduct a preliminary geotechnical investigation for a proposed residential development to be located at the aforementioned site. The following report presents our findings and recommendations from a geotechnical perspective.

1.0 Proposed Development

It is our understanding that consideration is being taken to construct 3 multi-storey residential buildings with up to one level of underground parking. It is also anticipated that asphaltic covered car parking, access lanes and landscaping areas are also planned for the proposed development.

2.0 Field Observations

Surface Conditions

Generally, the subject site is currently vacant and mostly overgrown with tall brush and small trees and observed to be approximately at grade with neighbouring properties and adjacent roadways. The site is bordered to the north and west by single family residential dwellings, to the south by CN-Rail and to the east by vacant commercial land.

Field Program

The field program for the current preliminary geotechnical investigation was carried out on July 8, 2016. At that time, a total of 7 test pits (TP 1 to TP 7) were excavated using a hydraulic excavator operated by a local contractor. The test pit locations were distributed in a manner to provide general coverage of the subject site while taking into consideration underground utilities and site features. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The testing procedure consisted of excavating to a maximum depth of 1.9 m depth and regularly sampling the overburden. The approximate test hole locations are shown on Drawing PG3836-1 - Test Hole Location Plan attached to this report.

Subsurface Conditions

Generally, the subsurface profile encountered at the test hole locations consist of a thin layer of topsoil and/or fill overlying a compact glacial till and/or black shale bedrock. The overlying fill generally consist of a relatively compact to dense silty sand to sandy silt with some clay, gravel, cobbles, boulders trace topsoil and organics extending to a maximum depth of 1.6 m at the test hole locations. The glacial till encountered at TP 6 and TP 7 consisted of a compact silty fine sand with gravel, cobbles and boulders, trace shale extending to a depth of 1.9 and 1.8 m, respectively.

A heavily fractured, weathered black shale bedrock overlying a moderately sound black shale bedrock was encountered at all test hole locations with the exception of TP 6 and TP 7 where a moderately sound bedrock was found immediately below the compact glacial till. The black shale bedrock was encountered at depths varying between 0.1 to 1.9 m across the site.

Based on available geological mapping, the subject site is in close proximity to the transition zone between the Carlsbad Formation and the Billings Formation where the bedrock consists of shale. Upon being exposed to air and then rewetted, the Billings Formation shale readily decomposes into thin flakes along the bedding planes. Previous studies have concluded that local pyrite bearing shales are subject to volume changes upon exposure to air as a result of the formation of jarosite crystals by aerobic bacteria under certain ambient conditions.

It has been determined that the expansion process does not occur or is retarded when air (i.e. oxygen) is prevented from coming into contact with the shale and/or the ambient temperature is kept below 20 degrees Celsius, and/or the shale is confined by pressures in excess of 70 kPa. The latter restriction on the heaving process is probably the major reason why damage to structures has, for the greater part, been confined to slabs-on-grade rather than footings.

Groundwater

Groundwater infiltration levels were measured within the open test holes upon completion of the sampling program on July 8, 2016. All test hole were observed to be dry upon completion and it is expected that the groundwater level will be encountered within the bedrock surface at depths ranging between 2 to 2.5 m below existing ground surface. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Subsurface conditions observed at the test hole locations were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets attached to this report for specific details of the soil profile encountered at the test hole locations.

3.0 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered adequate for the proposed residential development. It is expected that the proposed buildings will be constructed with conventional shallow foundations bearing on the moderately sound black shale bedrock surface.

Site Preparation and Fill Placement

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb the bearing surface below the subgrade level during site preparation activities.

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.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing.

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipments could be the source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of these equipments. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the proposed buildings footprint, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It 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 proposed buildings should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

It is anticipated that over-blasting of the bedrock may occur for the proposed building pads. It is recommended that a review of the blasted bedrock surface be reviewed by the geotechnical consultant at the time of construction to confirm if the blast rock is acceptable to remain in place below footings.

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 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.

Shallow Foundation

Footings placed on a clean, surface sounded shale bedrock surface, a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5, and a bearing resistance value at serviceability limit states (SLS) of **1,000 kPa**.

The settlement associated with the bearing resistance value at SLS is expected to be negligible.

A clean, surface-sounded shale 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.

Lateral Support

Adequate lateral support is provided to a moderately sound shale bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through a moderately sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered shale bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Design for Earthquakes

Based on our findings and previous studies in the immediate area, a seismic **Site Class A** can be used for design of the proposed buildings founded on the shale bedrock. However, a site specific shear wave velocity test is required to be completed at the subject site to confirm the recommended seismic site classification as per OBC 2012.

Basement Slab

With the removal of all topsoil and fill, containing deleterious material, within the footprint of the proposed building, the native soil will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with an appropriate backfill material prior to placing any fill. OPSS Granular B Type II is recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of a 19 mm clear crushed stone material. All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

Pavement Structure

Car only parking and heavy truck parking areas, as well as, access lanes are anticipated. The proposed pavement structures are presented in Tables 1 and 2.

Table 1 - Recommended Pavement Structure - Car Only Parking Areas

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

**Table 2 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas**

Thickness mm	Material Description
40	**WEAR COURSE** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**BINDER COURSE** - HL-8 or Superpave 19 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soils 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 98% of the material's SPMDD using suitable compaction 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 impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should extend in four orthogonal directions or longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines or the pipe, should be wrapped with suitable filter cloth. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

4.0 Design and Construction Precautions

Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed buildings. The system should consist of a 150 mm diameter, geotextile-wrapped, 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 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 pipe is provided. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose.

Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

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

It is expected that the parking garage will not require protection against frost action due to the founding depth. Unheated structures such as the access ramp may 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.

Frost Susceptibility of Bedrock

When bedrock is encountered above the proposed founding depth and soil frost cover is less than 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 - Sub-excavate 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.

Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be 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 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.

Protection of Potential Expansive Bedrock

It is possible that expansive shale will be encountered at the subject site. A potential for heaving and rapid deterioration of the shale bedrock exists at this site. To reduce the long term deterioration of the shale, exposure of the bedrock surface to oxygen should be kept as low as possible. The bedrock surface within the proposed building footprints should be protected from excessive dewatering and exposure to ambient air. To accomplish this a 50 mm thick concrete mud slab should be placed on the exposed bedrock surface within a 48 hour period of being exposed. A 15 MPa lean concrete may be used.

The excavated sides of the exposed bedrock should be sprayed with a bituminous emulsion to seal bedrock from exposure to air and dewatering.

Another option for protecting the shale from deterioration is placing granular fill over the exposed surface within a 48 hour period after exposure. Preventing the dewatering of the shale bedrock will also prevent the rapid deterioration and expansion of the shale bedrock. This can be accomplished by spraying bituminous emulsion as noted above.

Groundwater Control

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

A temporary MOE permit to take water (PTTW) may be required for this project if more than 50,000 L/day is 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.

Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil 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.

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

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice 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. Also, 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.

5.0 Recommendations

It is a requirement for the design data provided herein to be applicable that an acceptable filed review and materials testing program, including the aspects shown below, be performed by the geotechnical consultant.

☐ Complete a detailed geotechnical investigation to provide additional borehole coverage as per City of Ottawa Geotechnical Reporting Guidelines.

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

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

☐ Observation of all subgrades prior to backfilling.

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

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

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

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

6.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. Our recommendations should be reviewed when the project drawings and specifications are complete.

A preliminary geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our 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 CLV Group 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.

Carlos P. Da Silva, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Aug.25/10
C.P. DA SILVA
10304509
PROVINCE OF ONTARIO

Attachments

- ☐ Soil Profile and Test Data Sheets
- ☐ Symbols and Terms
- ☐ Figure 1 - Key Plan
- ☐ Drawing PG3836-1 - Test Hole Location Plan

Report Distribution

- ☐ CLV Group (3 hard copies and an electronic copy)
- ☐ Paterson Group (1 copy)

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

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Residential Development - 530 Tremblay Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the south end of Avenue U. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Hydraulic Shovel

DATE July 8, 2016

FILE NO. **PG3836**

HOLE NO. **TP 1**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	98.93		
TOPSOIL with boulders 0.30	■	G	1						
FILL: Brown sandy silt, trace organics 0.40	▩	G	2						
FILL: Very stiff, brown silty clay with sand and topsoil 0.60	▩	G	3						
BEDROCK: Weathered black shale 1.20	▤	G	4			1	97.93		
End of Test Pit Practical refusal to excavation on black shale bedrock at 1.20m depth (TP dry upon completion)									

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

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DATUM TBM - Top spindle of fire hydrant located at the south end of Avenue U. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

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DATE July 8, 2016

FILE NO. **PG3836**

HOLE NO. **TP 2**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	98.69		
FILL: Brown sandy silt to silty clay with topsoil, trace organics 0.12	▩	G	1						
BEDROCK: Weathered black shale 1.10	▤	G	2			1	97.69		

End of Test Pit

Practical refusal to excavation on black shale bedrock at 1.10m depth

(TP dry upon completion)

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

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Residential Development - 530 Tremblay Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the south end of Avenue U. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Hydraulic Shovel

DATE July 8, 2016

FILE NO. PG3836

HOLE NO. TP 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	98.37		
FILL: Brown silty clay to clayey silt with sand, topsoil and organics 0.20	▨	G	1						
BEDROCK: Weathered black shale 1.10	▤	G	2			1	97.37		
End of Test Pit Practical refusal to excavation on black shale bedrock at 1.10m depth (TP dry upon completion)									

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

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Residential Development - 530 Tremblay Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the south end of Avenue U. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Hydraulic Shovel

DATE July 8, 2016

FILE NO. PG3836

HOLE NO. TP 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	98.51		
TOPSOIL 0.20	▨	G	1						
FILL: Brown sandy silt, trace organics 0.40	▨	G	2						
BEDROCK: Weathered black shale 1.00	▤	G	3						
						1	97.51		

End of Test Pit

Practical refusal to excavation on black shale bedrock at 1.00m depth

(TP dry upon completion)

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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Prop. Residential Development - 530 Tremblay Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the south end of Avenue U. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

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DATE July 8, 2016

FILE NO. PG3836

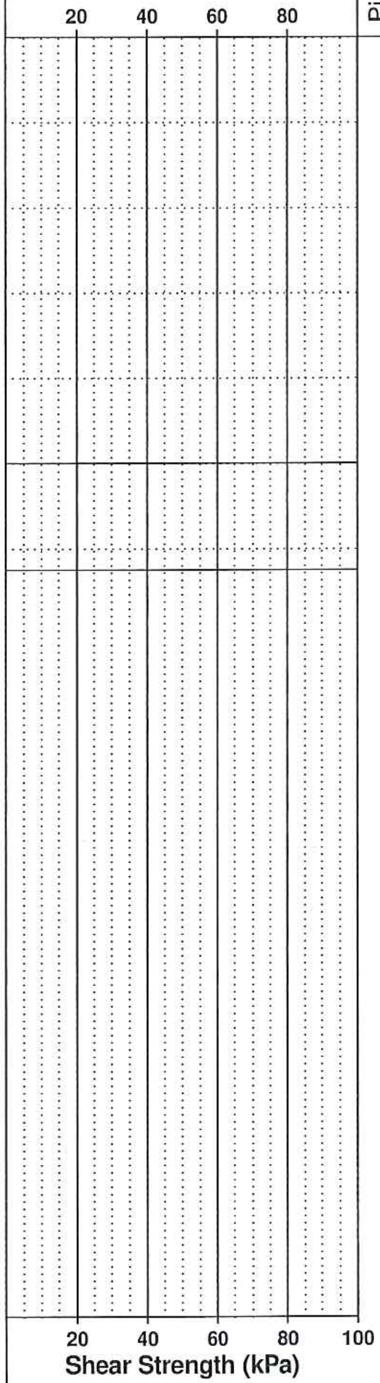
HOLE NO. TP 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.05
TOPSOIL 0.30	■	G	1				
FILL: Brown silty fine to medium sand with gravel, cobbles, trace boulders and organics 0.65	▨	G	2				
FILL: Brown to grey silty clay with sand, gravel, trace topsoil and organics 1.00	▨	G	3			1	98.05
BEDROCK: Weathered black shale 1.25	▤	G	4				
End of Test Pit Practical refusal to excavation on black shale bedrock at 1.25m depth (TP dry upon completion)							

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Prop. Residential Development - 530 Tremblay Road
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DATUM TBM - Top spindle of fire hydrant located at the south end of Avenue U. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

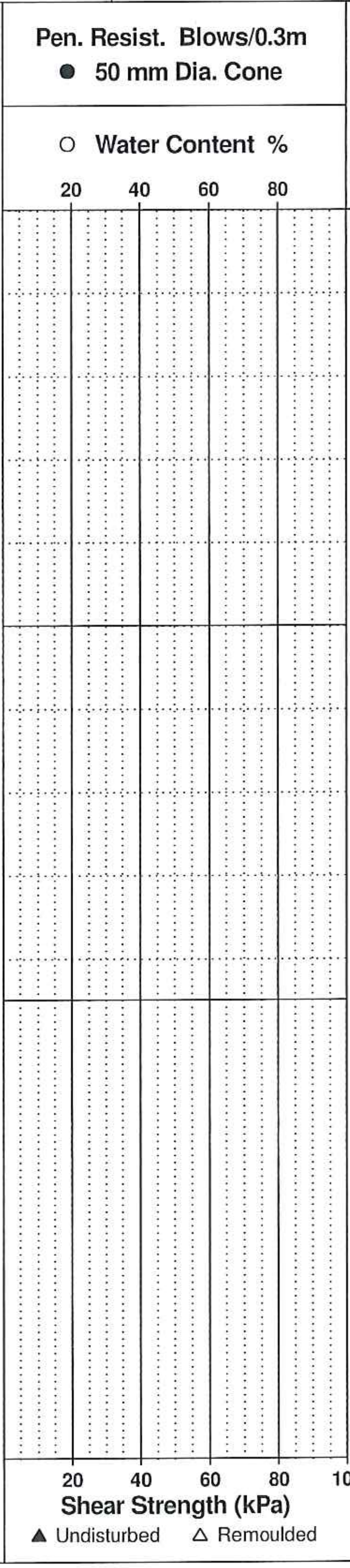
BORINGS BY Hydraulic Shovel

DATE July 8, 2016

FILE NO. PG3836

HOLE NO. TP 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	99.05
TOPSOIL 0.20	■	G	1				
FILL: Brown sandy silt to silty fine sand, some gravel, trace organics 0.50	▨	G	2				
FILL: Brown silty fine to medium sand with clay, gravel, trace topsoil and shale	▨	G	3				
1.30	▨	G	4			1	98.05
FILL: Brown silty clay to silty sand with gravel, cobbles and shale 1.60	▨	G	5				
GLACIAL TILL: Compact, brown silty fine sand with clay, gravel, cobbles, trace boulders and shale 1.90	^^^	G	6				
End of Test Pit Practical refusal to excavation on black shale bedrock at 1.90m depth (TP dry upon completion)							



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SOIL PROFILE AND TEST DATA

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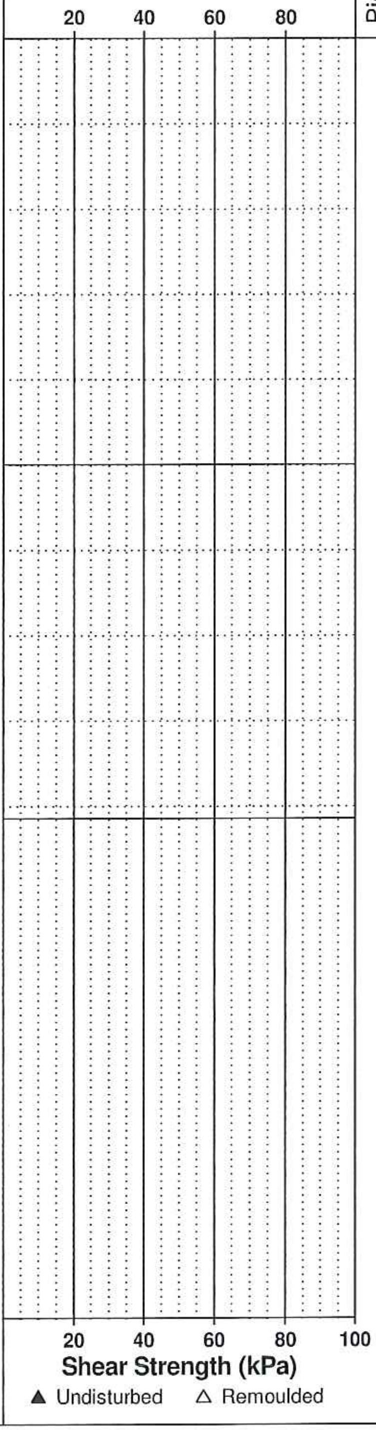
BORINGS BY Hydraulic Shovel

DATE July 8, 2016

FILE NO. **PG3836**

HOLE NO. **TP 7**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	98.81		
TOPSOIL 0.20		G	1						
FILL: Brown silty clay with sandy silt, gravel, trace topsoil and organics - trace shale fragments by 0.6m depth		G	2						
		G	3						
1.20		G	4			1	97.81		
GLACIAL TILL: Compact silty fine to medium sand with clay, gravel, cobbles, trace boulders and shale		G	5						
1.83		G	6						
End of Test Pit Practical refusal to excavation on black shale bedrock at 1.83m depth (TP dry upon completion)									



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)

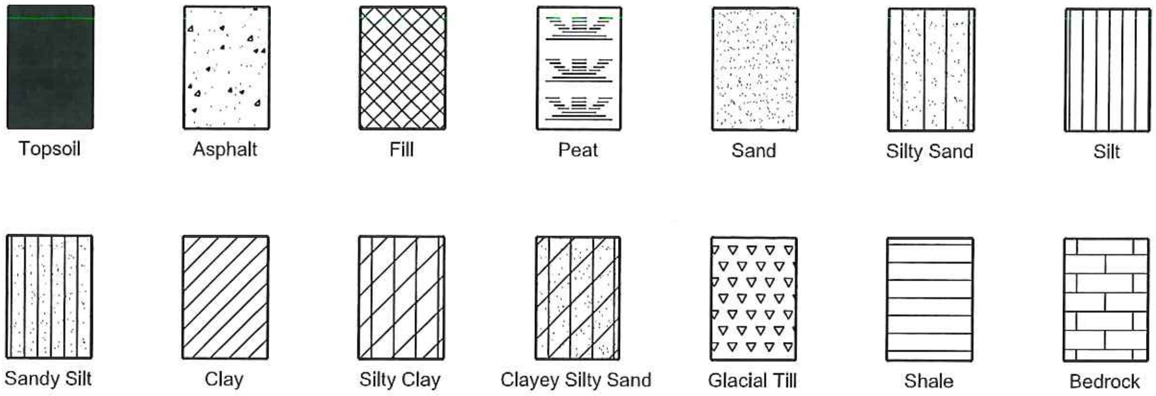
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

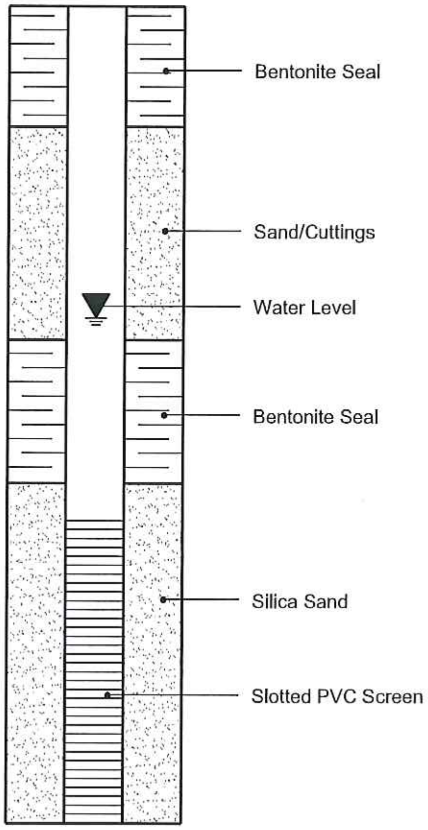
SYMBOLS AND TERMS (continued)

STRATA PLOT

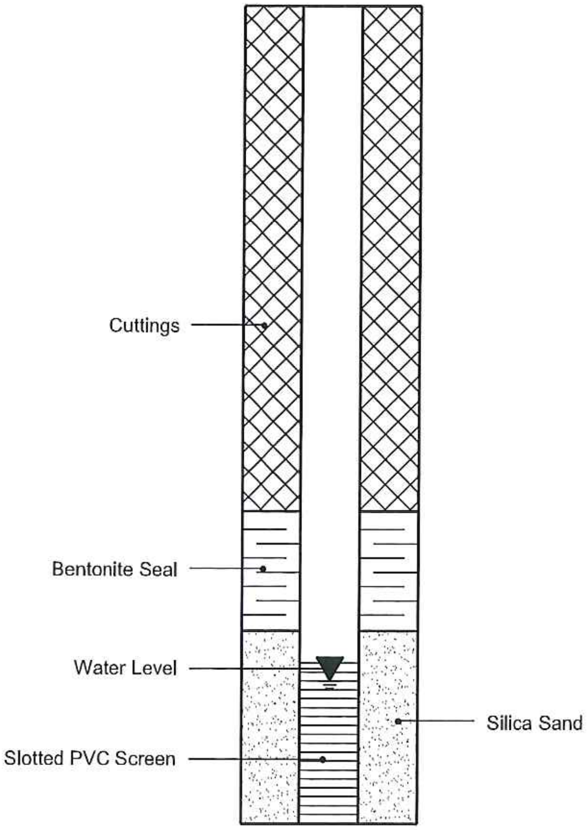


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



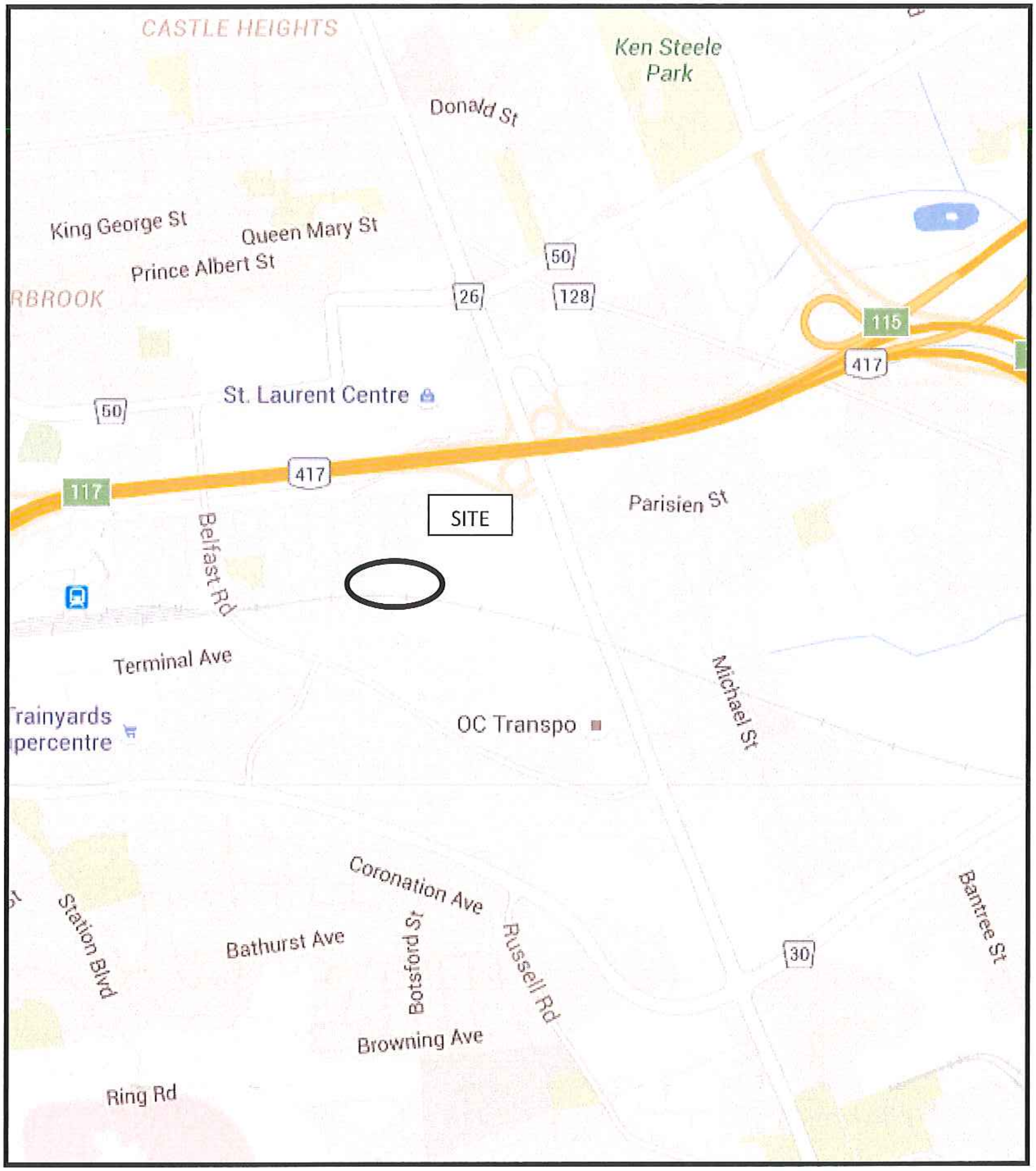
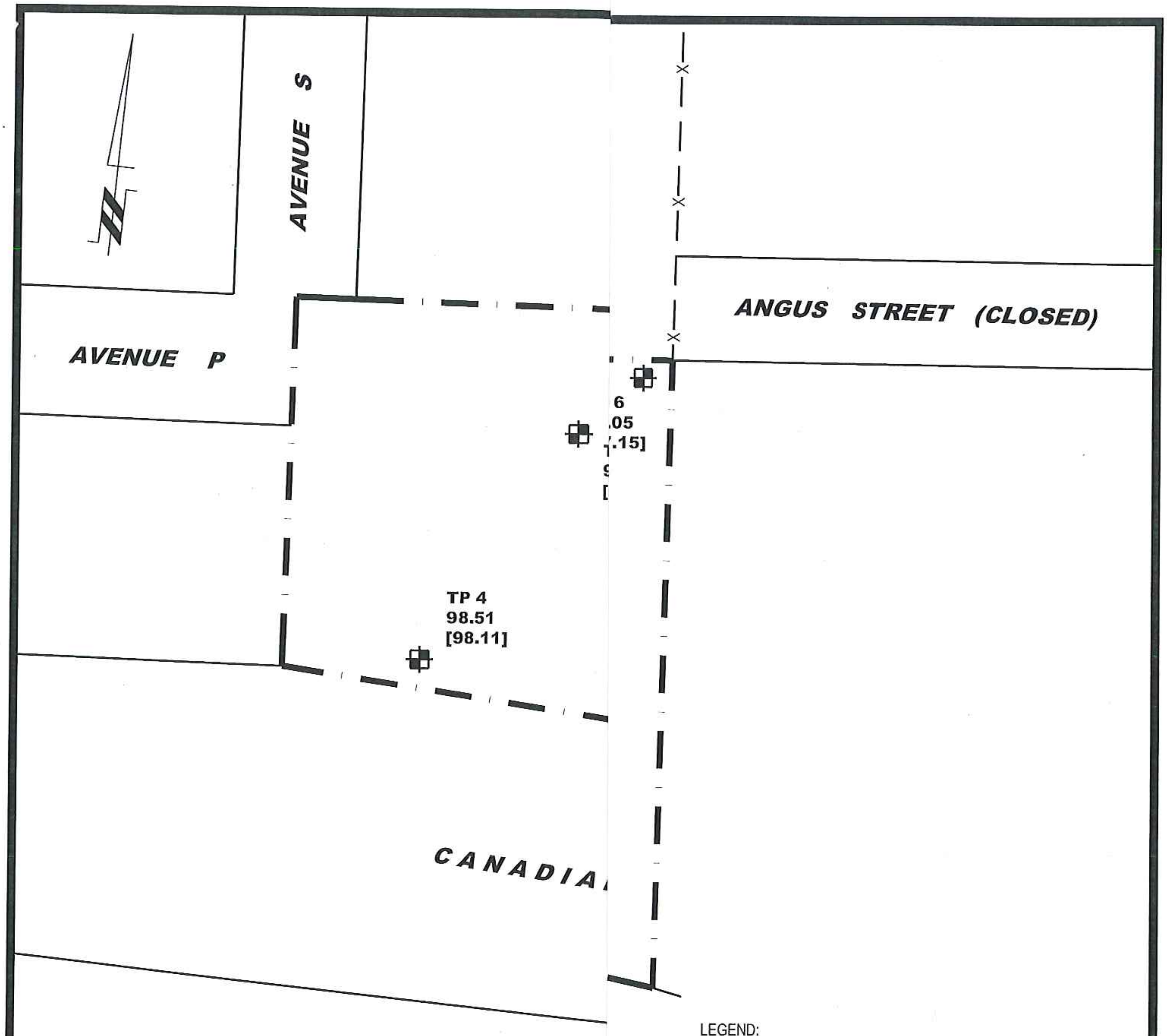


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

⊞	APPROXIMATE TEST PIT LOCATION
99.05	GROUND SURFACE ELEVATION (m)
[98.5]	BEDROCK SURFACE ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT LOCATED AT THE SOUTH END OF AVENUE U. AN ARBITRARY ELEVATION OF 100.00m WAS ASSIGNED TO THE TBM.

SCALE: 1:750

0 10 20 30 50m

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0	
NO.	

TARIO

Scale: 1:750	Date: 07/2016
Drawn by: MPG	Report No.: PG3836-LET.01
Checked by: RG	Dwg. No.: PG3836-1
Approved by: DJG	Revision No.: 0

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