Geotechnical Engineering

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Geotechnical Investigation

Proposed Gas Station & Convenience Store
6175 Rockdale Road
Vars, Ontario

Prepared For

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Report: PG4132-1 Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Mr. Abdu El-Arab to conduct a geotechnical investigation for the proposed gas station and convenience store to be constructed at 6175 Rockdale Road located in the Town of Vars (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was to:

- ❑ determine the subsurface soil and groundwater conditions by means of boreholes.
- ❑ 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. The report contains the geotechnical findings and includes recommendations pertaining to the design and construction of the subject development as 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.

2.0 Proposed Development

Based on the conceptual drawing provided, it is our understanding that the proposed development will consist of a one-storey building of slab-on-grade construction to occupy the north portion of the property with a drive thru lane north of the building. An overhead canopy and underground tanks are also proposed as part of the construction of the gas station pumping area.

It should be noted that the existing gas station building will be demolished as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on May 26, 2017. At that time, a total of four (4) boreholes were advanced to a maximum depth of 5.4 m below existing ground surface. The test hole locations were determined in the field by Paterson personnel and distributed in a manner to provided general coverage of the proposed development taking into considered site features and underground utilities. The test hole locations are presented on Drawing PG4132-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The test hole procedures consisted of augering or excavating to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights or a 50 mm diameter split-spoon sampler. All soil samples were classified on site, placed in sealed plastic bags and transported to the laboratory for further review. The depths at which the auger and split spoon samples were recovered from the test holes are presented as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Standard Penetration Testing (SPT) was conducted and 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 sample 300 mm into the soil after a 150 mm initial penetration with a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing was conducted at regular intervals in cohesive soils and completed using a shear vane apparatus.

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

Flexible standpipes were installed in the majority of the boreholes during the field investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations were determined and surveyed in the field by Paterson personnel with consideration of existing site features. The ground surface elevations at the test hole locations were referenced to a temporary benchmark (TBM), consisting of the top of the existing well on site. A geodetic elevation of 76.72 m was provided to the TBM on a plan by WSP Canada Inc..

The location of the TBM, test hole locations and ground surface elevation at each test hole locations are presented on Drawing PG4132-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The majority of the ground surface across the subject site was noted to be relatively flat and slightly higher than the neighbouring roadways. The ground surface was asphalt covered with an existing gas station building and pumping area occupying the centre of the site.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of asphaltic concrete overlying a crushed stone layer underlain by a dark grey sandy clay with silt and gravel fill. Compact silty sand over glacial till were encountered below the fill layer. A layer of silty clay was encountered below the fill layer at BH 1. Practical refusal to augering was encountered at depths ranging from 4.2 m to 5.4 m on shale bedrock.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in this area consists of shale with dolomitic layers from the Carlsbad formation. The overburden drift thickness within the subject site is expected between 3 to 6 m below ground surface.

4.3 Groundwater

The measured groundwater levels are summarized below in Table 1 and presented on the Soil Profile and Test Data sheets in Appendix 1. The long-term groundwater level can also be estimated based on the recovered soil samples' moisture levels, colouring and consistency. Based on these observations, the long-term groundwater level is anticipated at 3 to 4 m depth. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings				
Test Hole Number	**Ground Surface Elevation (m)**	**Groundwater Depth (m)**	**Groundwater Elevation (m)**	**Date**
BH 1	76.69	1.24	75.45	June 1, 2017
BH 2	76.82	2.04	74.78	June 1, 2017
BH 3	76.69	2.72	73.97	June 1, 2017
Note: The ground surface at the test hole locations was referenced to a temporary benchmark (TBM) consisting of the top of the existing well casing. A geodetic elevation of 76.72 m was assigned to the TBM.				

5.0 Discussion

5.1 Geotechnical Assessment

Based on the results of our investigation, it is expected that the proposed building will be founded over conventional shallow footings placed on an undisturbed, silty sand, glacial till or engineered fill bearing surface. From a geotechnical perspective, the subject site is satisfactory for the proposed development.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. It is anticipated that the existing fill, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprints, outside of lateral support zones for the footings, and below the proposed parking area and access lanes. It is recommended that the existing fill layer be proof-rolled by a vibratory roller and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

Existing foundation walls and other construction debris should be entirely removed from within the proposed building perimeter. Where pavement areas are proposed above the demolished building footprint, a 1 m separation is required between the existing foundations and the bottom of the pavement structures.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material’s standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts compacted by the tracks of the spreading equipment to minimize voids. If the material is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 95% of the material's SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Footings founded on an undisturbed, compact silty sand bearing surface can be designed a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS0 of **180 kPa**.

Footings placed over glacial till or an approved engineered fill extending to an undisturbed, glacial till bearing surface can be designed using the bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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

Existing Fill

In areas where the existing fill is present below the design underside of footing elevation, the following options are recommended:

- ❑ Option A - Extend foundation wall and footings to reach an undisturbed, compact silty sand or glacial till bearing surface.
- ❑ Option B - Remove existing fill and deleterious materials, such as those containing significant amounts of organics, to provide an undisturbed, compact silty sand or glacial till bearing surface. An engineered fill as detailed in Subsection 5.2 should be placed over the undisturbed, silty clay and extend to

design underside of footing elevation. The engineered fill should extend down and out from outside face of footing at a 1.5H:1V slope to provide adequate lateral support.

- ❏ Option C - A near vertical zero entry trench should be extended through the existing fill and deleterious materials, such as those containing significant amounts of organics, to an undisturbed, silty clay bearing surface. The trench width should be at least 300 mm beyond the footing edge and be in-filled with a minimum 15 MPa lean concrete to design underside of footing elevation.

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 a stiff silty clay or engineered fill above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

5.4 Design for Earthquakes

The subject site can be taken as seismic site response **Class C** as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A) for foundations considered at this site. The soils underlying the site are not susceptible to liquefaction.

5.5 Slab on Grade Construction

With the removal of all topsoil and deleterious fill, such as those containing significant amounts of organic matter, within the footprint of the proposed building, undisturbed native soil surface or existing fill approved by the geotechnical consultant at the time of construction will be considered acceptable subgrade on which to commence backfilling for floor slab construction. It is recommended that the existing fill layer, free of deleterious and organic materials, be proof-rolled by a vibratory roller making several passes and approved by the geotechnical consultant at the time of construction.

Any soft areas should be removed and backfilled with appropriate backfill material. It is recommended that the upper 200 mm of sub-slab fill consist of Granular A crushed stone.

5.6 Pavement Structure

For design purposes, the following pavement structures presented below could be used for the design of car parking areas and access lanes. It is anticipated that both pavement structures provided would be adequate for use as a fire route.

Table 2 - 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 soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 3 - Recommended Pavement Structure - Access Lanes and Heavy Loading Parking Area

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

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, 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 SPMDD using suitable vibratory equipment.

Where the proposed pavement structure meets the existing asphalt surface, the following recommendations should be followed:

- ❑ A 300 mm wide section of the existing asphalt roadway should be saw cut from the existing pavement edge to provide a sound surface to abut the proposed pavement structure.
- ❑ It is recommended to mill a 300 mm wide and 40 mm deep section of the existing asphalt at the saw cut edge.
- ❑ The proposed pavement structure subbase materials should be tapered no greater than 3H:1V to meet the existing subbase materials.
- ❑ Clean existing granular road subbase materials can be reused upon assessment by the geotechnical consultant at the time of excavation (construction) as to its suitability.

Rigid Pavement Structure

It is understood that a rigid pavement structure may be considered for the drive-thru and/or the pumping area. The rigid pavement structure presented in Table 4 is recommended for the subject drive-through lane for areas where a concrete pad is required. It should be noted that the reinforced concrete will be susceptible to frost heave if frost protection is not provided. Therefore, control and isolation joints are required for the subject concrete slabs.

Table 4 - Rigid Pavement Structure

Thickness (mm)	Material Description
150	**Reinforced Concrete -** Minimum 32 MPa - Class C1 Concrete
100	**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, fill and/or Bedrock

Note: It is recommended that the concrete slab be reinforced with 152 x 152 MW25.8/25.8 gauge welded wire mesh. The reinforcement would be adequately supported by 50 mm thick masonry blocking with a wire mesh overlap of a minimum 150 mm.

To minimize the potential differential frost heave at the interface between the rigid pavement structure and adjacent asphalt pavement structures, a frost taper should be over-excavated below the asphalt pavement structure. It is recommended that a minimum 500 mm thick frost taper, consisting of a Granular B Type II placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of the SPMDD using suitable vibratory equipment, extend horizontally at least 1.5 m beyond the outside edge of the concrete pad. The frost taper beyond the horizontal section should slope up to match the pavement structure subgrade level at a 3H:1V slope.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD using suitable vibratory equipment.

Full depth isolation joints consisting of approximately 12 mm thick compressible material are recommended adjacent to any existing rigid structure such as curbs, poles, sidewalks and buildings to allow minor movement to occur independently from each other.

Control joints, also known as contraction joints provide a location where drying shrinkage cracks or cracking attributed to frost heave can occur without affecting the appearance of the concrete pad. The saw cut control joints should be placed at a minimum 2.4 m grid with a depth of 50 mm and a maximum width of 5 mm.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter foundation drainage system is considered optional for the proposed structure. A perimeter foundation drainage system provides an outlet for perched surface water trapped below frost heave sensitive structures such as exterior sidewalks.

The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet.

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

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or equivalent) 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. The recommended minimum thickness of soil cover is 2.1 m (or equivalent).

6.3 Excavation Side Slopes

The excavations for the proposed development will be through a native silty clay material. The subsurface soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Above the groundwater level, for excavations to depths of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Shallower slopes should be provided for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be installed.

The slope cross-sections recommended above are for temporary slopes. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance 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.

A trench box is recommended to be installed 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 should not remain open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with City of Ottawa standards and specifications.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extend at a minimum to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A crushed stone, should extend from the spring line of the pipe to a minimum of 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should consist of the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

6.5 Groundwater Control

The groundwater infiltration into the excavations should be low to moderate depending on the subsurface soil conditions. The contractor should be prepared to collect and pump groundwater infiltration volumes from the excavation trenches.

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

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

6.6 Winter Construction

Precautions should be provided if winter construction is considered for this project. The subsurface soil conditions 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 installation of straw, propane heaters and tarpaulins or other suitable means. 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 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 during construction. 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.

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

The following is recommended to be completed once the site plan and development are determined:

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

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

❑ Observation of all subgrades prior to backfilling.

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

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

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

A report confirming the construction has been completed in general accordance with the 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 report recommendations are in accordance with the present understanding of the project. Paterson requests permission to review the grading plan, once available, and recommendations when the drawings and specifications are complete.

The recommendations are based on information gathered at the specific test locations and could only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities.

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 Mr. Abdu El-Arab or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 12, 2019
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Report Distribution

- ❏ Mr. Abdu El-Arab (3 copies)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Gas Station & Convenience Store
6175 Rockdale Road, Vars, Ontario

DATUM TBM - Top of existing well cover. Geodetic elevation = 76.72m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2017 May 27

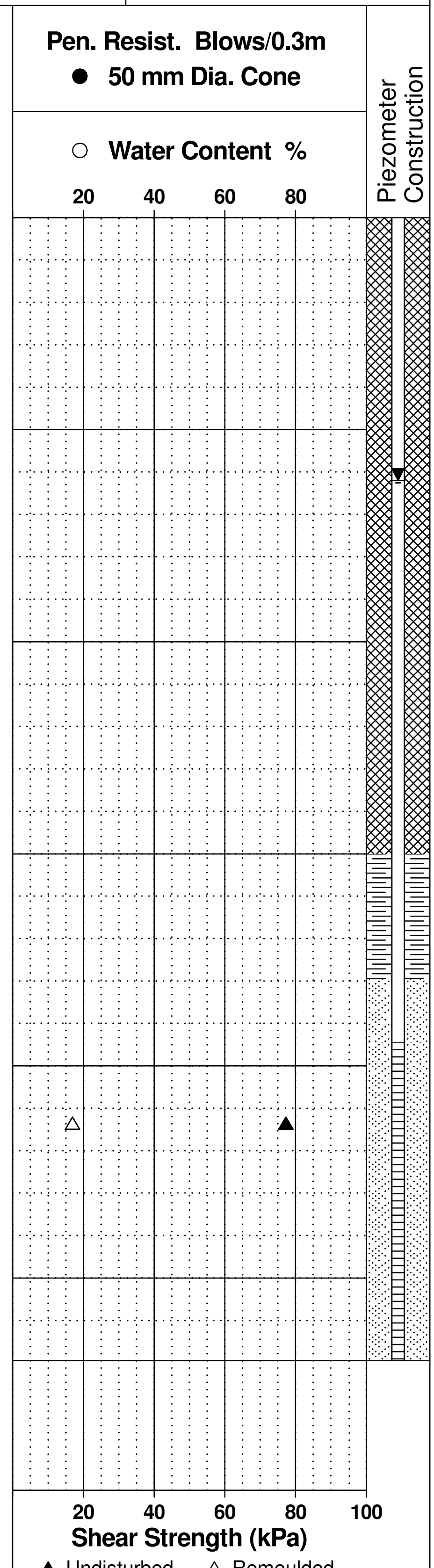
FILE NO. PG4132

HOLE NO. BH 1

SOIL DESCRIPTION (GROUND SURFACE)	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Asphaltic concrete	0.05					0	76.49
FILL: Crushed stone	0.30						
FILL: Dark grey sandy clay, some silt and gravel		SS	1	67	29	1	75.49
	2.13	SS	2	25	6	2	74.49
Loose, grey **SILTY SAND,** some clay	2.44						
Stiff, grey **SILTY CLAY,** some sand and gravel		SS	3	100	4		
	3.60	SS	4	100	3	3	73.49
Stiff, grey **SILTY CLAY,** some sand	4.27					4	72.49
GLACIAL TILL: Very dense, grey gravel, some clay, silt and sand	5.18	SS	5	40	50+	5	71.49
BEDROCK: Fractured shale	5.39	SS	6	50	50+		

End of Borehole
Practical refusal to augering at 5.39m depth

(GWL @ 1.24m - June 1, 2017)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Gas Station & Convenience Store
6175 Rockdale Road, Vars, Ontario

DATUM TBM - Top of existing well cover. Geodetic elevation = 76.72m.

FILE NO. PG4132

REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger

DATE 2017 May 27

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.64
Asphaltic concrete	0.06						
FILL: Crushed stone	0.20						
FILL: Dark grey sandy clay, some silt and gravel	1.68	SS	1	33	15	1	75.64
GLACIAL TILL: Loose, grey silty sand, some clay and gravel	2.90	SS	2	42	8	2	74.64
		SS	3	83	9		
GLACIAL TILL: Compact, brown to grey clayey sand, some silt and gravel	4.19	SS	4	100	14	3	73.64
		SS	5	0	50+	4	72.64

End of Borehole

Practical refusal to augering at 4.19m depth

(GWL @ 2.04m - June 1, 2017)

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Gas Station & Convenience Store
6175 Rockdale Road, Vars, Ontario

DATUM TBM - Top of existing well cover. Geodetic elevation = 76.72m.

FILE NO. **PG4132**

REMARKS

HOLE NO. **BH 3**

BORINGS BY CME 55 Power Auger

DATE 2017 May 27

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Crushed stone	0.46
FILL: Dark grey silty clay with sand and gravel	1.83
Compact, grey **SILTY SAND**	2.59
GLACIAL TILL: Very dense, gravel with clay, silt and sand	4.37

End of Borehole

Practical refusal to augering at 4.37m depth

(GWL @ 2.72m - June 1, 2017)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	92	9
SS	2	83	9
SS	3	33	11
SS	4	58	69
SS	5	94	50+

DEPTH (m)	ELEV. (m)
0	76.69
1	75.69
2	74.69
3	73.69
4	72.69

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Gas Station & Convenience Store
6175 Rockdale Road, Vars, Ontario

DATUM TBM - Top of existing well cover. Geodetic elevation = 76.72m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2017 May 27

FILE NO. PG4132

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.37
Asphaltic concrete	0.08						
FILL: Crushed stone	0.30						
FILL: Brown sand	2.03	SS	1		7	1	75.37
		SS	2		5	2	74.37
FILL: Grey silty sand	2.13						
End of Borehole							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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

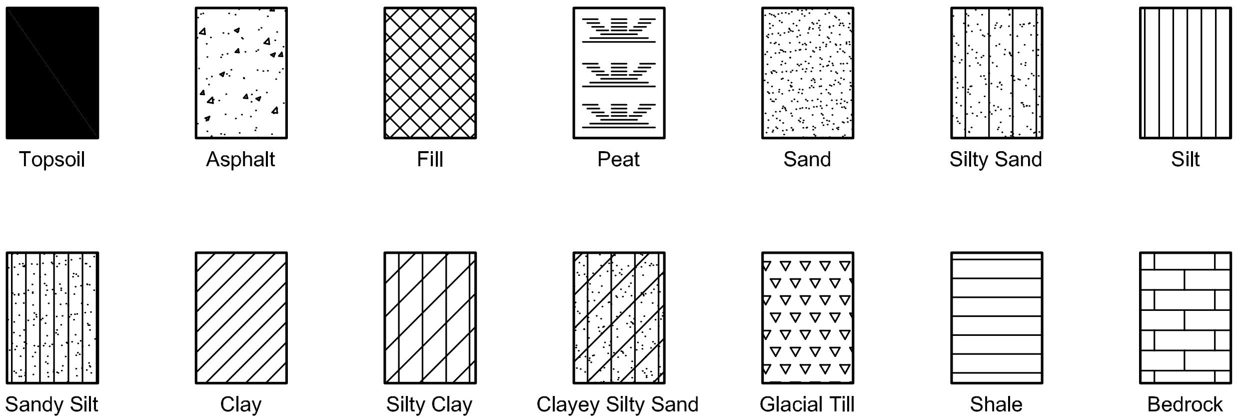
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

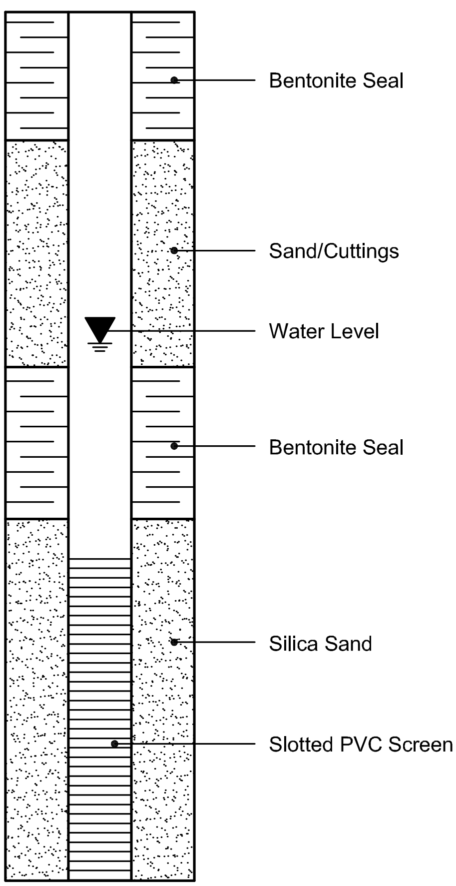
SYMBOLS AND TERMS (continued)

STRATA PLOT

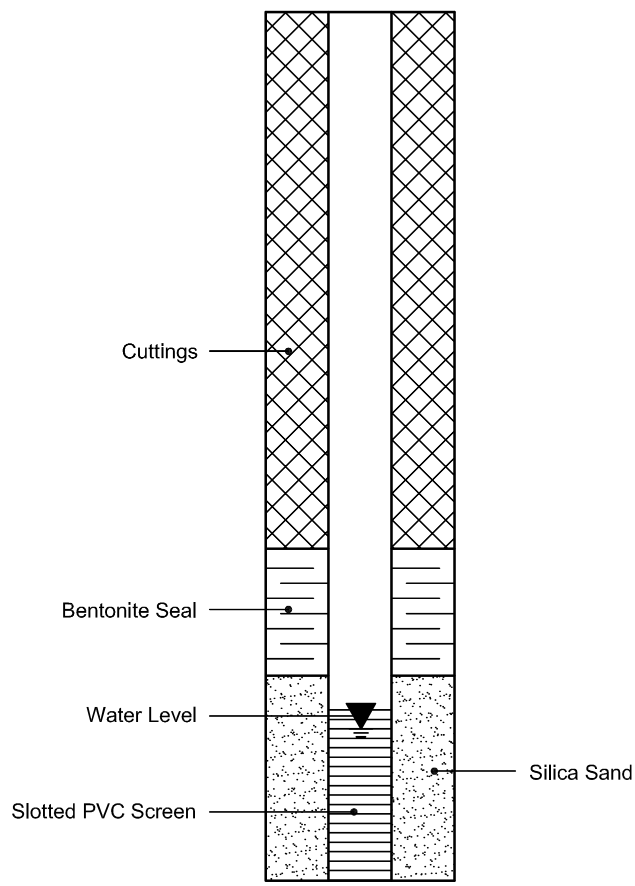


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 21582

Report Date: 02-Jun-2017

Order Date: 29-May-2017

Project Description: PG4132

	Client ID:	BH2-SS3	-	-	-
	Sample Date:	26-May-17	-	-	-
	Sample ID:	1722126-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.29	-	-	-
Resistivity	0.10 Ohm.m	14.7	-	-	-
Anions					
Chloride	5 ug/g dry	335	-	-	-
Sulphate	5 ug/g dry	60	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4132-1 - TEST HOLE LOCATION PLAN

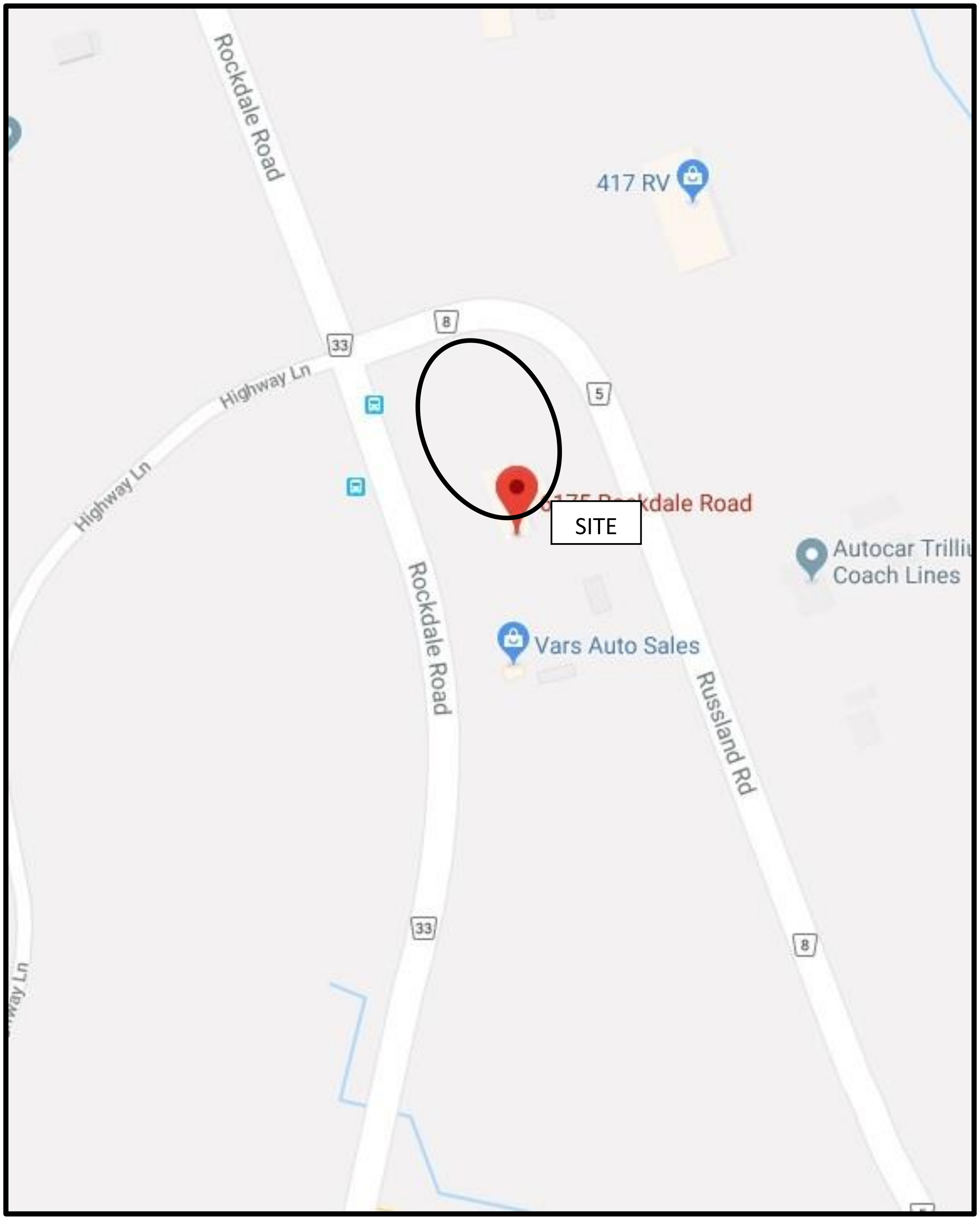
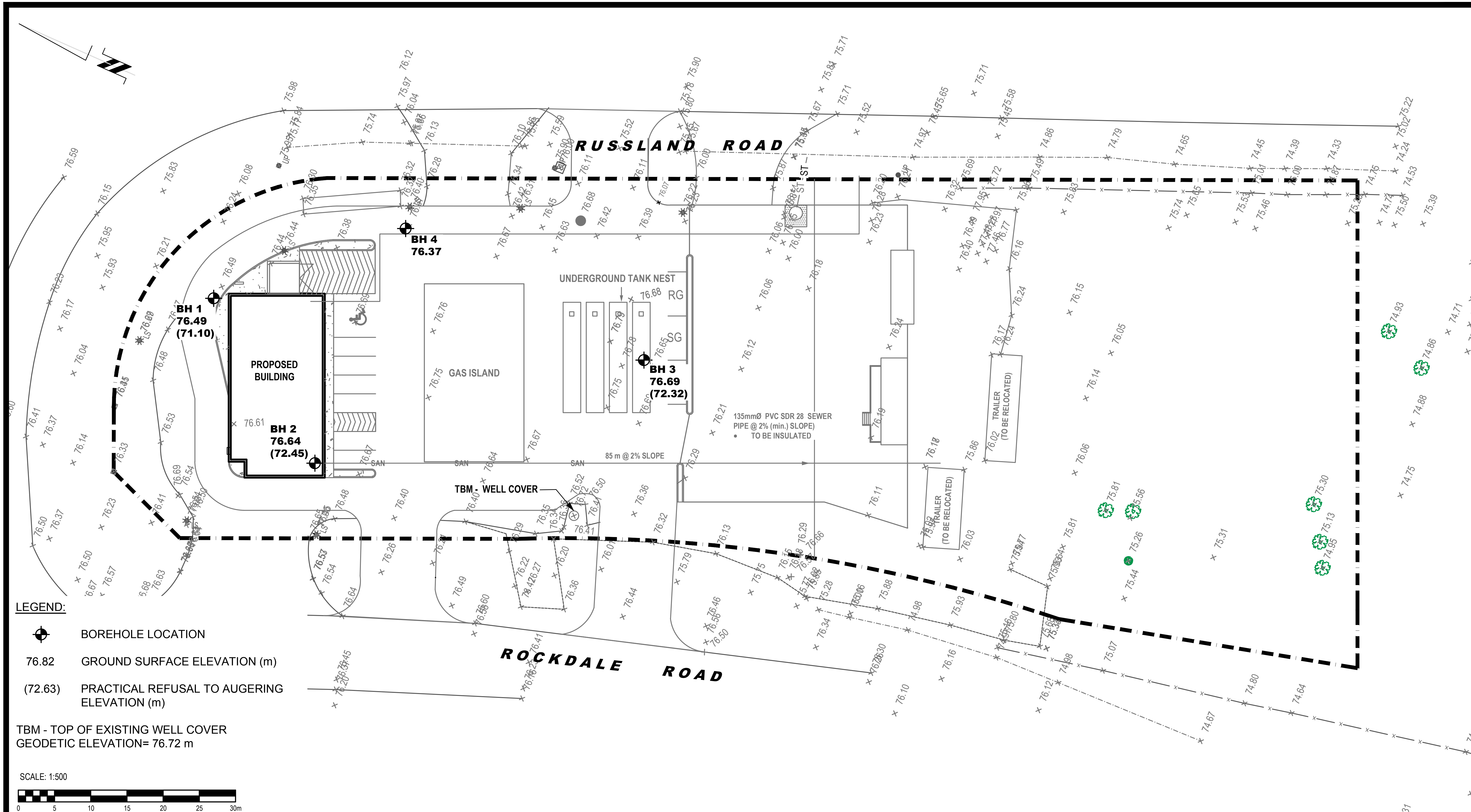


FIGURE 1

KEY PLAN



LEGEND:

BOREHOLE LOCATION

76.82 GROUND SURFACE ELEVATION (m)

(72.63) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - TOP OF EXISTING WELL COVER
GEODETIC ELEVATION= 76.72 m

SCALE: 1:500

0 5 10 15 20 25 30m

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NO.	REVISIONS	DATE	INITIAL
1	UPDATED BASE PLAN AND UPDATED TO GEODETIC ELEVATIONS	12/06/2019	CB

MR. ABDU EL-ARAB

GEOTECHNICAL INVESTIGATION - PROPOSED GAS STATION & CONVENIENCE STORE
6175 ROCKDALE ROAD, VARS

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 06/2019
Drawn by: RCG	Report No.: PG4132-1
Checked by: CB	Dwg. No.: PG4132-1
Approved by: FA	Revision No.: 1

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