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May 10, 2019
PG3565-LET.01 Revision 1

Prestige Design & Construction (FISHER) Ltd.
50 Camelot Drive
Ottawa, Ontario
K2G 5X8

Attention: **Mr. Enzo Di Chiara, P.Eng.**

Subject: **Geotechnical Investigation**
Proposed 9 Storey Residential Building
1110 Fisher Avenue - Ottawa

Dear Sir,

Further to your request, Paterson Group (Paterson) carried out a geotechnical investigation for the proposed multi-storey residential development to be located at 1110 Fisher Avenue in the City of Ottawa, Ontario.

The proposed multi-storey residential development is understood to consist of a 9 storey building with 3 levels of underground parking. The following report presents the findings and provides geotechnical recommendations for the design and construction of the subject development.

1.0 Field Investigation

The field program for the geotechnical investigation was conducted on July 17, 2015. At that time, 5 boreholes were completed to depths ranging from 2.1 to 6.7 m below the existing grade. The boreholes were completed with a low clearance track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of field personnel under the direction of a senior engineer from the geotechnical department.

Ground surface elevations at the test hole locations were referenced to a temporary benchmark (TBM), consisting of the southeast corner of an existing bell high speed switch concrete pad. An assumed geodetic elevation of 82.03 m was assigned to the TBM.

The test hole locations and TBM are presented on Drawing PG3565-1 - Test Hole Location Plan.

2.0 Field Observations

The subject site is currently occupied by a two storey residential building along with the associated asphalt covered driveways on the south and north side of the existing structure. Also, mature trees were surround the property boundary, the site is relatively flat and at grade with Fisher Avenue.

The subsurface profile at the borehole locations consists of a topsoil or fill material overlying firm to stiff native silty clay layer. A loose to dense glacial till layer, consisting of a grey silty sand matrix with some gravel and cobbles and trace clay, was encountered below the silty clay layer at all borehole locations. Practical refusal to augering was encountered at depths between 2.1 and 6.7 m below the existing grade. Specific details of the subsurface profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of limestone from the Bobcaygeon formation with an anticipated overburden thickness of 3 to 10 m.

Established on the field measurements, soil sample moisture contents, consistency and colouring, the long-term groundwater level is expected at a depth of 2.5 to 3.5 m below original ground surface. Groundwater is subject to seasonal fluctuations and could vary at the time of construction. The following table provides the groundwater measurements at the borehole locations:

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level (m)		Date
		Depth	Elevation	
BH 1	81.35	Dry	--	July 28, 2015
BH 2	80.99	3.45	77.54	July 28, 2015
BH 3	81.93	3.38	78.55	July 28, 2015
BH 4	81.52	Dry	--	July 28, 2015
BH 5	82.77	Dry	--	July 28, 2015

Note: Ground surface elevations at the test hole locations were referenced to a temporary benchmark (TBM), consisting of the southeast corner of an existing bell high speed switch concrete pad. An assumed geodetic elevation of 82.03 m was assigned to the TBM.

3.0 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed residential development. The proposed multi-storey building will be founded on conventional spread footings placed within the bedrock unit.

Site Grading and Preparation

Asphalt, topsoil and deleterious fill, such as material containing organic materials, should be stripped from under any building and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the proposed building perimeter. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Fill placed for grading beneath the proposed building 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. The fill material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where surface settlement is a minor concern. The material should be spread in thin lifts and at least 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 material should be compacted in maximum 300 mm thick loose lifts and compacted to a minimum density of 95% of the SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless tested and approved for placement or placed in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Foundation Design

Footings placed on the sound bedrock can be designed using a bearing resistance value at serviceability limit states (SLS) of **2,500 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **4,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Auxiliary footings (canopy, air shafts and ramps) placed on the undisturbed silty clay or the glacial till deposits can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **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, in the dry, prior to the placement of concrete for footings. The contractor could consider placing a lean concrete mud slab over the footing bearing surface to minimize disturbances due to worker traffic and weather.

The bearing resistance values at SLS for footings placed on soil will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. Footings bearing on the sound bedrock will have negligible post-construction settlements.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Above the groundwater level, adequate lateral support is provided to a compact glacial till when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1.5H:1V passing through in situ soil, engineered fill or weathered bedrock. For sound bedrock, the lateral support zone is 1H:6V.

Design for Earthquakes

The proposed building can be designed to a seismic site response **Class A** as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A). The soils underlying the site are not susceptible to liquefaction. However, a site specific shear wave velocity test will be required to confirm this higher seismic classification.

Basement Slab

All overburden soil will be removed for the proposed building and the basement floor slab will be founded on a bedrock medium. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone.

In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration rates can be better assessed.

Basement Wall

It is understood that the basement walls are to be poured against a waterproofing system, which will be placed against the exposed bedrock face. A nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressures will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

Where soil is to be retained, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Two distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Static Conditions

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

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

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

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

Seismic Conditions

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

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5 \cdot K_o \cdot \gamma \cdot H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

The unconfined compressive strength of limestone typically exceeds 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on our experience with limestone bedrock of this type, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented below. Load specified rock anchor lengths can be provided, if required.

For our calculations the following parameters were used.

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	80 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the total anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 3.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	1.2	0.55	1.75	250
	2	0.8	2.8	500
	3.2	1.4	4.6	1000
	5.3	2.2	7.5	2000
125	1	0.5	1.5	250
	1.7	0.7	2.4	500
	2.6	1.1	3.7	1000
	4.1	1.8	5.9	2000

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

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

Pavement Design

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

Table 4 - Recommended Flexible 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 5 - Recommended Flexible Pavement Structure - Access Lane

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 6 - Recommended Rigid Pavement Structure Lower Level of Parking Garage	
Thickness (mm)	**Material Description**
150	**Wear Course -** Concrete slab
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Bedrock or OPSS Granular B Type II material placed over bedrock.	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD.

4.0 Design and Construction Precautions

Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is expected that insufficient room is available for exterior backfill. It is suggested that an adequate drainage system would be as follows:

- Bedrock vertical surface should be prepared to receive the proposed drainage and/or waterproofing system for the entire exposed vertical bedrock excavation. The surface will be prepared by grinding to smooth out angular sections of the bedrock.

- The requirement for a waterproofing membrane, such as a bentonite layer, will be evaluated during the excavation program and will be based on the expected groundwater infiltration volumes.

- A composite drainage layer will be placed against the bentonite membrane and along the entire height of the excavation (L1, L2 and L3 levels) and will be fastened to the bedrock and shoring system.

- It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall/footing interface to allow the infiltration of water to flow to the interior drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to the sump pit(s) within the lower basement area.

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed below the lower parking garage slab. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Protection of Footings 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 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).

Excavation Side Slopes and Temporary Shoring

It is expected that insufficient room will be available to permit excavation by open-cut methods (i.e. unsupported excavations).

Horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface. The requirement for horizontal rock anchors will be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Temporary Shoring

Temporary shoring is anticipated to be required to support the overburden for the entire perimeter of the excavation.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

For design purposes, the temporary system will most likely consist of a soldier pile and timber lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc. should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure by means of rock bolts or extending the piles into the bedrock through pre-augered holes.

Generally, its is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

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

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. Because the depth at which the apex shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less capacity, than one where the bonded length was just the bottom part of the overall anchor.

The design of the rock anchors for temporary shoring can be based on the values provided above in Section 3.0 of the current report.

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

Table 7 - Soil Parameters for Shoring System Design	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

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

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

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

Underpinning of Adjacent Structures

The proposed building will be set back from the buildings on neighbouring properties. The excavations will require review of the bedrock vertical surface by the geotechnical engineer and, if required, the stabilization of the bedrock using rock bolts or rock anchors.

Groundwater Control for Building Construction

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

The rate of groundwater flow into the excavation through the overburden and bedrock should be low to moderate for the expected subsurface conditions at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

A temporary MECP permit to take water (PTTW) will be required for this project if more than 400,000 L/day is anticipated to be pumped during the construction phase. At least 3 to 4 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground and/or surface water volumes being pumped during the construction phase (between 50,000 and 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 on anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long Term Groundwater Control

Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e. less than 25,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Winter Construction

Precautions should be provided if winter construction is considered for this project.

The subsurface conditions mainly 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, tarpaulins or other suitable means. Any excavation base 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 to avoid the introduction of frozen materials, snow or ice into the trenches.

Corrosion Potential and Sulphate

The analytical test results indicate the sulphate content is less than 0.1%. The result is indicative that Type 10 Portland cement. The chloride content, pH and resistivity of the sample are indicative of non significant factors in creating a corrosive environment for exposed ferrous metals.

Table 8 - Corrosion Potential			
Parameter	**Laboratory Results**	**Threshold**	**Commentary**
	BH3 SS4		
Chloride	15 µg/g	Chloride content less than 400 mg/g	Negligible concern
pH	7.0	pH value less than 5.0	Neutral Soil
Resistivity	51.5 ohm.m	Resistivity greater than 1,500 ohm.cm	Low Agressive
Sulphate	62 µg/g	Sulphate value greater than 1 mg/g	Negligible Concern

5.0 Recommendations

For the foundation design data provided to be applicable, a materials testing and observation services program is required to be completed. The following aspects should be performed by the geotechnical consultant:

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

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

6.0 Statement of Limitations

The recommendations provided in the report are in accordance with Paterson's present understanding of the project. Paterson request permission to review the recommendations when the drawings and specifications are completed.

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

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

The present report applies only to the project described in the report. The use of the report for purposes other than those described above or by person(s) other than Prestige Design & Construction (FISHER) Ltd. or their agents is not authorized without review by Paterson.

Best Regards,

Paterson Group Inc.

Nathan F. S. Christie, P.Eng.

LICENSED PROFESSIONAL ENGINEER
May 10, 2019
C. P. DA SILVA
PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng., ing., QP_{ESA}

Attachments
- ❏ Soil Profile and Test Data sheets
- ❏ Symbols and Terms
- ❏ Figure 1 - Key Plan
- ❏ Drawing PG3565-1 - Test Hole Location Plan

Report Distribution
- ❏ Prestige Design & Construction (FISHER) Ltd. (3 copies)
- ❏ Paterson Group (1 copy)

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Townhouse Development - 1110 Fisher Avenue
Ottawa, Ontario

DATUM TBM - Southeast corner of existing Bell high speed switch concrete pad located near the northeast corner of subject site. Geodetic elevation = 82.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 17, 2015

FILE NO. PG3565

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	81.35
TOPSOIL with wood chips (to 0.10)							
FILL: Brown silty sand with gravel (0.10 to 1.06)		AU	1				
		SS	2	75	10	1	80.35
Stiff, brown **SILTY CLAY,** trace sand (1.06 to 2.13)		SS	3	83	9	2	79.35
GLACIAL TILL: Compact to loose, brown silty sand with some clay, gravel, trace cobbles (2.13 to 4.67)		SS	4	71	13		
		SS	5	79	11	3	78.35
		SS	6	83	4	4	77.35
		SS	7	75	50+		

End of Borehole

Practical refusal to augering at 4.67m depth

(BH dry - July 28, 2015)

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

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Prop. Townhouse Development - 1110 Fisher Avenue
Ottawa, Ontario

DATUM TBM - Southeast corner of existing Bell high speed switch concrete pad located near the northeast corner of subject site. Geodetic elevation = 82.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 17, 2015

FILE NO. PG3565

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	80.99
TOPSOIL — 0.08							
		AU	1				
Stiff to firm, brown **SILTY CLAY**		SS	2	83	9	1	79.99
		SS	3	100	9	2	78.99
- grey by 2.1m depth		SS	4	83	6		
2.90						3	77.99
GLACIAL TILL: Brown silty sand with some clay and gravel		SS	5	75	8		
		SS	6	100	8	4	76.99
		SS	7	17	6	5	75.99
		SS	8	100	12	6	74.99
		SS	9	100	50+		
6.50							

End of Borehole

Practical refusal to augering at 6.50m depth

(GWL @ 3.45m-July 28, 2015)

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

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Townhouse Development - 1110 Fisher Avenue
Ottawa, Ontario

DATUM TBM - Southeast corner of existing Bell high speed switch concrete pad located near the northeast corner of subject site. Geodetic elevation = 82.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 17, 2015

FILE NO. PG3565

HOLE NO. BH 3

SOIL DESCRIPTION	Bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.13				
FILL: Brown silty sand, some clay	0.76	AU	1		
Stiff to firm, brown **SILTY CLAY,** some to trace sand	3.66	SS	2	75	9
		SS	3	100	7
		SS	4	100	4
		SS	5	100	3
GLACIAL TILL: Grey silty clay with sand, trace gravel	5.20	SS	6	100	5
		SS	7	67	4
GLACIAL TILL: Compact to dense, grey silty sand, some gravel, trace clay and cobbles	6.70	SS	8	83	21
		SS	9	58	35
End of Borehole					
(GWL @ 3.39m-July 28, 2015)					

DEPTH (m)	ELEV. (m)
0	81.93
1	80.93
2	79.93
3	78.93
4	77.93
5	76.93
6	75.93

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
Prop. Townhouse Development - 1110 Fisher Avenue
Ottawa, Ontario

DATUM TBM - Southeast corner of existing Bell high speed switch concrete pad located near the northeast corner of subject site. Geodetic elevation = 82.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 17, 2015

FILE NO. PG3565

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	81.51
FILL: Brown silty sand with gravel	0.76	AU	1				
FILL: Brown sand	1.10	SS	2	62	5	1	80.51
Firm, brown **SILTY CLAY,** trace sand	2.13	SS	3	75	7	2	79.51
GLACIAL TILL: Brown silty sand with gravel, trace cobbles	2.39	SS	4	100	50+		

End of Borehole

Practical refusal to augering at 2.39m depth

(BH dry upon completion)

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

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Townhouse Development - 1110 Fisher Avenue
Ottawa, Ontario

DATUM TBM - Southeast corner of existing Bell high speed switch concrete pad located near the northeast corner of subject site. Geodetic elevation = 82.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 17, 2015

FILE NO. PG3565

HOLE NO. BH 4A

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	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	81.52		
FILL: Brown silty sand with gravel — 0.76									
FILL: Brown sand — 1.10						1	80.52		
Firm, brown **SILTY CLAY,** trace sand — 2.13						2	79.52		
GLACIAL TILL: Brown silty sand with gravel, trace cobbles — 2.41		SS	1	100	50+				
End of Borehole Practical refusal to augering at 2.41m depth (BH dry upon completion)									

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
Prop. Townhouse Development - 1110 Fisher Avenue
Ottawa, Ontario

DATUM TBM - Southeast corner of existing Bell high speed switch concrete pad located near the northeast corner of subject site. Geodetic elevation = 82.03m.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** July 17, 2015

FILE NO. PG3565

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	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	82.77		
TOPSOIL 0.25									
FILL: Brown silty sand 0.60									
Stiff, brown **SILTY CLAY**		SS	1	67	10	1	81.77		
1.96		SS	2	64	10	2	80.77		
GLACIAL TILL: Brown silty sand, some gravel 2.08									
End of Borehole									
Practical refusal to augering at 2.08m depth									
(BH dry upon completion)									

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
Prop. Townhouse Development - 1110 Fisher Avenue
Ottawa, Ontario

DATUM TBM - Southeast corner of existing Bell high speed switch concrete pad located near the northeast corner of subject site. Geodetic elevation = 82.03m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 17, 2015

FILE NO. PG3565

HOLE NO. BH 5A

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	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	82.73		
TOPSOIL 0.25									
FILL: Brown silty sand 0.60									
Stiff, brown SILTY CLAY 2.29						1	81.73		
						2	80.73		
GLACIAL TILL: Brown silty sand with clay and gravel 2.69		SS	1	71	50+				
End of Borehole									
Practical refusal to augering at 2.69m depth									
(BH dry upon completion)									

Shear Strength (kPa): 20 40 60 80 100

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
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

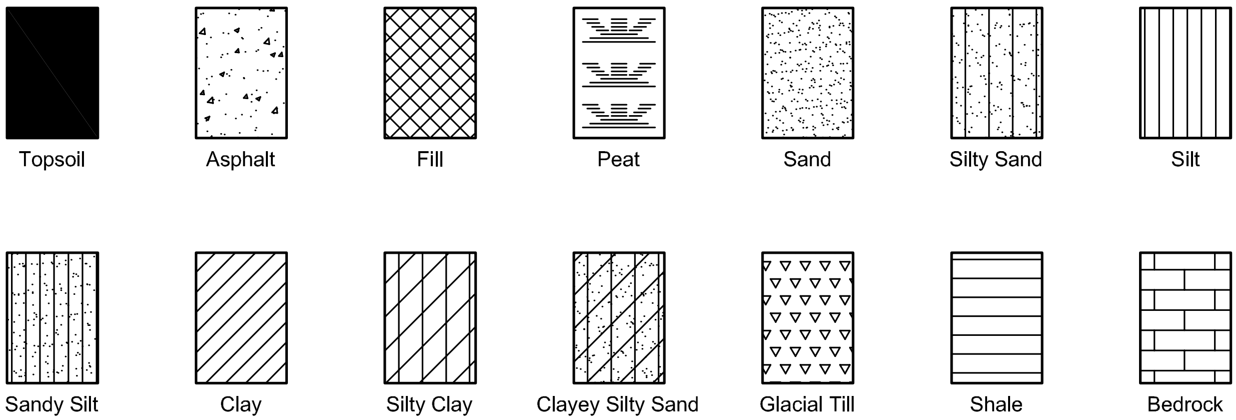
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

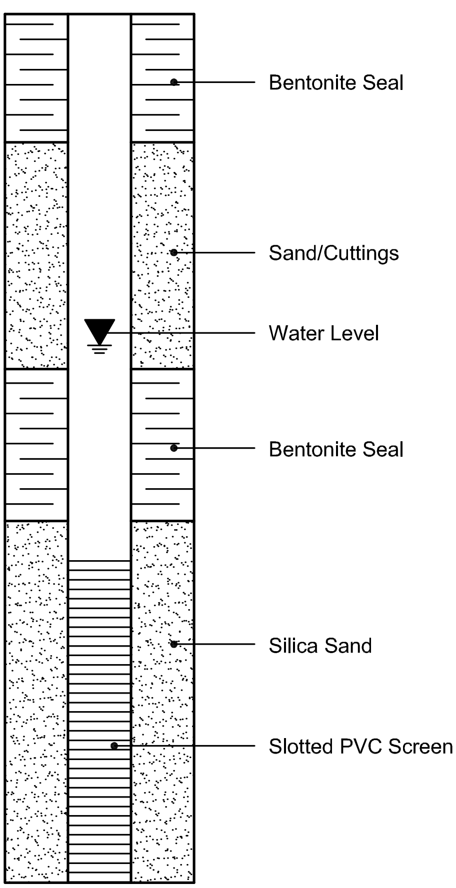
SYMBOLS AND TERMS (continued)

STRATA PLOT

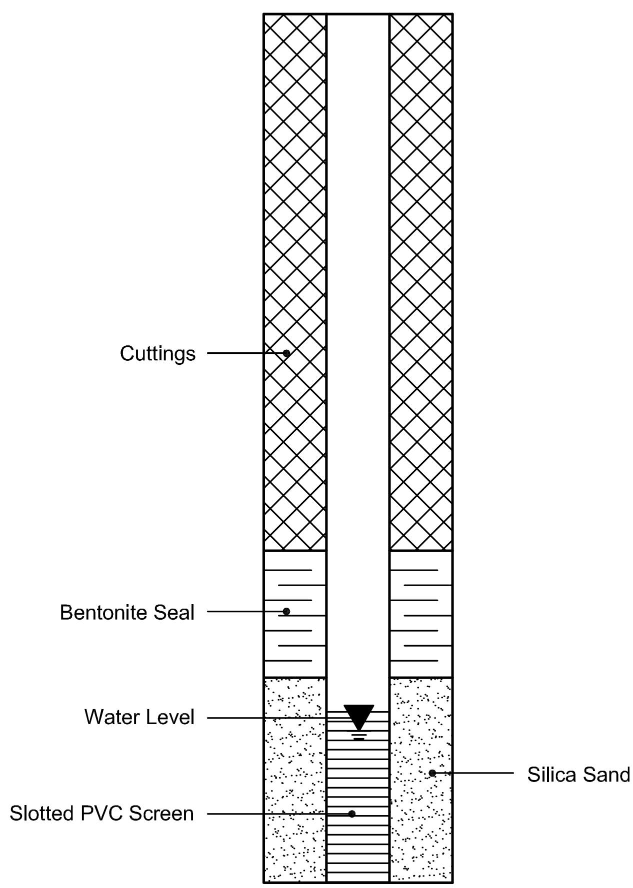


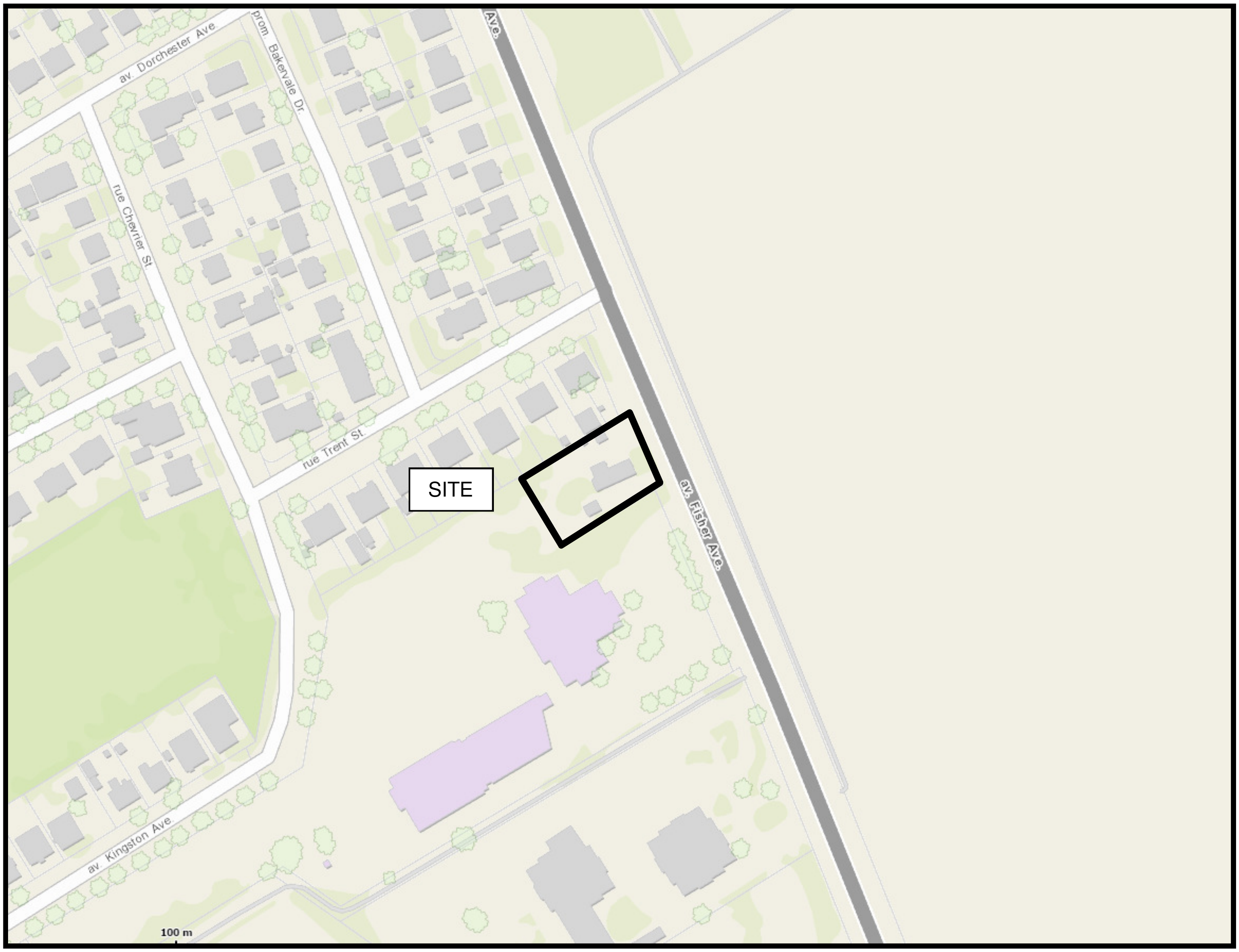
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





Source: City of Ottawa Emaps.

FIGURE 1

KEY PLAN

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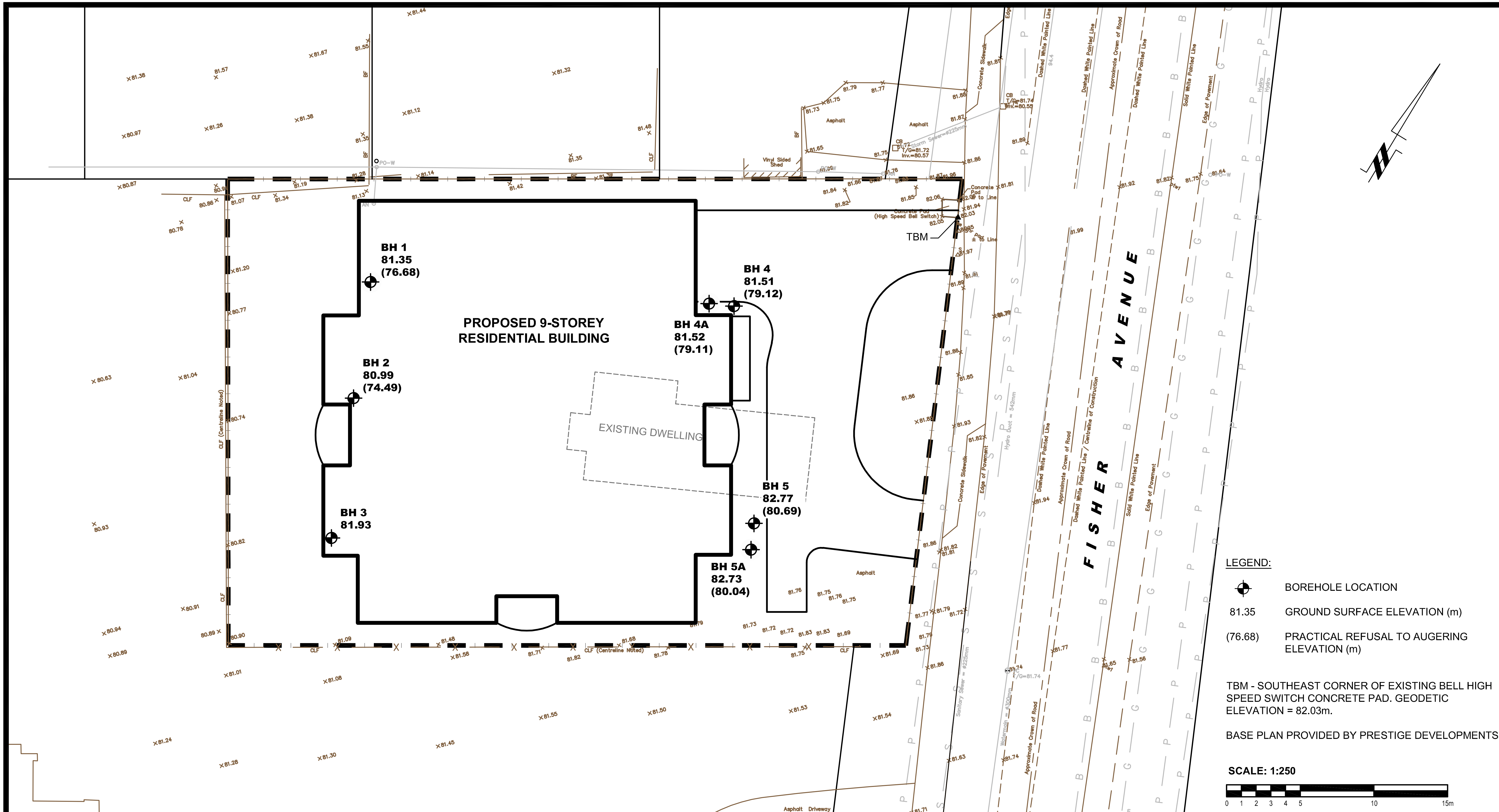

LEGEND:

(borehole symbol)	BOREHOLE LOCATION
81.35	GROUND SURFACE ELEVATION (m)
(76.68)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - SOUTHEAST CORNER OF EXISTING BELL HIGH SPEED SWITCH CONCRETE PAD. GEODETIC ELEVATION = 82.03m.

BASE PLAN PROVIDED BY PRESTIGE DEVELOPMENTS.

SCALE: 1:250

0 1 2 3 4 5 10 15m

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NO.	REVISIONS	DATE	INITIAL
1	BASE PLAN UPDATED	10/05/2019	NC

PRESTIGE DESIGN & CONSTRUCTION (FISHER) LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING - 1110 FISHER AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 08/2015
Drawn by: MPG	Report No.: PG3565-LET.01
Checked by: NC	Drawing No.: PG3565-1
Approved by: CDS	Revision No.: 1

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