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Geotechnical Investigation

Proposed Multi-Storey Building
112 Nelson Street
Ottawa, Ontario

Prepared For

Domicile Developments Inc.

November 20, 2017

Report PG4297-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Domicile Developments Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 112 Nelson Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the investigation were 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 the design.

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

2.0 Proposed Project

Based on preliminary design details, it is understood that the proposed development consists of a nine storey residential building with one level of underground parking. Associated access lanes and landscaped areas are also anticipated. All buildings currently occupying the subject site 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 current investigation was conducted on November 2, 2017. The investigation consisted of drilling three (3) boreholes extending to a maximum depth of 9.8 m below existing grade. The test hole locations were selected in a manner to provide general coverage of the proposed development. The test hole locations are shown on Drawing PG4297-1-Test Hole Location Plan included in Appendix 2.

The boreholes were advanced with a truck-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The drilling procedure consisted of augering to the required depth at the selected location, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split spoon and auger samples were classified on site and placed in sealed plastic bags. All soil samples were transported to our laboratory. The depths at which the split spoon samples were recovered from the boreholes are shown as SS on the Soil Profile and Test Data sheets in Appendix 1.

In conjunction with the recovery of the split spoon samples, the Standard Penetration Test (SPT) was conducted. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) at two borehole locations, BH 1 and BH 3. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

Monitoring wells were installed in the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ❏ 3.0 m of slotted 51 mm diameter PVC screen at base the base of the boreholes.
- ❏ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ❏ No.3 silica sand backfill within annular space around screen.
- ❏ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ❏ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The borehole locations were laid out in the field and surveyed by Paterson. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM) consisting of the top of spindle of the fire hydrant located on the site. An assumed elevation of 100.00 m was assigned to the TBM. The location and ground surface elevations at the borehole locations are presented on Drawing PG4297-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and the results are discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a two-storey warehouse, loading dock, one-storey warehouse, and an associated asphalt surfaced parking lot. It is anticipated that the existing buildings will be demolished as part of the subject development.

The ground surface at the subject site, which has an approximate L-shape, is relatively flat and at-grade with the surrounding properties. The subject site is bordered by commercial properties to the south, west, and north, a multi-storey residential property to the northeast, and Nelson Street to the east.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consists of an asphalt pavement structure underlain by fill extending to depths of approximately 2.0 to 2.2 m. The fill was observed to consist of about 0.1 m to 0.3 m of crushed stone overlying a loose to compact silty sand with trace to some gravel and occasional construction debris. A stiff to very stiff, brown to grey silty clay deposit was encountered underlying the fill, extending to depths of about 7.1 to 7.5 m. Underlying the silty clay deposit, a glacial till deposit was observed consisting of a firm to very stiff, grey silty clay to clayey silt with trace gravel. Practical refusal to the DCPTs was encountered at 11.7 and 11.6 m in boreholes BH 1 and BH 3, respectively.

Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and shale of the Verulam Formation with an approximate drift thickness of 10 to 15 m.

4.3 Groundwater

The groundwater level (GWL) readings were recorded at the borehole locations on November 9, 2017 and are presented in Table 1 below and in the Soil Profile and Test Data sheets. Based on the field observations, experience with the local area, moisture levels and the colouring of the recovered samples, the water level is expected between 5 and 6 m below existing grade. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole due to the seasonal changes, which can lead to water perching inside the boreholes resulting in higher water levels than noted during the investigation. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore groundwater levels could differ at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 1	99.33	6.10	93.23	November 9, 2017
BH 2	99.07	4.50	94.57	November 9, 2017
BH 3	98.95	Dry	-	November 9, 2017

Note: The test hole locations were located in the field and surveyed by Paterson Group. The elevations are referenced to an assumed datum.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed development. It is recommended that a raft style foundation bearing on the stiff to very stiff silty clay layer provide foundation support for the proposed multi-storey building. Alternatively, a deep foundation consisting of end-bearing piles would also provide a suitable foundation. Exterior structures, if present, can be founded over conventional style shallow foundation placed over an undisturbed, stiff to very stiff brown-grey silty clay bearing surface.

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

5.2 Site Preparation

Stripping Depth

Asphalt, topsoil, and any deleterious fill, such as those containing organic materials, should be removed from within the perimeter of the proposed building and other settlement sensitive structures.

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

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the proposed building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing resistance values - Conventional Shallow Footings

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, place on an undisturbed, stiff to very stiff silty clay bearing surface 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**.

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 footings.

Footings designed using the above noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to an engineered fill, silty clay bearing medium when a plane extending a minimum of 1.5H:1V, from the footing perimeter to the founding soil/engineered fill.

Permissible Grade Raise

Due to the high building loads anticipated and the undrained shear strength testing values noted within the silty clay deposit encountered at the test hole locations, a permissible grade raise restriction of 1 m is recommended for grading in close proximity of the proposed building.

Raft Foundation

Based on the expected loads of the proposed building, it is anticipated that a raft foundation will be required to found the proposed building. For our design calculations, a nine storey building with one level of underground parking was assumed. It is expected that the excavation will extend between 3 to 4 m below existing ground surface. The maximum serviceability limit state (SLS) contact pressure (includes the raft embedment compensation) can be taken to be **185 kPa**. It should be noted that the weight of the raft slab and everything above has to be included when designing with this value. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **250 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **7 MPa/m** for a contact pressure of **185 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, sensitive silty clay is not elastic and limits have to be placed on the stress ranges of a particular modulus. The proposed building can be designed using the above parameters and a total and differential settlement of 25 and 20 mm, respectively.

Pile Foundation

Given the depth of the DCPT refusal encountered in the field investigation, consideration may be given to using a deep foundation system driven to refusal in the bedrock for foundation support of the proposed multi-storey building. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ultimate limit state (ULS) values are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two to four piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data					
Pile Outside Diameter (mm)	**Pile Wall Thickness (mm)**	**Geotechnical Axial Resistance**		**Final Set (blows/ 12 mm)**	**Transferred Hammer Energy (kJ)**
		SLS (kN)	**Factored at ULS (kN)**		
245	9	925	1110	6	27
245	11	1050	1260	6	31
245	13	1200	1440	6	35

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

If a higher seismic site class is required (Class C), a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the OBC 2012.

5.5 Basement Slab

It is recommended that the upper 200 mm of sub-slab fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD. A concrete mud slab should be poured to protect the native soil from construction activities.

5.6 Basement Wall

The earth pressures acting on earth retaining structures are dependent on the characteristics of the structure, particularly with respect to whether it is a yielding or an unyielding structure. Due to the wall deflections expected during a seismic event, a basement wall is considered to be a yielding structure. For a typical building (not designed to post-disaster standards), walls are designed for a deflection limit of $0.025H_s$ (where H_s is the interstorey height). It should also be noted that the magnitude of wall rotation required to reach an active earth pressure state is 0.01H and to reach a passive earth pressure state is 0.02H for a stiff, cohesive soil.

The total earth pressure (P_{AE}) includes both the static earth pressure component (P_A) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

The static horizontal earth pressure (P_A) can be calculated using a triangular earth pressure distribution equal to $K_{ah}\ \gamma\ H$ where:

K_{ah} = active earth pressure coefficient of the applicable retained soil
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

There are several combinations of backfill materials and retained soils that could be applicable for the basement wall. However, it is our opinion that, provided free-draining granular backfill is used, 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 drained unit weight, γ, of 20 kN/m^3. The applicable effective unit weight of the material is 13 kN/m^3, where applicable.

For a yielding wall, the active earth pressure can be calculated using an active earth pressure coefficient, K_{ah}, of 0.33 for the free-draining granular backfill described above and a horizontal backfill profile.

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 a_c \gamma H^2/g$ 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 total earth pressure (P_{AE}) is considered to act at a height, h, (m) from the base of the wall. Where:

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

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

5.7 Pavement Structure

For design purposes, the pavement structures presented in the following tables are recommended for the design of car only parking areas and access lanes.

Table 3 - 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 4 - Recommended Flexible Pavement Structure - Access Lanes

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

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

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

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

6.2 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 of 1.5 m of soil cover (or equivalent) should be provided in this regard.

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

6.3 Excavation Side Slopes

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

Unsupported Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or should be retained by shoring systems from the beginning of the excavation until the structure is backfilled. Insufficient room is expected for majority of the excavation to be constructed 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 excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

The design and approval of the temporary shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring system 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.

The temporary shoring system could consist of a soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.

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

Table 5 - Soil Parameters	
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
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Underpinning of Adjacent Structures

If the footings of the proposed building are anticipated to undermine the footings of the neighbouring building, underpinning of this structure may be required. The depth of the underpinning will be dependent on the depth of the neighbouring foundations relative to the foundation depths of the proposed building at the subject site.

Prior to construction, it is recommended that test pits be completed along the foundation walls of the neighbouring building to evaluate the existing underside of footing elevations for underpinning design requirements.

6.4 Pipe Bedding and Backfill

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

Generally, the brown silty clay should be possible to place above the cover material if the excavation and backfilling operations are completed in dry weather conditions. Wet silty clay materials will be difficult for placement, as the high water content are impractical for the desired compaction without an extensive drying period.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

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

Infiltration levels are anticipated to be low through the excavation face. The groundwater infiltration will be controllable with open sumps and pumps.

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, typically 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.

Long-Term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or underfloor drainage system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (i.e.- less than 50,000 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.

Impacts on Neighbouring Structures

Based on our observations, no groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The neighbouring structures are expected to be founded within the silty clay or glacial till. Issues are not expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 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.

Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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 a severe to very aggressive corrosive environment.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- ❏ Review of the final design details, from a geotechnical perspective.
- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Observation of all pile installations (if utilized).
- ❏ 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 that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

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

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

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, 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 Domicile Developments Inc. or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
112 Nelson Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant (refer to Test Hole Location Plan for location. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

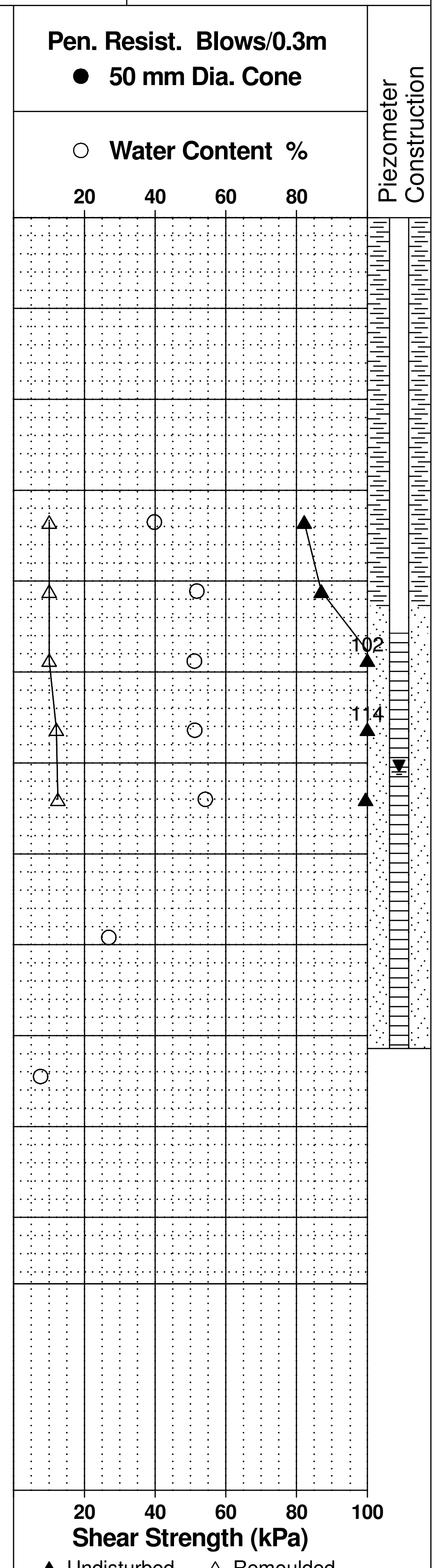
DATE 2 November 2017

FILE NO. PG4297

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10				
FILL: Crushed stone with silt and sand	0.41	AU	1		
FILL: Brown silty sand, some gravel and cobbles, trace boulders Concrete encountered at 1.8 m depth	2.23	SS	2	12	12
		SS	3	21	50+
Stiff to very stiff, brown-grey **SILTY CLAY**, trace sand and gravel -grey by 4.5 m depth -stiff to very stiff by 4.6 m depth	7.16	SS	4	100	1
		SS	5	100	P
		SS	6	83	P
		SS	7	100	P
		SS	8	100	P
		SS	9	83	P
GLACIAL TILL: Grey silty clay, trace sand and gravel	9.75	SS	10	71	P
		SS	11	62	18
Dynamic Cone Penetration commenced at 9.75m depth. Inferred **GLACIAL TILL**	11.73				
End of Borehole Practical DCPT refusal at 11.73m depth (GWL @ 6.1 m depth - Nov 9/17)					

DEPTH (m)	ELEV. (m)
0	99.33
1	98.33
2	97.33
3	96.33
4	95.33
5	94.33
6	93.33
7	92.33
8	91.33
9	90.33
10	89.33
11	88.33



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
112 Nelson Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant (refer to Test Hole Location Plan for location. Assumed elevation = 100.00m.

REMARKS

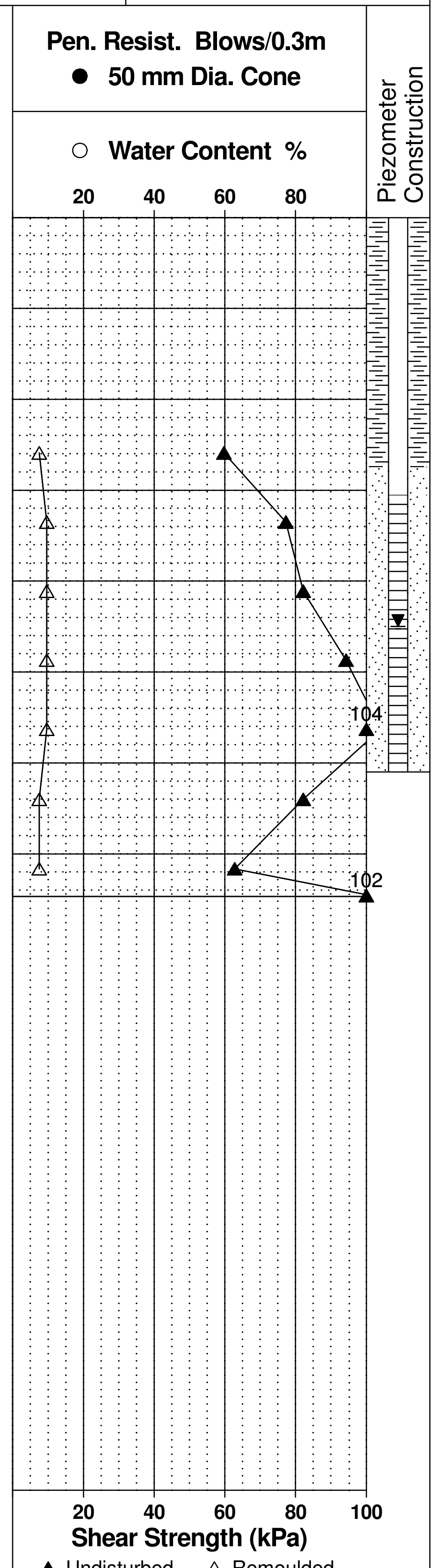
BORINGS BY CME 55 Power Auger

DATE 2 November 2017

FILE NO. PG4297

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	99.07
Asphaltic concrete	0.08	AU	1				
FILL: Crushed stone with silt and sand	0.36						
FILL: Brown silty sand, trace gravel and topsoil	1.07	SS	2	58	5	1	98.07
Loose, brown **SILTY SAND**	2.01	SS	3	42	7	2	97.07
Stiff brown-grey **SILTY CLAY,** trace sand -grey by 4.5 m depth	7.47	SS	4	83	P	3	96.07
		SS	5	100	P	4	95.07
		SS	6	75	P	5	94.07
		SS	7	83	P	6	93.07
		SS	8	83	P	7	92.07
		SS	9	100	P		
		SS	10	100	P		
End of Borehole (GWL @ 4.5 m depth - Nov 9/17)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
112 Nelson Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant (refer to Test Hole Location Plan for location. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2 November 2017

FILE NO. PG4297

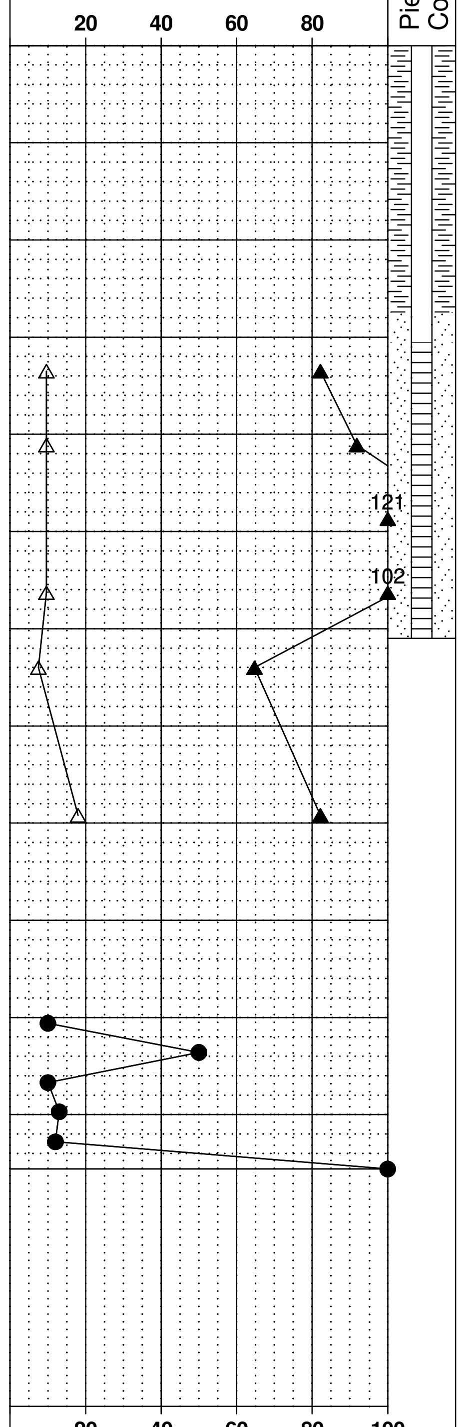
HOLE NO. BH 3

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.95
25mm Asphaltic concrete over crushed stone with silt and sand	0.28	AU	1				
Loose, brown **SILTY SAND**		SS	2	42	8	1	97.95
	2.06	SS	3	33	4	2	96.95
Stiff, brown-grey **SILTY CLAY,** trace sand		SS	4	83	2	3	95.95
		SS	5	79	P		
- very stiff to stiff and grey by 4.6m depth		SS	6	100	P	4	94.95
		SS	7	100	P	5	93.95
		SS	8	100	P	6	92.95
		SS	9	100	P		
	7.16					7	91.95
GLACIAL TILL: Grey silty clay, trace sand, gravel, cobbles and boulders		SS	10	58	P	8	90.95
	9.75	SS	11	4	4	9	89.95
Dynamic Cone Penetration commenced at 9.75m depth.						10	88.95
Inferred **GLACIAL TILL**	11.56					11	87.95
End of Borehole							
Practical DCPT refusal at 11.56m depth							
(BH dry - Nov 9/17)							

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

○ Water Content %

Piezometer Construction



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 = $(D30)^2 / (D10 \times D60)$
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

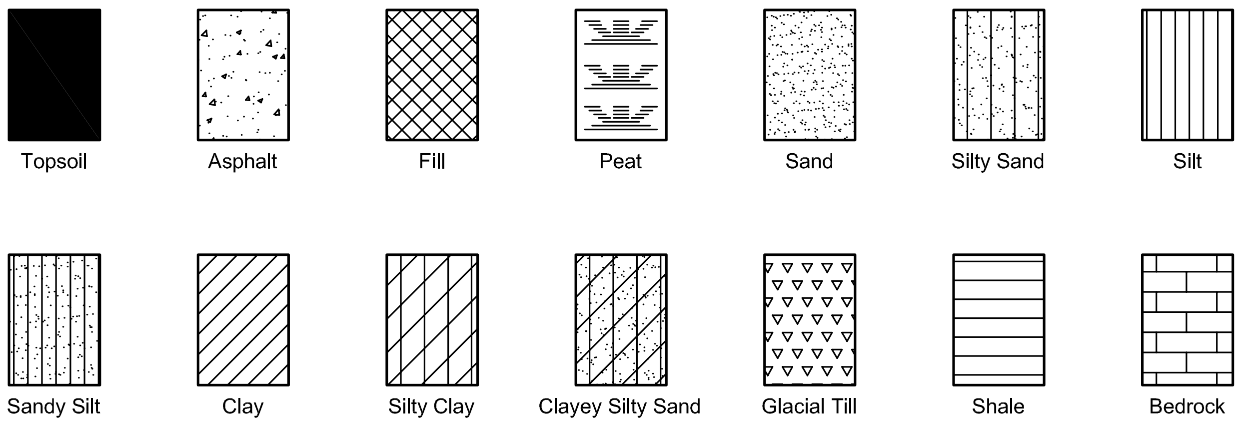
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

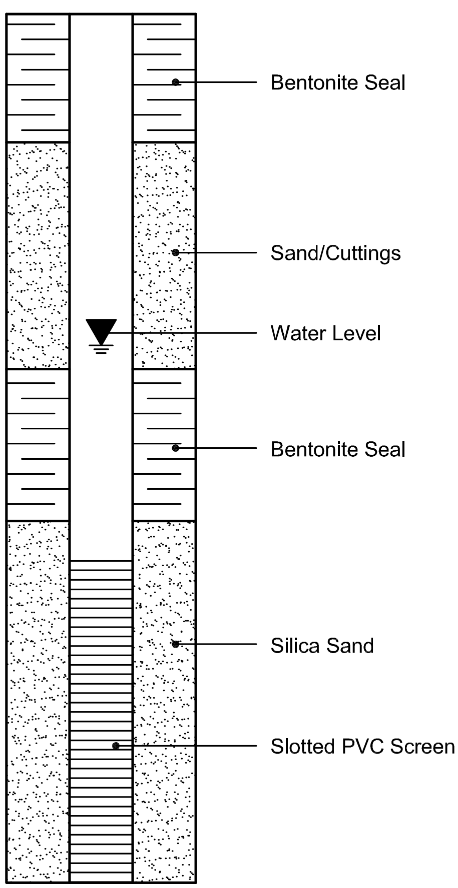
SYMBOLS AND TERMS (continued)

STRATA PLOT

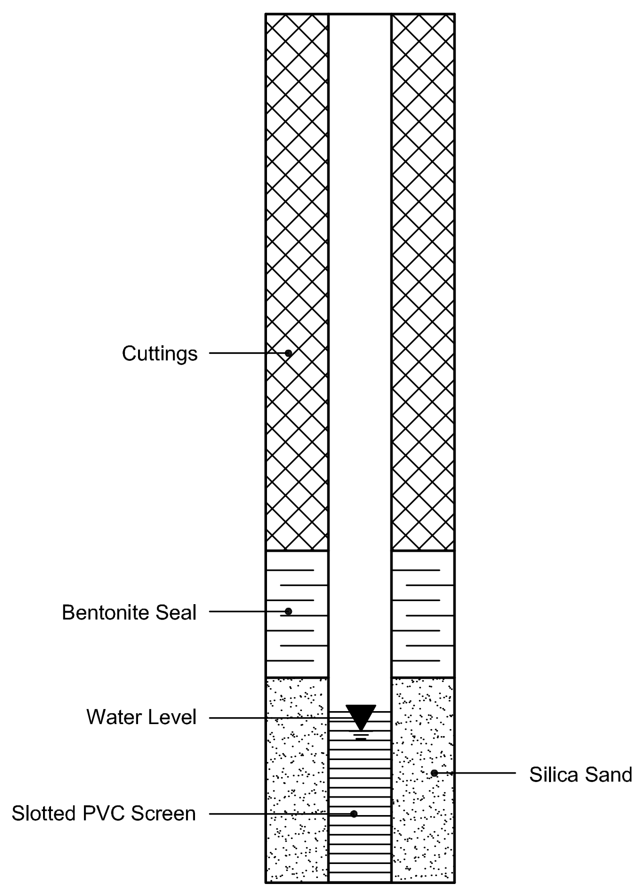


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL
LABORATORIES LTD.

Order #: 1745078

Certificate of Analysis
Client: Paterson Group Consulting Engineers
Client PO: 23054

Report Date: 09-Nov-2017
Order Date: 6-Nov-2017
Project Description: PG4297

	Client ID:	BH1 SS6	-	-	-
	Sample Date:	02-Nov-17	-	-	-
	Sample ID:	1745078-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	68.3	-	-	-
General Inorganics					
pH	0.05 pH Units	8.11	-	-	-
Resistivity	0.10 Ohm.m	11.2	-	-	-
Anions					
Chloride	5 ug/g dry	215	-	-	-
Sulphate	5 ug/g dry	521	-	-	-

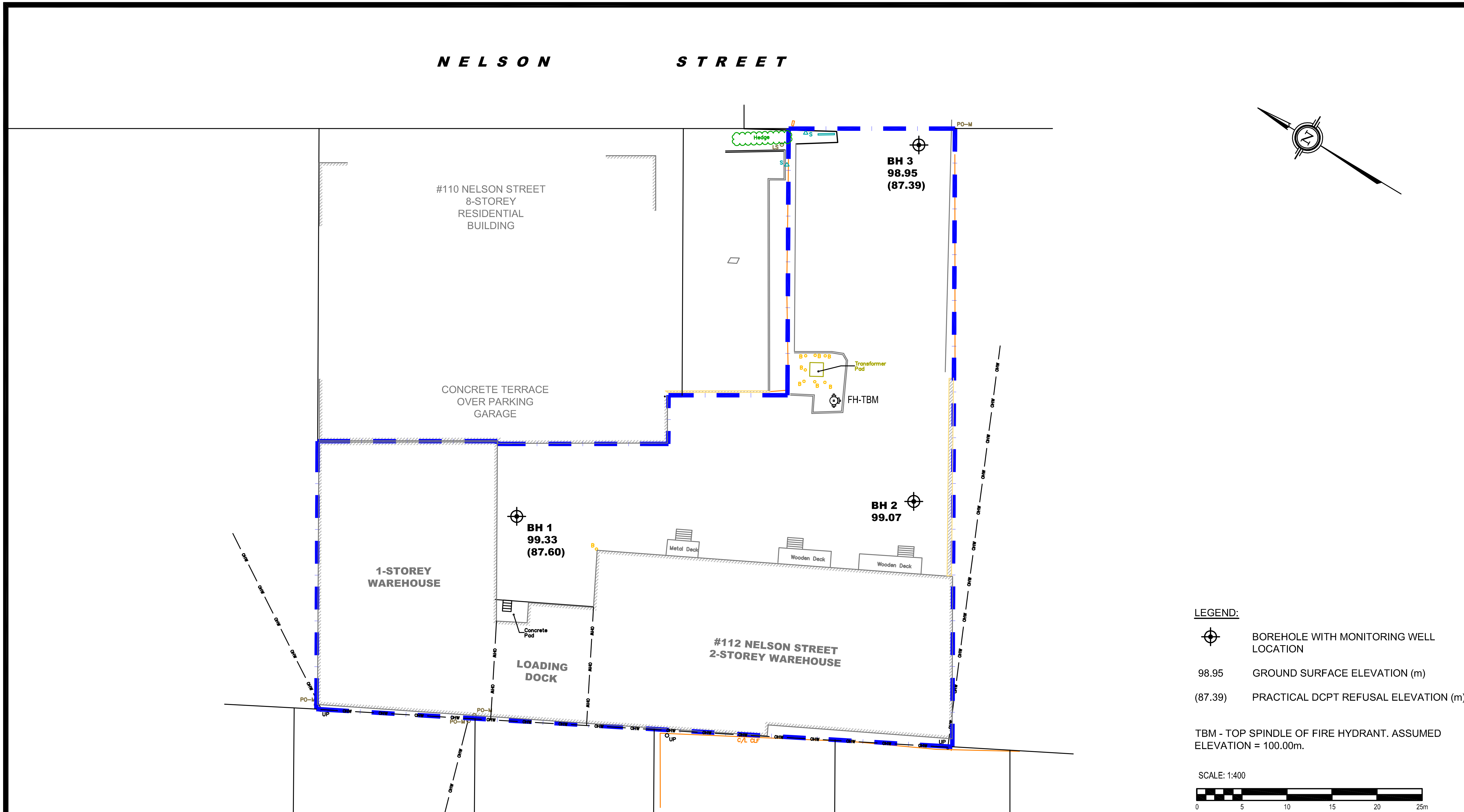
APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4297-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN



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NO.	REVISIONS	DATE	INITIAL
0			

DOMICILE DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING - 112 NELSON STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 11/2017
Drawn by: MPG	Report No.: PG4297-1
Checked by: SD	Dwg. No.: PG4297-1
Approved by: DJG	Revision No.: 0

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