

**Geotechnical
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Materials Testing

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Geotechnical Investigation

Proposed Multi-Storey Buildings
861 Clyde Avenue
Ottawa, Ontario

Prepared For

Claridge Homes

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Report PG5434-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the proposed multi-storey buildings to be located at 861 Clyde Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site 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. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Although drawings were not available during the preparation of this report, it is anticipated that the proposed development will consist of multi-storey buildings with one or more levels of underground parking. It is also anticipated that the proposed building will be surrounded by asphalt paved access lanes and parking areas with landscaped margins.

It is further anticipated that the proposed development will involve demolition of the existing commercial structure on-site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out between July 10 and 28, 2020 and consisted of 13 boreholes advanced to a maximum depth of 14.9 m below the existing ground surface. A previous geotechnical investigation was also completed at the subject site by this firm in March 2002. At that time, 7 boreholes were advanced to a maximum depth of 2.3 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG5434-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed with a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of augering and rock coring to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Bedrock samples were recovered using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Groundwater

Monitoring wells were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the current sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5434-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

4.0 Observations

4.1 Surface Conditions

The majority of the subject site is currently occupied by an existing low-rise commercial building which is generally surrounded by associated asphalt paved, heavy truck loading areas and a parking lot. Based on available aerial photos, another low-rise commercial building was located in the northwest corner of the subject site as recently as 2014. Reference should be made to aerial photographs in Figure 2 - Aerial Photograph - 2014 and Figure 3 - Aerial Photograph - 2015.

The site is bordered to the north by commercial properties and associated parking lots, to the northeast by Churchill Avenue North, to the east and south by the TransCanada Highway 417, and to the west by Clyde Avenue North. The existing ground surface across the site is relatively level at approximate geodetic elevation 75 to 76 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of an approximate 50 to 130 mm thick asphalt surface underlain by fill extending to approximate depths of 0.2 to 3.1 m. The fill was generally observed to transition from a brown to grey silty sand with crushed stone to brown silty sand with gravel.

A 200 mm thick concrete slab was observed at ground surface, overlying the fill layer at borehole BH 6 which was conducted inside the existing building.

A loose brown sandy silt to silty sand with gravel was encountered underlying the fill material at approximate depths of 0.2 to 2.6 m below the existing ground surface. The sandy silt was noted at ground surface at borehole BH 2.

A clayey silt to silty clay was encountered underlying either the fill material or silty sand to sandy silt deposit at boreholes BH 1B, BH 4, BH 7, BH 7-2 and BH 9-20, at depths ranging from 0.7 to 2.1 m.

Bedrock

Practical refusal to augering was encountered at depths ranging from 0.9 to 2.3 m in boreholes BH 1 to BH 7 during the 2002 field investigation. Within the boreholes from the current investigation, a very poor to excellent quality limestone bedrock with interbedded shale was encountered at approximate depths ranging from 1.6 to 3.2 m, and was cored to depths ranging from 5.5 to 15.3 m below the existing ground surface.

Based on available geological mapping, the bedrock at the subject site consists of interbedded limestone and dolomite of the Gull River formation with a drift thickness of 1 to 3 m.

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

4.3 Groundwater

Groundwater levels were measured in the monitoring wells installed in the boreholes on July 31 or August 5, 2020. The observed groundwater levels are summarized in Table 1. It should be noted that the groundwater level readings in the monitoring wells can be influenced by surface water becoming trapped in the backfill materials. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples.

Based on these observations, the long-term groundwater table can be expected below the bedrock surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheets presented in Appendix 1.

It should also be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Levels				
Borehole Number	Ground Surface Elev. (m)	Measured Groundwater Level		Recording Date
		Depth (m)	Elevation (m)	
BH 1-20	77.52	1.38	76.14	August 5, 2020
BH 2-20	77.41	1.01	76.40	July 31, 2020
BH 3-20	78.05	1.78	76.27	July 31, 2020
BH 4-20	78.48	2.18	76.30	August 5, 2020
BH 5-20	77.79	1.46	76.33	July 31, 2020
BH 6-20	77.59	1.93	75.66	July 31, 2020
BH 7-20	78.24	3.28	74.96	July 31, 2020
BH 8-20	78.05	2.83	75.22	July 31, 2020
BH 9-20	78.06	2.82	75.24	July 31, 2020
BH 10-20	78.32	2.88	75.44	July 31, 2020
BH 11-20	77.95	2.64	75.31	August 5, 2020
BH 12-20	78.48	2.78	75.70	August 5, 2020
BH 13-20	78.49	1.82	76.67	August 5, 2020
Note: Ground surface at test hole locations were surveyed by Paterson and are referenced to a geodetic datum				

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered suitable for the proposed development, from a geotechnical perspective. The proposed multi-storey buildings at the subject site are recommended to be founded on conventional shallow footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the excavation of the underground parking levels. Hoe ramming is an option where only small quantities of bedrock need to be removed. Line drilling and controlled blasting is recommended where large quantities of bedrock need to be removed. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

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

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

Due to the relatively shallow depth of the bedrock surface and the anticipated founding level for the proposed building, all existing overburden material should be excavated from within the proposed building footprint.

Bedrock Removal

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

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

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

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

Vibration Considerations

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

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

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

Fill Placement

Fill used for grading beneath the proposed buildings should consist 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 buildings and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded limestone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa**, incorporating a geotechnical resistance factor of 0.5. Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Footings placed over a clean, surface sounded limestone bedrock bearing surface can also be designed using a factored bearing resistance value at ultimate limit states (ULS) of **4,500 kPa**, incorporating a geotechnical resistance factor of 0.5, if founded on limestone bedrock and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footprint of the footings. At least one drill hole should be completed per major footing. The drill hole inspection should be carried out by the geotechnical consultant.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock or soil bearing medium will require a lateral support zone of 1H:1V (or shallower).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If a higher seismic site class is required (Class A or B), 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 Ontario Building Code (OBC) 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

For the proposed development, it is anticipated overburden soil will be removed from the building footprints, leaving the bedrock as the founding medium for the basement floor slabs. It is anticipated that the basement area for the proposed buildings will be mostly parking and the recommended pavement structures noted in Subsection 5.8 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

Any soft areas in the basement slab subgrades should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions encountered at the time of the field investigation, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the subfloor fill under the lower basement floor slabs. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed multi-storey buildings. However, 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 of 20 kN/m^3 .

It is expected that a portion of the basement walls are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 23.5 kN/m^3 (effective 15.5 kN/m^3) where this condition occurs. Further, a seismic earth pressure component will not be applicable for the foundation wall which is poured against the bedrock face. It is expected that the seismic earth pressure 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.

Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective 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.

Lateral Earth Pressures

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 material
- γ = unit weight of fill of the applicable retained material (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 Earth Pressures

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:

$$a_c = (1.45 - a_{max}/g) a_{max}$$

$$\gamma = \text{unit weight of fill of the applicable retained soil (kN/m}^3\text{)}$$

$$H = \text{height of the wall (m)}$$

$$g = \text{gravity, 9.81 m/s}^2$$

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 $P_o = 0.5 K_o \gamma H^2$, 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.

5.7 Rock Anchor Design

Overview of Anchor Features

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 a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. 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 individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

The anchor should be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed building, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of shale ranges between about 50 and 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 0.4, can be calculated. 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 existing bedrock information, 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

Parameters used to calculate rock anchor lengths are provided in Table 2 on the next page.

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=0.575 and s=0.00293
Unconfined compressive strength - Limestone	50 MPa
Unit weight - Submerged Bedrock	15.2 kN/m ³
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 3. The factored tensile resistance values given in Table 3 are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed buildings are determined.

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	2.0	1.0	3.0	450
	2.8	1.2	4.0	600
	3.3	1.7	5.0	750
	4.5	2.0	6.5	1000
125	1.7	0.8	2.5	600
	2.0	1.2	3.2	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

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.

5.8 Pavement Structure

Car only parking areas, heavy truck parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 4 and 5.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas	
Thickness (mm)	Material Description
50	Wear Course - 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 Pavement Structure Access Lanes and Heavy Truck Parking Areas	
Thickness (mm)	Material Description
40	Wear Course - Superpave 12.5 Asphaltic Concrete
50	Binder Course - Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil 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.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is anticipated that the portion of the proposed building foundation walls located below the long-term groundwater table (approximate geodetic elevation 75 m) will be placed against a groundwater infiltration control system. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater which comes in contact with the proposed building's foundation walls.

For the groundwater infiltration control system for the foundation walls, the following is recommended:

- ☐ Line drill the excavation perimeter.
- ☐ Hoe ram any irregularities and prepare bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface, as required based on site inspection by Paterson.
- ☐ Place a suitable membrane against the prepared bedrock surface, such as a bentomat liner system or equivalent. The membrane liner should extend from 4 m below existing grade down to footing level. The membrane liner should also extend horizontally a minimum 600 mm below the footing at underside of footing level.
- ☐ Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
- ☐ Pour foundation wall against the composite drainage system.

It is recommended that 100 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of any water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration for the underground parking levels. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where required, 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, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose. A waterproofing system should be provided for the elevator pits (pit bottom and walls).

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 alone, or a minimum of 0.6 m of soil cover in conjunction with adequate foundation insulation, should be provided.

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

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

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.

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

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

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

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The design and approval of the 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 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 may consist of a soldier pile and 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, the shoring systems should be provided with tie-back rock anchors to ensure the stability.

The toe of the shoring is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is used.

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

Table 6 - 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
Unit Weight (γ), kN/m ³	21
Submerged 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 used above the groundwater level while the effective unit weight should be used below the groundwater level.

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

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover

materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the SPMDD.

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

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. 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.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

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 MECP review of the PTTW application.

Impacts on Neighbouring Properties

It is anticipated that one or more levels of underground parking are planned for the proposed development. Due to the presence of relatively shallow bedrock at, and in the vicinity of, the subject site, no adverse effects to neighbouring properties are expected as a result of any dewatering which may be required.

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 the 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, 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.

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 geotechnical aspects of the excavating contractor's shoring design, prior to construction.
- ☐ Review the bedrock stabilization and excavation requirements.
- ☐ Review proposed waterproofing and foundation drainage design and requirements.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ 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 work 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.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our 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 those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 Claridge Homes or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Scott S. Dennis, P.Eng.



David J. Gilbert, P.Eng.

Report Distribution

- ☐ Claridge Homes (email copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 10, 2020

FILE NO. PG5434

HOLE NO. BH 1-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE												
Asphaltic concrete	0.10	AU	1			0	77.52					
FILL: Brown silty sand with crushed stone - with gravel by 0.5m depth	2.01	SS	2	79	91	1	76.52					
		SS	3	56	50+	2	75.52					
BEDROCK: Good to excellent quality, grey limestone interbedded with shale		RC	1	93	81	3	74.52					
		RC	2	100	81	4	73.52					
		RC	3	100	98	5	72.52					
						6	71.52					
		RC	4	100	98	7	70.52					
						8	69.52					
		RC	5	100	98	9	68.52					
						10	67.52					
		RC	6	100	100	11	66.52					
						12	65.52					
		RC	7	100	100	13	64.52					
						14	63.52					
		RC	8	100	100							
		RC	9	100	96							
End of Borehole	14.96											
(GWL @ 1.38m - July 28, 2020)												

DATUM	Geodetic
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FILE NO. PG5434

REMARKS

HOLE NO. **BH 2-20**

BORINGS BY CME-55 Low Clearance Drill

DATE July 10, 2020

[illegible]

SOIL PROFILE AND TEST DATA

**Geotechnical Investigation
Proposed Multi-Storey Development - 861 Clyde Avenue
Ottawa, Ontario**

FILE NO. PG5434

HOLE NO. **BH 3-20**

DATE July 13, 2020

[illegible]

SOIL PROFILE AND TEST DATA

**Geotechnical Investigation
Proposed Multi-Storey Development - 861 Clyde Avenue
Ottawa, Ontario**

FILE NO. PG5434

HOLE NO. **BH 4-20**

DATE July 13, 2020

[illegible]

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill



DATE July 13, 2020

FILE NO.

PG5434

HOLE NO.

BH 5-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %					
								20	40	60	80		
GROUND SURFACE													
Asphaltic concrete	0.08		AU	1		0	77.79						
FILL: Brown silty sand with crushed stone	0.59		SS	2	50	8	1	76.79					
Loose, light brown SANDY SILT	1.88					2	75.79						
			RC	1	100	68	3	74.79					
			RC	2	100	95	4	73.79					
			RC	3	100	90	5	72.79					
			RC	4	100	100	6	71.79					
			RC	5	100	100	7	70.79					
			RC	6	100	98	8	69.79					
			RC	7	100	100	9	68.79					
			RC	8	100	100	10	67.79					
			RC	9	100	78	11	66.79					
							12	65.79					
							13	64.79					
							14	63.79					
							15	62.79					
End of Borehole	15.24												
(GWL @ 1.46m - July 28, 2020)													
								20	40	60	80	100	
								Shear Strength (kPa)					
								▲ Undisturbed △ Remoulded					

BEDROCK: Fair to excellent quality, grey limestone interbedded with shale

[illegible]

[illegible]

SOIL PROFILE AND TEST DATA

**Geotechnical Investigation
Proposed Multi-Storey Development - 861 Clyde Avenue
Ottawa, Ontario**

FILE NO. PG5434

HOLE NO. **BH 8-20**

DATE July 14, 2020

[illegible]


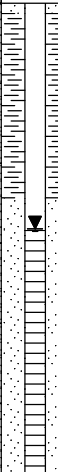









SOIL PROFILE AND TEST DATA

**Geotechnical Investigation
Proposed Multi-Storey Development - 861 Clyde Avenue
Ottawa, Ontario**

FILE NO. PG5434

HOLE NO. **BH 9-20**

DATE July 14, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
GROUND SURFACE								20	40	60	80	
Asphaltic concrete 0.08		AU	1			0	78.06					
FILL: Brown silty sand with crushed stone, some gravel 1.42		SS	2	50	14	1	77.06					
TOPSOIL 1.58		SS	3	100	50+							
Compact, brown CLAYEY SILT 2.01						2	76.06					
		RC	1	100	70							
						3	75.06					
		RC	2	100	93							
						4	74.06					
		RC	3	100	92							
						5	73.06					
End of Borehole 5.89												
(GWL @ 2.82m - July 28, 2020)												

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

**Geotechnical Investigation
Proposed Multi-Storey Development - 861 Clyde Avenue
Ottawa, Ontario**

FILE NO. PG5434

HOLE NO. **BH10-20**

DATE July 15, 2020

[illegible]

SOIL PROFILE AND TEST DATA

**Geotechnical Investigation
Proposed Multi-Storey Development - 861 Clyde Avenue
Ottawa, Ontario**

DATUM	Geodetic
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FILE NO. PG5434

REMARKS

HOLE NO. **BH11-20**

BORINGS BY CME-55 Low Clearance Drill

DATE July 15, 2020

[illegible]

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill


DATE July 15, 2020

FILE NO.

PG5434

HOLE NO.

BH12-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %					
								20	40	60	80		
GROUND SURFACE													
Asphaltic concrete	0.10		AU	1			0	78.49					
FILL: Brown silty sand with crushed stone	0.60		SS	2	54	12	1	77.49					
FILL: Brown silty sand-gravel with asphalt fragments			SS	3	46	10	2	76.49					
	2.62		SS	4	31	50+							
			RC	1	100	71	3	75.49					
			RC	2	100	88	4	74.49					
							5	73.49					
			RC	3	100	100	6	72.49					
							7	71.49					
			RC	4	100	97	8	70.49					
							9	69.49					
			RC	5	100	84	10	68.49					
							11	67.49					
			RC	6	100	87	12	66.49					
							13	65.49					
			RC	7	100	95	14	64.49					
							15	63.49					
			RC	8	100	97							
			RC	9	100	89							
End of Borehole	15.26												
(GWL @ 2.78m - July 28, 2020)													
									20	40	60	80	100
									Shear Strength (kPa)				
									▲ Undisturbed △ Remoulded				

BEDROCK: Fair to excellent quality, grey limestone interbedded with shale

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

**Proposed Multi-Storey Development - 861 Clyde Avenue
Ottawa, Ontario**

DATUM	Geodetic
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FILE NO.

PG5434

REMARKS

HOLE NO.

BH13-20

BORINGS BY CME-55 Low Clearance Drill

DATE July 29, 2020

[illegible]

JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation

**Neilson Dairy Extension, 861 Clyde Avenue
Ottawa, Ontario**

DATUM TBM - Finished floor level of existing building @ loading dock. Elevation = 78.40m.

FILE NO.

G8545

REMARKS

HOLE NO.

BH 1A

BORINGS BY CME 75 Power Auger

DATE 28 MAR 02

[illegible]

JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation

**Neilson Dairy Extension, 861 Clyde Avenue
Ottawa, Ontario**

DATUM TBM - Finished floor level of existing building @ loading dock. Elevation = 78.40m.

FILE NO.

G8545

REMARKS

HOLE NO.

BH 1B

BORINGS BY CME 75 Power Auger

DATE 28 MAR 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	77.63					
Asphaltic concrete	0.06											
FILL: Grey crushed stone	0.18											
FILL: Dark brown silty sand with gravel												
	0.68											
		AU	3			1	76.63					
Very stiff, brown SILTY CLAY												
		SS	4	45	25							
	2.18					2	75.63					
End of Borehole												
Practical refusal to augering @ 2.18m depth (BH dry upon completion)												

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation

**Neilson Dairy Extension, 861 Clyde Avenue
Ottawa, Ontario**

DATUM TBM - Finished floor level of existing building @ loading dock. Elevation = 78.40m.

REMARKS

FILE NO.

G8545

HOLE NO.

BH 2

BORINGS BY CME 75 Power Auger

DATE 28 MAR 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	77.75					
Brown SANDY SILT, trace gravel		AU	5									
- sandier with depth		SS	6		50 +							
End of Borehole												
Practical refusal to augering @ 0.94m depth (BH dry upon completion)												

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

JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation

**Neilson Dairy Extension, 861 Clyde Avenue
Ottawa, Ontario**

DATUM TBM - Finished floor level of existing building @ loading dock. Elevation = 78.40m.

FILE NO.

G8545

REMARKS

HOLE NO.

BH 3

BORINGS BY CME 75 Power Auger

DATE 28 MAR 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	77.80					
Asphaltic concrete	0.06											
FILL: Grey crushed stone	0.20											
Brown SILTY fine to medium SAND, occasional gravel		SS	7	70	11	1	76.80					
End of Borehole	1.37											
Practical refusal to augering @ 1.37m depth (BH dry upon completion)												

Shear Strength (kPa)
 ▲ Undisturbed △ Remoulded

SOIL PROFILE & TEST DATA

Geotechnical Investigation

Neilson Dairy Extension, 861 Clyde Avenue

Ottawa, Ontario

DATUM TBM - Finished floor level of existing building @ loading dock. Elevation = 78.40m.

FILE NO.

G8545

REMARKS

HOLE NO.

BH 4

BORINGS BY CME 75 Power Auger

DATE 28 MAR 02

[illegible]

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation

**Neilson Dairy Extension, 861 Clyde Avenue
Ottawa, Ontario**

DATUM TBM - Finished floor level of existing building @ loading dock. Elevation = 78.40m.

FILE NO.

G8545

REMARKS

HOLE NO.

BH 6

BORINGS BY CME 75 Power Auger

DATE 28 MAR 02

[illegible]

JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation

Neilson Dairy Extension, 861 Clyde Avenue

Ottawa, Ontario

DATUM TBM - Finished floor level of existing building @ loading dock. Elevation = 78.40m.

FILE NO.

G8545

REMARKS

HOLE NO.

BH 7

BORINGS BY CME 75 Power Auger

DATE 28 MAR 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE												
FILL: Grey crushed stone	0.08	CH	15			0	77.64					
FILL: Brown silty sand with gravel		SS	16	0	50+							
	0.76	CH	17									
Dark brown CLAYEY SILT with gravel and cobbles		SS	17									
	1.42	CH				1	76.64					
End of Borehole												
Practical refusal to augering @ 1.42m depth (BH dry upon completion)												

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
D _{xx}	-	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
D ₁₀	-	Grain size at which 10% of the soil is finer (effective grain size)
D ₆₀	-	Grain size at which 60% of the soil is finer
C _c	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C _u	-	Uniformity coefficient = D_{60} / D_{10}

C_c and C_u are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < C_c < 3$ and $C_u > 4$

Well-graded sands have: $1 < C_c < 3$ and $C_u > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

C_c and C_u 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

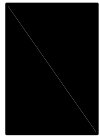
p' _o	-	Present effective overburden pressure at sample depth
p' _c	-	Preconsolidation pressure of (maximum past pressure on) sample
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
W _o	-	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.
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SYMBOLS AND TERMS (continued)

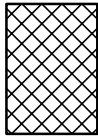
STRATA PLOT



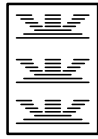
Topsoil



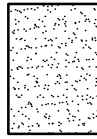
Asphalt



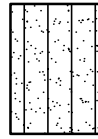
Fill



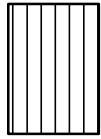
Peat



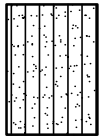
Sand



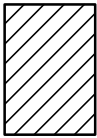
Silty Sand



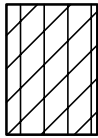
Silt



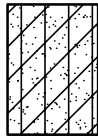
Sandy Silt



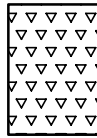
Clay



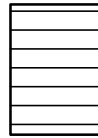
Silty Clay



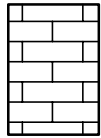
Clayey Silty Sand



Glacial Till



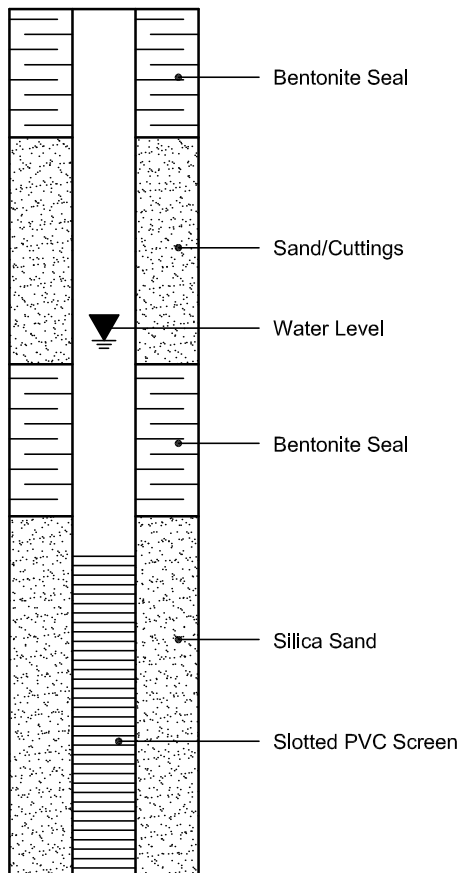
Shale



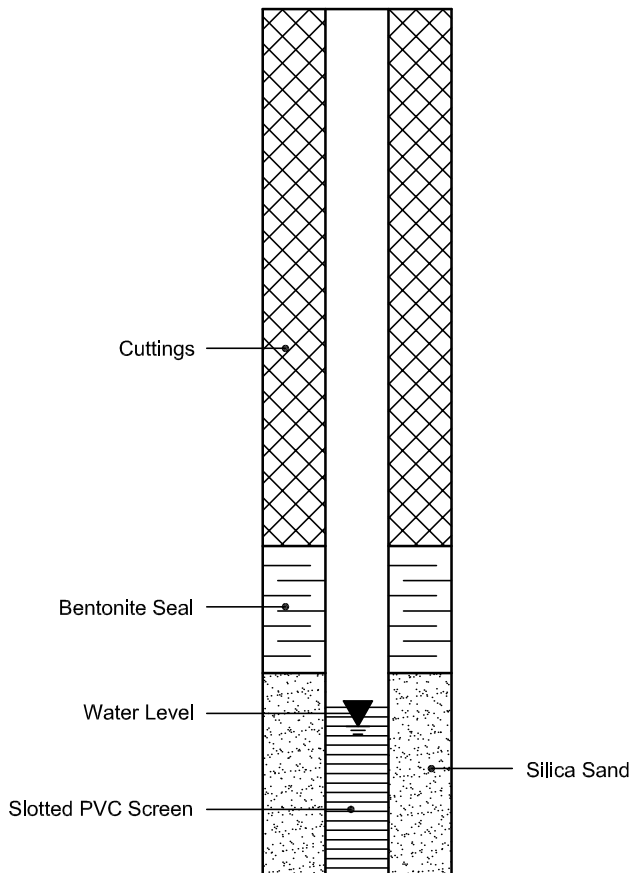
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - AERIAL PHOTOGRAPH - 2014

FIGURE 3 - AERIAL PHOTOGRAPH - 2015

DRAWING PG5434-1 - TEST HOLE LOCATION PLAN

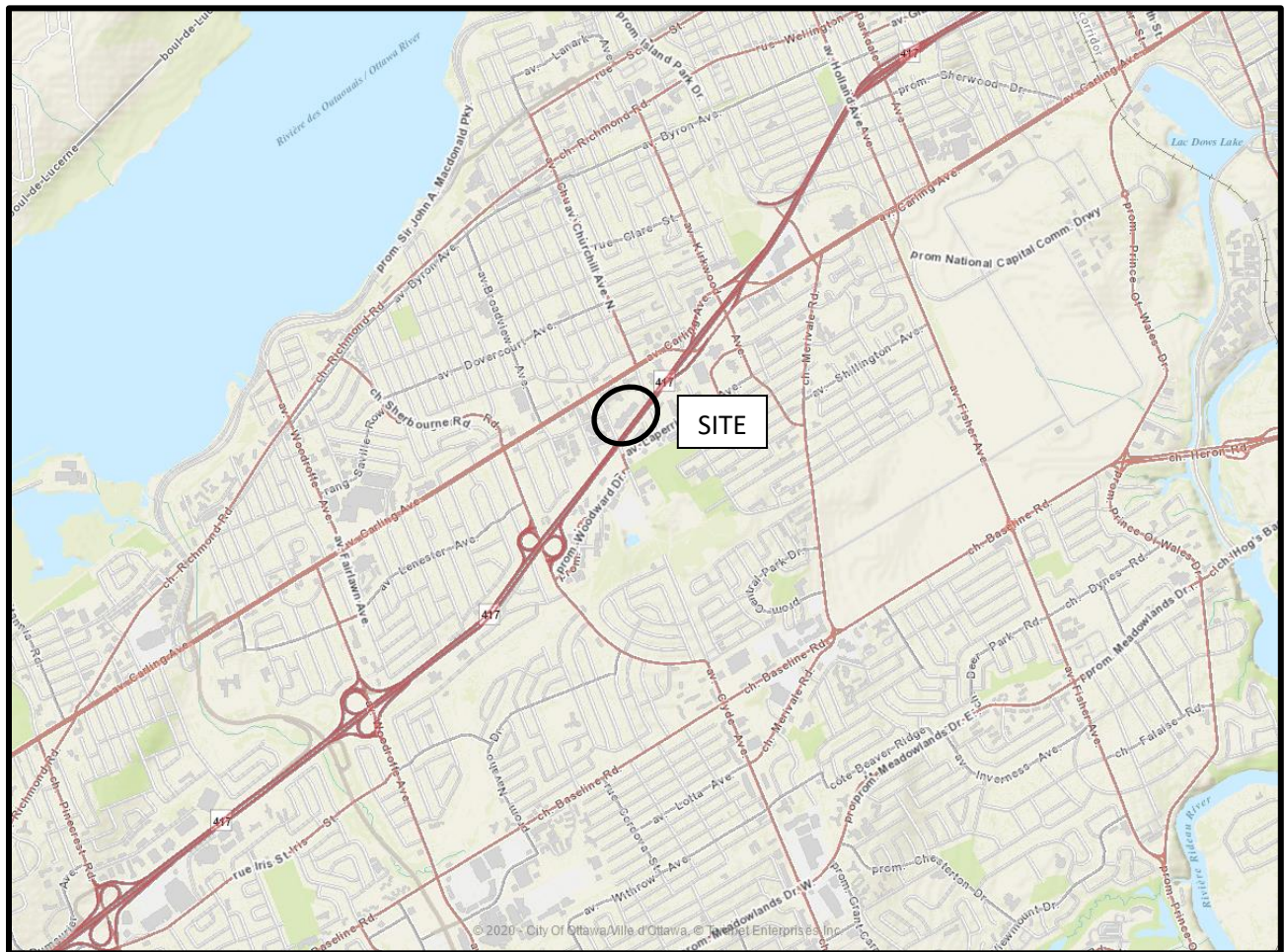


FIGURE 1

KEY PLAN

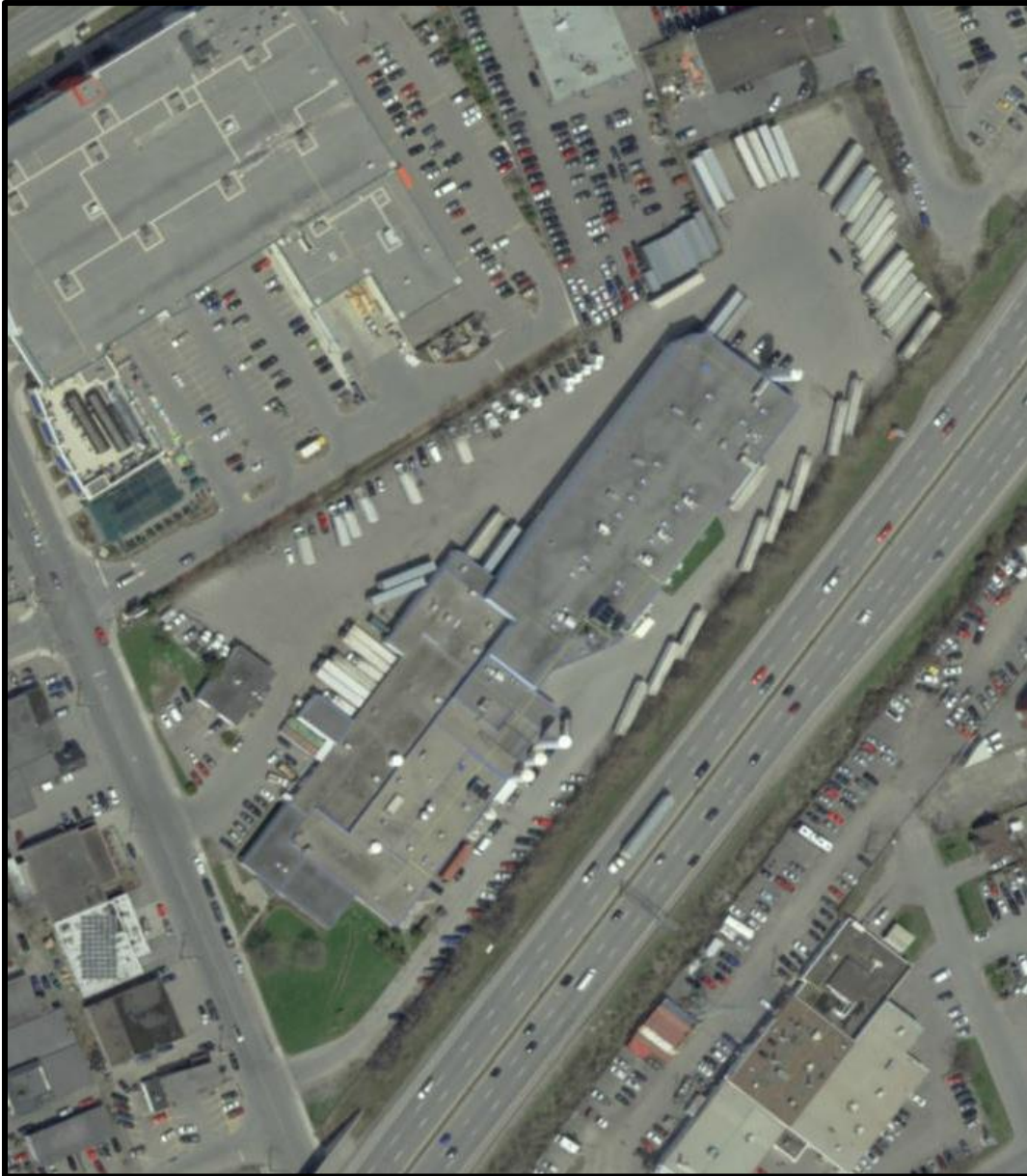


FIGURE 2

Aerial Photograph - 2014

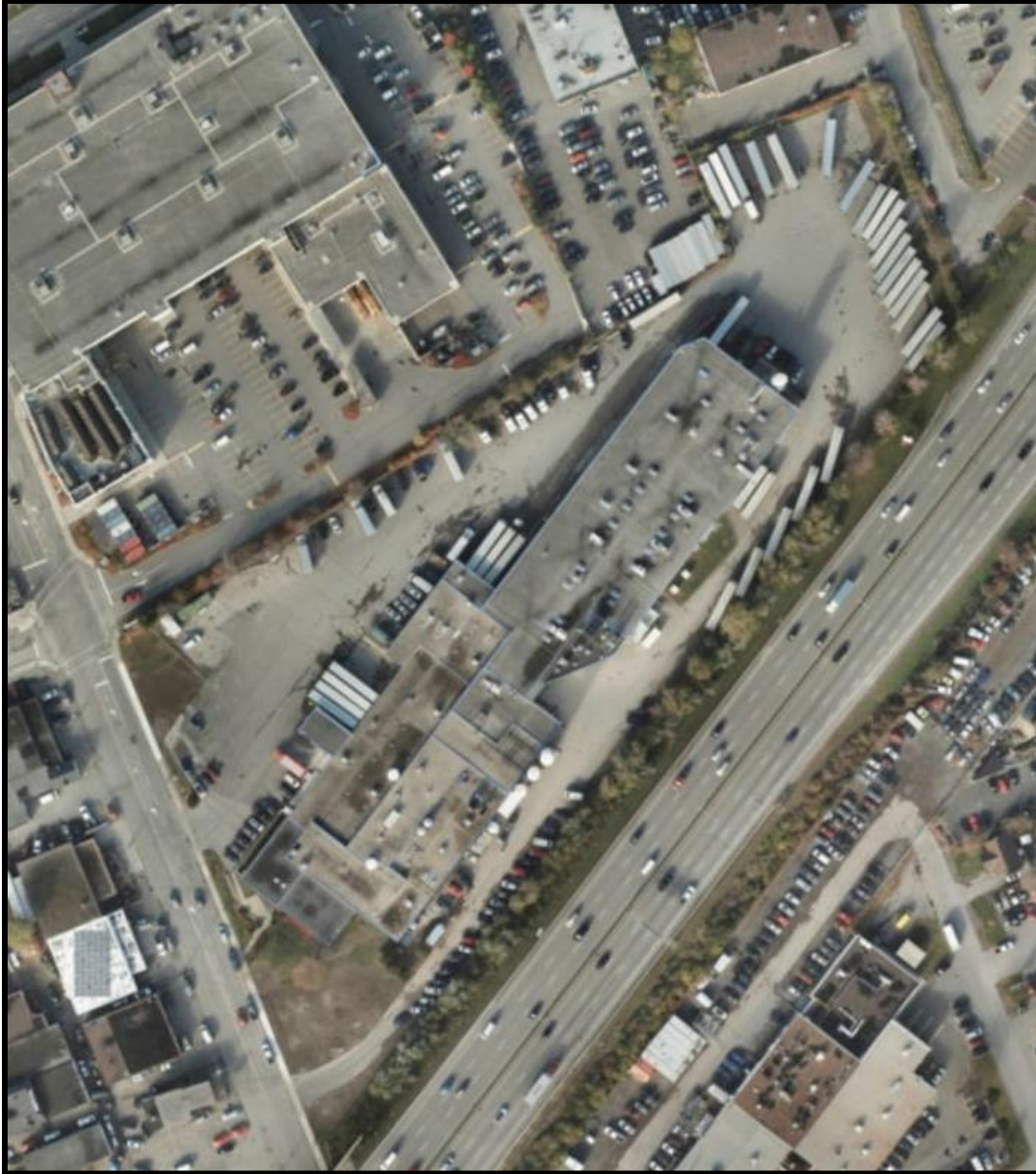
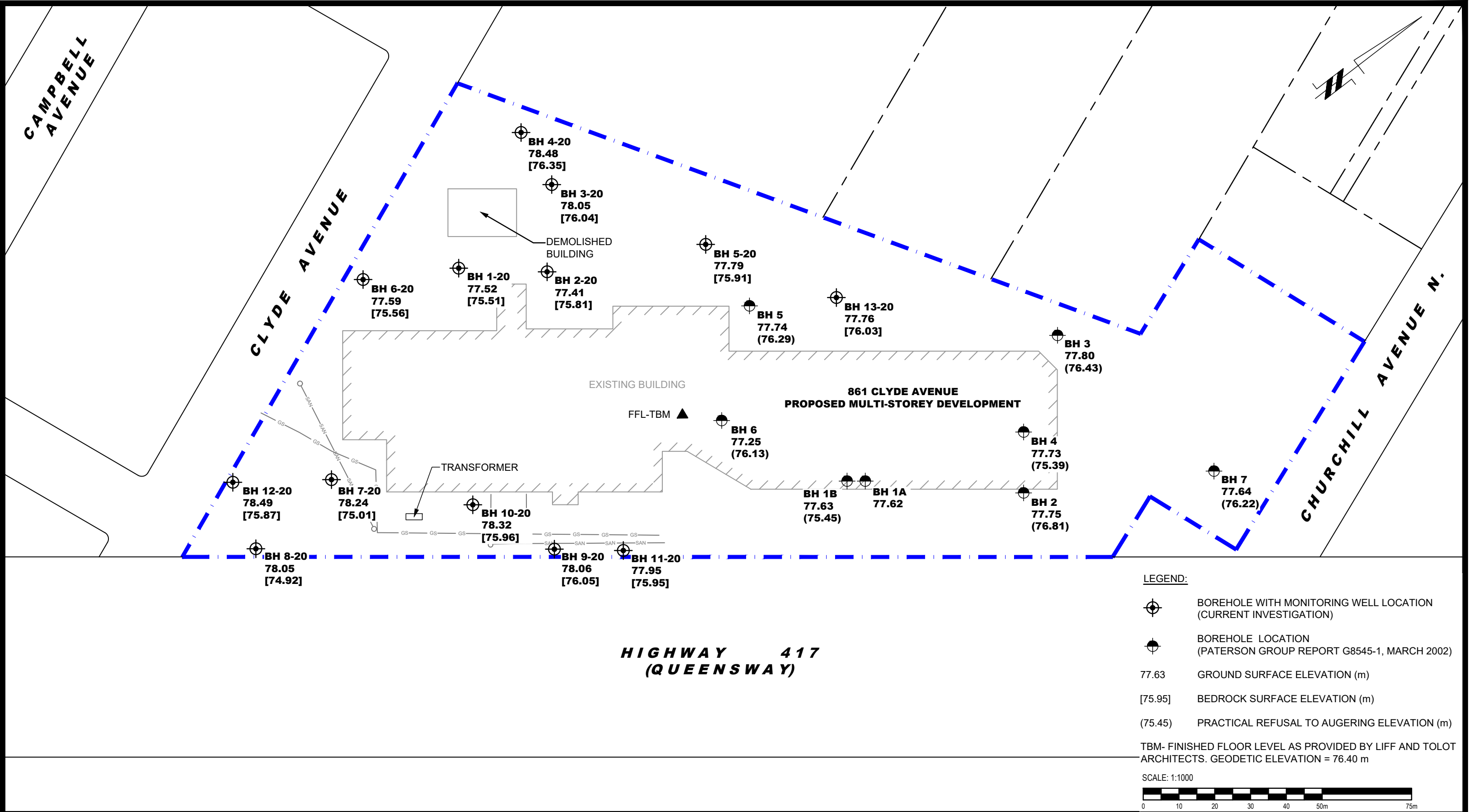


FIGURE 3

Aerial Photograph - 2015



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NO.	REVISIONS	DATE	INITIAL

CLARIDGE HOMES

GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-STOREY DEVELOPMENT - 861 CLYDE AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:1000
Drawn by:	YA
Checked by:	KP
Approved by:	SD

Date:	08/2020
Report No.:	PG5434-1
Dwg. No.:	PG5434-1
Revision No.:	

p:\autocad\drawings\geotechnical\pg5434\pg5434-1-test hole location plan.dwg