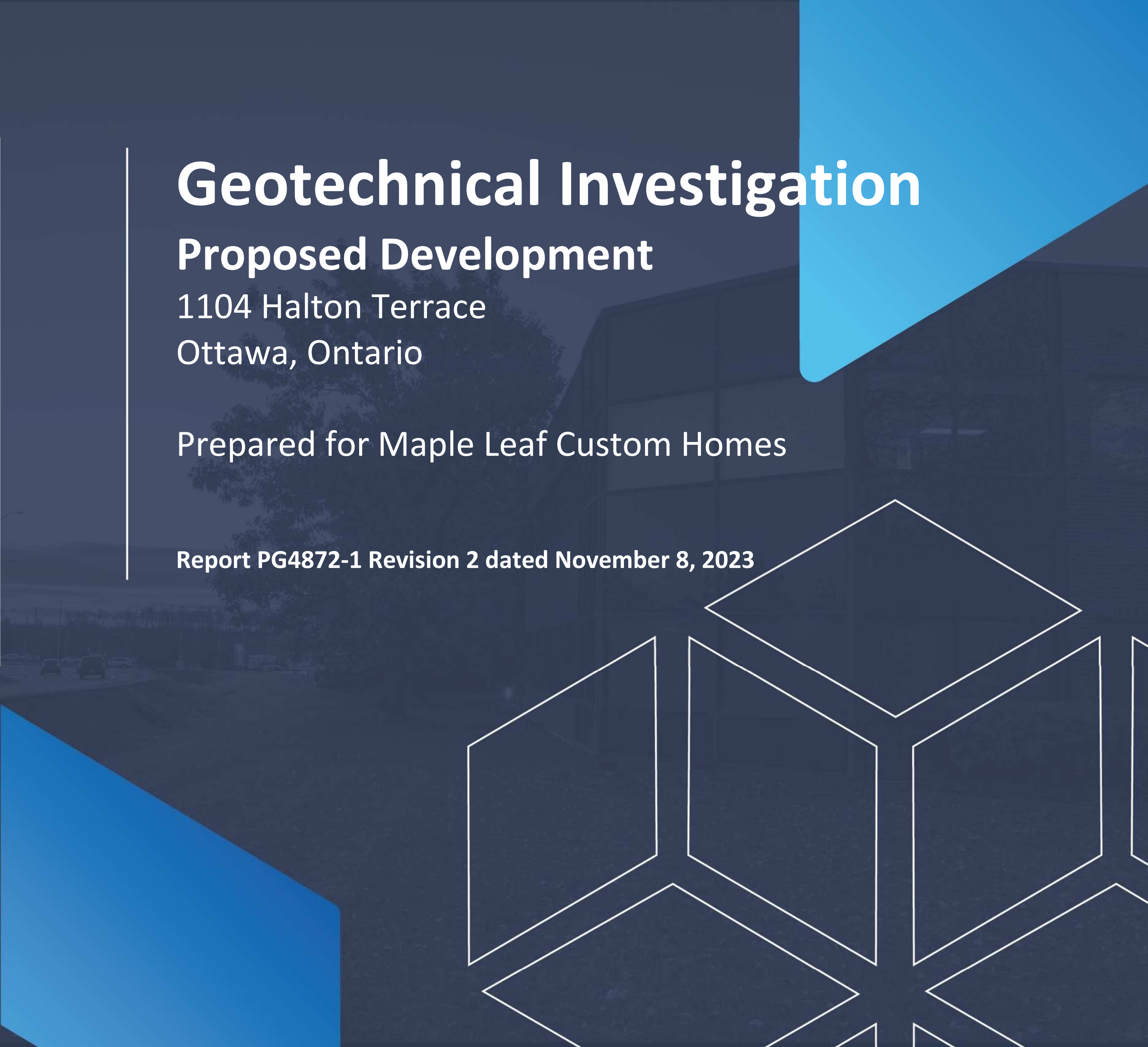
PATERSON GROUP



Geotechnical Investigation

Proposed Development

1104 Halton Terrace
Ottawa, Ontario

Prepared for Maple Leaf Custom Homes

Report PG4872-1 Revision 2 dated November 8, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Maple Leaf Custom Homes to conduct a geotechnical investigation for the proposed development to be located at 1104 Halton Terrace in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

- ☐ Obtain subsurface soil and groundwater information by means of test holes completed within the subject site.

- ☐ Provide geotechnical recommendations for 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 Development

Based on the available drawings, it is understood that the proposed development at the subject site will consist of a multi-storey building with 1 underground parking level. The proposed building will be immediately surrounded by an asphalt-paved parking lot to the west, an asphalt-paved access lane to the south, and landscaped areas to the east and north.

The proposed development is also expected to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Geotechnical investigations have included 5 test pits (TP 1-19 through TP 4-19, and TP 7) which were excavated within the boundaries of the subject site. The test pits were advanced to a maximum depth of 2.2 m below existing ground surface. The locations of the test holes are shown on Drawing PG4872-1 - Test Hole Location Plan included in Appendix 2.

The test pits were advanced using a track-mounted excavator. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The test hole procedure consisted of excavating to the required depths at the selected locations and sampling the overburden.

The Soil Profile and Test Data Sheets for these test holes are also provided in Appendix 1.

Sampling and In Situ Testing

Soil samples were recovered from the sidewalls of the test pits. All soil samples were visually inspected and classified on site. The soil samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the soil samples were recovered from the test holes are shown as G on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Ground water

Where present, the elevation at which groundwater was encountered at the completion of the test pit excavation was noted in the field.

3.2 Field Survey

The location and ground surface elevation of each test pit was surveyed by Novatech. It is understood that the ground surface elevations at the test pit locations are referenced to a geodetic datum. The test pit locations, and ground surface elevation at the each test pit location, are presented on Drawing PG4872-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

All soil samples were recovered from the subject site and visually examined in our laboratory to review the soil investigation results.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is predominantly vacant with scattered mature trees. The site is bordered by Old Carp Road to the north, Halton Terrace to the east and south, and mostly-vacant residential property to the west. The ground surface across the site generally slopes downward gradually from west to east from approximate geodetic elevation 85 m to 82 m.

4.2 Subsurface Profile

Overburden

Generally, the soil profile encountered at the test pit locations consists of topsoil extending 0.05 to 0.07 m below the existing ground surface.

Fill was encountered underlying the topsoil and extending to approximate depths of 0.6 to 1.9 m below the existing ground surface. The fill was observed to vary from a silty clay with some sand and gravel, to crushed stone with sand, clay, and organics.

In test pits TP 3-19 and TP 4-19, a 0.3 m thick topsoil layer was encountered underlying the fill, which was underlain by a 0.3 m thick layer of native clayey silt, and finally, a 0.3 to 0.65 m layer of glacial till.

The glacial till was generally observed to consist of a light brown clayey silt with some sand, gravel, cobbles, and boulders.

Practical refusal was encountered on the inferred bedrock surface at all test pit locations at depths ranging from about 0.4 m at the western end of the site, to 2.2 m near the southeast corner of the site.

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

Bedrock

Based on available geological mapping, the bedrock in this area consists of interbedded sandstone and dolomite of the March Formation.

4.3 Groundwater

Groundwater levels were noted in the completed test pits by Paterson personnel at the time of test pit excavation. Groundwater was encountered at TP 1-19 at approximate depths of 0.80 m. No groundwater was noted in the remaining test pits at the completion of excavation.

Based on field observations, experience in the local area, and colour of the recovered soil samples, the long-term groundwater level is expected to be within the bedrock. However, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed building is recommended to be founded on conventional spread footings bearing on the clean, surface sounded bedrock.

Bedrock removal will be required to complete the underground parking level and site servicing for the proposed building.

The above and other considerations are 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 the proposed building, paved areas, pipe bedding and other settlement sensitive structures.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

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

As a general guideline, peak particle velocity (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 an experienced blasting consultant.

Vibration Considerations

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

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

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

Fill Placement

Fill used for grading beneath the building area 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 building and paved 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 where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock can be designed using a factored bearing resistance value at serviceability limit states (SLS) and ultimate limit states (ULS) of **1,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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.

Footings bearing on clean, surface sounded bedrock and designed using the above-mentioned bearing resistance values will be subjected to negligible post-construction total and differential settlements.

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.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at the subject site. A higher site class, such as Class A or B, may be applicable for this site. However, the higher site class would need to be confirmed by a site-specific shear wave velocity test. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

With the removal of all topsoil and fill, containing deleterious or organic materials, the native soil or bedrock will be considered an acceptable subgrade surface on which to commence backfilling for floor slab construction.

Any soft or disturbed areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, or approved granular alternative material are recommended for backfilling below the floor slab.

All backfill material within the footprints of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the material's SPMDD.

For the proposed building, it is anticipated that the underground level will be mostly parking, and the recommended pavement structures noted in Section 5.7 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 300 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

5.6 Basement Wall

The Proposed building will have an underground parking level, there are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. 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 bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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 soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m3)
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 $0.375 \cdot a \cdot H^2/g$ where:

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

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

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

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.

Table 1 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 2 - Recommended Pavement Structure - Access Lanes

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	

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% 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 building, which will have an underground level. The system should consist of a 150 mm diameter, geotextile- wrapped, perforated and corrugated plastic pipe surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the building. The pipe should have a positive outlet, such as a gravity connection to the catch basins or running drainage ditches.

Underslab drainage is also recommended to control water infiltration. For preliminary design purposes, it is recommended that 100 or 150 mm perforated pipes be placed at approximate 6 m centers. 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.

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 Miradrain G100N, connected to the perimeter foundation drainage system.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided.

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

However, where footings are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of excavation, the minimum soil cover, listed above, is not required.

If the bedrock is cracked or fissured and within the frost cover depths, noted above, and the following measures should be implemented:

☐ Option A - Sub-excavate the weathered bedrock to sound bedrock or to the required frost cover depth. Pour footings at the lower level.

☐ Option B - Use insulation to protect footings. It is preferable to pour footings on the insulation overlying weathered bedrock. However, due to the potential undulating bedrock surface, consideration may have to be given to adopting an insulation detail that allows the footing to be poured directly on the weather bedrock.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open- cut methods (i.e. unsupported excavations).

Overburden Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Bedrock Stabilization

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.

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 temporary chainlink fencing, shotcrete, and/or rock bolts should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage of the project.

6.4 Pipe Bedding and Backfill

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

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, 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.

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

It is anticipated that groundwater infiltration into the excavations should be low and 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.

A temporary Ministry of the Environment, Conservation and Parks (MECP) 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 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.

Impacts to Neighbouring Properties

Based on available soils information, neighbouring structures are anticipated to be founded within the bedrock. Therefore, no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed development.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site 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 using 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.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

6.7 Corrosion Potential and Sulphate

The results of analytical testing indicate 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 low to moderate corrosive environment.

7.0 Recommendations

The following material testing and observation program should be performed by a geotechnical consultant and is required for the foundation design data provided herein to be applicable:

☐ Review of the bedrock stabilization and excavation 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.

☐ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

A soil 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 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 the 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 Maple Leaf Custom Homes, 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.

Deepak K Rajendran, E.I.T.

LICENSED PROFESSIONAL ENGINEER
Nov. 8, 2023
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis
Scott S. Dennis, P.Eng.

Report Distribution:

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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
Proposed Development - 1104 Halton Terrace
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG4872

REMARKS

HOLE NO. TP 1-19

BORINGS BY Hydraulic Shovel

DATE March 28, 2019

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	82.15		
Topsoil and organics 0.07	■								
FILL: Crushed stone 0.30	▨								
FILL: Brown silty clay with sand, crushed stone and gravel 1.00	▨	G	1						▽
FILL: Blast rock 1.90	▨	G	2			1	81.15		
End of Test Pit TP terminated on bedrock surface at 1.90m depth (GW infiltration at 0.8m depth)									

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1104 Halton Terrace
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY Hydraulic Shovel

DATE March 28, 2019

FILE NO. PG4872

HOLE NO. TP 2-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	82.46		
Topsoil and organics 0.05									
FILL: Crushed stone with sand, some clay 0.40									
FILL: Brown silty clay with sand, gravel, trace brick, topsoil and cobbles		G	1			1	81.46		
1.60									
Grey **SILTY CLAY,** trace sand and gravel 1.70		G	2						

End of Test Pit

TP terminated on bedrock surface at 1.70m depth

(TP dry upon completion)

Pen. Resist. / Water Content: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1104 Halton Terrace
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG4872

REMARKS

HOLE NO. TP 3-19

BORINGS BY Hydraulic Shovel

DATE March 28, 2019

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	82.68
Topsoil and organics 0.05							
FILL: Crushed stone with sand, some clay 0.60							
TOPSOIL 0.90							
Firm, reddish brown **CLAYEY SILT** with sand 1.20		G	1			1	81.68
GLACIAL TILL: Light brown clayey silt with sand,gravel, cobbles, boulders 1.50		G	2				
End of Test Pit TP terminated on bedrock surface at 1.50m depth (TP dry upon completion)							

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

○ **Water Content %**

20 40 60 80

Piezometer Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1104 Halton Terrace
Ottawa, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

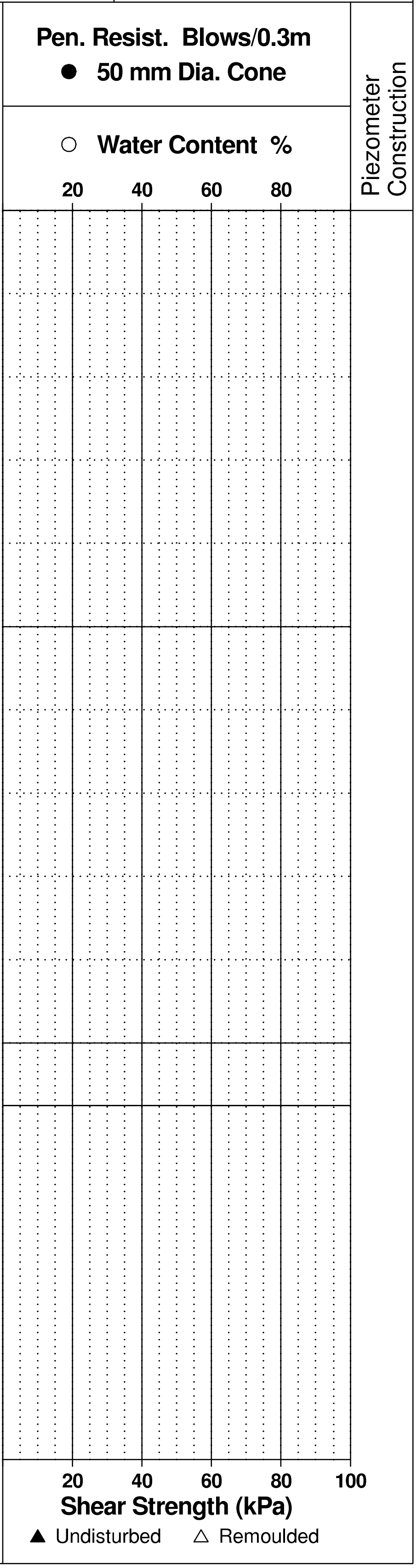
FILE NO. PG4872

REMARKS

HOLE NO. TP 4-19

BORINGS BY Hydraulic Shovel **DATE** March 28, 2019

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	83.64
Topsoil and organics — 0.05							
FILL: Crushed stone with sand, clay, trace organics — 0.60							
TOPSOIL — 0.90							
Brown **CLAYEY SILT,** trace organics — 1.20		G	1			1	82.64
Brown **CLAYEY SILT,** trace sand and topsoil — 1.50		G	2				
GLACIAL TILL: Compact to dense, light brown clayey silt with sand, gravel, cobbles, boulders — 2.15		G	3			2	81.64
End of Test Pit TP terminated on bedrock surface at 2.15m depth (TP dry upon completion)							



JOHN D. PATERSON & ASSOCIATES LTD.
Consulting Geotechnical and Environmental Engineers
28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Minto Dev. Inc.-Geotechnical Investigation
Bidgood Property-Old Carp Road @ March Road
Kanata, Ontario

DATUM Ground surface elevations provided by Webster and Simmonds Surveying Limited.

REMARKS

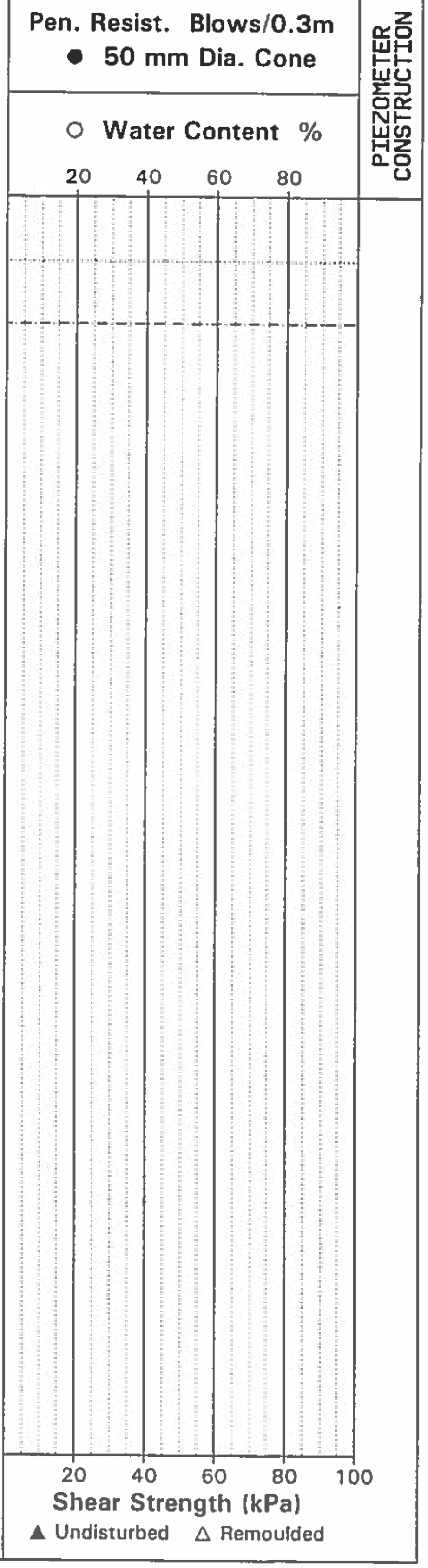
BORINGS BY Backhoe

DATE 2 NOV 00

FILE NO. G8020

HOLE NO. TP 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	84.85
Dark brown silty sand TOPSOIL, occasional gravel 0.40							
End of Test Pit TP terminated on bedrock surface @ 0.40m depth (TP dry upon completion)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))

TW - Thin wall tube or Shelby tube

PS - Piston sample

AU - Auger sample or bulk sample

WS - Wash sample

RC - Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

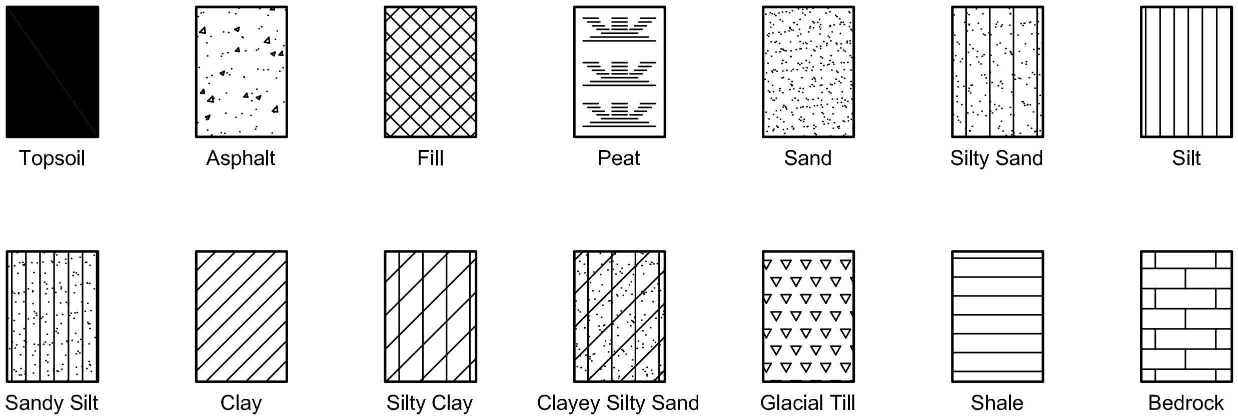
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

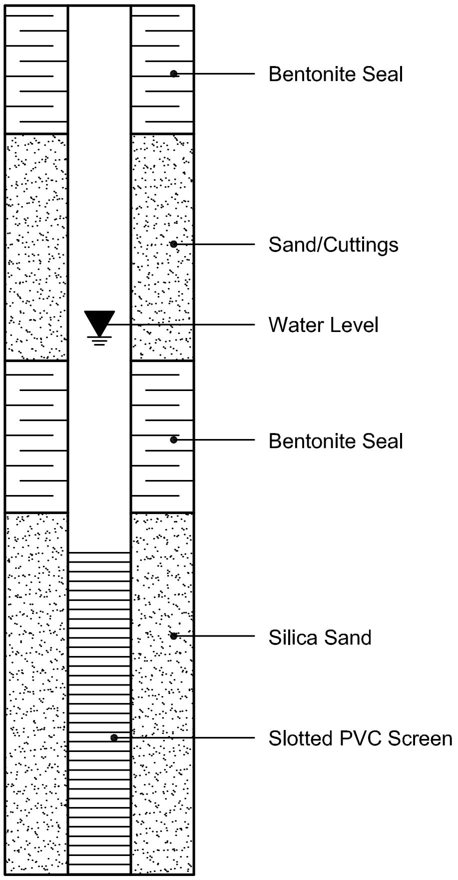
SYMBOLS AND TERMS (continued)

STRATA PLOT

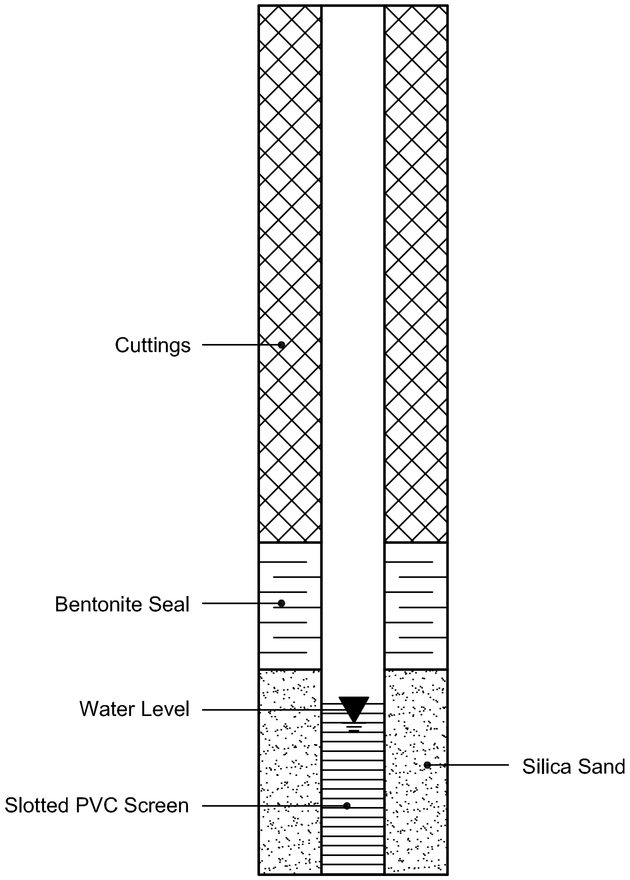


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 21949

Report Date: 21-Jul-2017

Order Date: 17-Jul-2017

Project Description: PG4194

	Client ID:	BH3-SS2	-	-	-
	Sample Date:	14-Jul-17	-	-	-
	Sample ID:	1729076-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	90.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.56	-	-	-
Resistivity	0.10 Ohm.m	56.6	-	-	-
Anions					
Chloride	5 ug/g dry	11	-	-	-
Sulphate	5 ug/g dry	9	-	-	-

APPENDIX 2

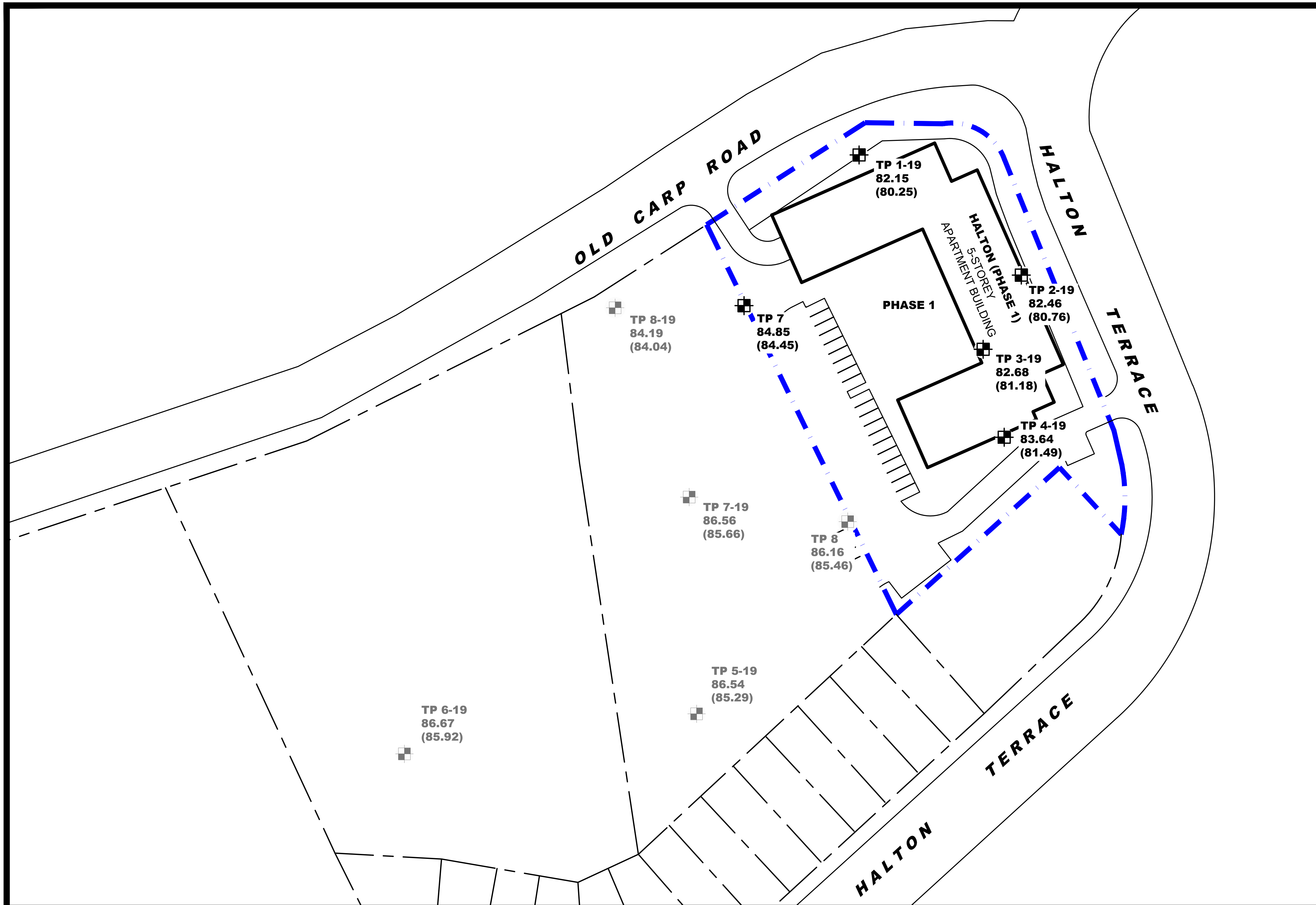
FIGURE 1 - KEY PLAN

DRAWING PG4872-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON
GROUP



LEGEND:

⊕	TEST PIT LOCATION, CURRENT INVESTIGATION
■	TEST PIT LOCATION, PATERSON GROUP REPORT G8020, NOVEMBER 2000
82.15	GROUND SURFACE ELEVATION (m)
(80.25)	ELEVATION OF PRACTICAL REFUSAL TO EXCAVATION (m)

CONCEPTUAL PLAN PROVIDED BY COLIZZA BRUNI ARCHITECTURE.

TEST PIT LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY NOVATECH ENGINEERING CONSULTANTS LIMITED.

SCALE: 1:1000

0 10 20 30 40 50m 75m

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2.	UPDATED FOR PHASED DEVELOPMENT	07/11/2023	SD
1.	UPDATED CONCEPTUAL PLAN	30/05/2023	DR

MAPLE LEAF CUSTOM HOMES

GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT - 1104 HALTON TERRACE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000
Date: 04/2019
Drawn by: MPG
Report No.: PG4872-1
Checked by: SD
Approved by: SD

PG4872-1

Revision No.: 2

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