Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

patersongroup

Geotechnical Investigation

Proposed Residential Buildings
1452-1470 Hunt Club Road and
1525-1531 Sieveright Avenue
Ottawa, Ontario

Prepared For

DCR Phoenix Group

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

September 29, 2020

Report: PG5499-1

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by DCR Phoenix Group to conduct a geotechnical investigation for the proposed residential buildings to be located at 1452-1470 Hunt Club Road and 1525-1531 Sieveright Avenue in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objectives of the geotechnical investigation were to:

- ❑ determine the subsurface soil and groundwater conditions by means of boreholes.

- ❑ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

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

2.0 Proposed Development

It is understood that the proposed development will consist of two multi-storey residential buildings with associated access lanes, at grade parking and landscaped areas. The proposed buildings will each consist of a 4-storey building of slab-on-grade construction and a 6-storey building with one basement level of underground parking at the north and south portions of the site, respectively. It is anticipated that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on August 25 and August 26, 2020. At that time, seven (7) boreholes were advanced to a maximum depth of 7.6 m below the existing ground surface. The test hole locations were determined by Paterson personnel and distributed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations are presented on Drawing PG5499-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low clearance auger drill rig operated by a two-person crew. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department.

Sampling and In Situ Testing

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

Standard Penetration Testing (SPT) was conducted 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.

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

The 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 boreholes BH 2-20, BH 4-20, BH 6-20 and BH 7-20 during the field investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Flexible polyethylene standpipes were installed within all remaining boreholes.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration existing site features and underground utilities. The test hole locations and ground surface elevations at each test hole location were surveyed by Paterson personnel. The ground surface elevations at the borehole locations were referenced to a geodetic datum. The test hole locations are presented on Drawing PG5499-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an automobile dealership at 1452-1470 Hunt Club Road and commercial storage buildings at 1525-1531 Sieveright Avenue. Several one-storey structures are present throughout the west and north portion of the subject site. The north and west portions of the subject site are surfaced with asphalt while the remaining portions of the site are grass-covered.

The subject site is relatively flat and at grade with the surrounding properties. The subject site is bordered to the north by Hunt Club Road, to the west by one-storey commercial buildings, to the south by Sieveright Avenue and to the east by existing residential dwellings along Issam Private.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of topsoil, asphalt or fill underlain by a layer of sand or silty sand. Fill was encountered overlying the native sand layer and extending to depths between 0.5 to 2.2 m below existing ground surface at all boreholes with the exception of BH 5-20. The native sand deposit was observed to be underlain by a deposit of stiff grey silty clay extending to depths between 3.0 and 6.7 m below the existing ground surface. The silty clay deposit was observed to be further underlain by layers of sandy silty and silty sand throughout the north and central portion of the site.

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

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of shale of the Carlsbad formation with a drift thicknesses between 15 and 25 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells and piezometers on September 2 and September 8, 2020, respectively. The observed groundwater levels are summarized in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-20	92.87	Damaged	n/a	September 8, 2020
BH 2-20*	93.36	4.61	88.75	September 2, 2020
BH 3-20	93.31	Damaged	n/a	September 8, 2020
BH 4-20*	92.66	2.41	90.25	September 2, 2020
BH 5-20	93.40	0.95	92.45	September 8, 2020
BH 6-20*	92.91	4.35	88.56	September 2, 2020
BH 7-20*	93.01	5.48	87.53	September 2, 2020

Note: The ground surface elevations at the borehole locations are referenced to a geodetic datum.
- “*” indicates monitoring well installed within borehole.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 2.5 to 3.5 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

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. It is expected that the proposed buildings will be founded on conventional spread footings placed on a stiff grey silty clay, compact brown silty sand and/or engineered fill bearing surface.

Where the existing fill is encountered at design underside of footing elevation, it is anticipated that the footings will be extended to an undisturbed bearing surface or placed on an approved engineered fill placed on an undisturbed bearing surface.

Due to the presence of a silty clay deposit throughout the subject site, a permissible grade raise restriction will be required for grading around the proposed buildings founded over the existing silty clay deposit.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. It is anticipated that the existing fill, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprint, outside of lateral support zones for the footings, and below the proposed parking area and access lane. However, it is recommended that the existing fill layer where encountered within settlement sensitive areas **be proof-rolled several times under dry conditions and above freezing temperatures and approved by the geotechnical consultant at the time of construction**. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

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

Fill Placement

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

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

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 8 m wide, placed on an undisturbed, stiff grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **240 kPa**.

Footings placed on an undisturbed, compact silty sand bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **240 kPa**.

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

Where existing fill is encountered directly below the underside of footing (USF), the footings may be required to be lowered to an undisturbed, native bearing surface.

Footings placed over an approved engineered fill bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **240 kPa**.

Footings designed using the bearing resistance value at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance values at ULS.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support. Adequate lateral support is provided to a silty sand, sandy silt, silty clay or engineered fill bearing medium above the groundwater table when a plane extending down and out from the bottom edge of the footing, at a minimum of 1.5H:1V passes through in situ soil of the same or high bearing medium soil.

Permissible Grade Raise Restrictions

Consideration must also be given to potential settlements which could occur due to the presence of the silty clay deposit encountered throughout the southern half of the subject site and the combined loads from the proposed footings, any groundwater lowering effects, and grade raise fill. Based on the undrained shear strength testing results and experience with the local silty clay deposit, a permissible grade raise restriction for the subject site of **2.0 m** can be used for design purposes.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The subject site can be taken as seismic site response **Class D** as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012 for foundations considered at this site. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill, such as those containing organic materials, within the footprint of the proposed buildings, the native soil or engineered fill surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

It is anticipated that the existing fill, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprint, outside of lateral support zones for the footings. However, it is recommended that the existing fill layer **be proof-rolled several times under dry conditions and above freezing temperatures and approved by Paterson personnel at the time of construction**. The fill is recommended to be proof-rolled with a suitably sized vibratory smooth-drum or sheepsfoot roller for sandy or clayey fill, respectively. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II with a maximum particle size of 50 mm, compacted to a minimum of 98% of the material's SPMDD are recommended for backfilling below the floor slab.

It is expected that the basement area for the proposed 6-storey building located at the south portion of the site will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level are proposed where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone.

The upper 200 mm of sub-slab fill for the 4-storey slab-on-grade structure within the north portion of the subject site is recommended to consist of OPSS Granular A crushed stone. The sub-slab fill should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the material's SPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab where a basement level is provided. The spacing of the sub-slab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, if any. 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 subject structure. However, the conditions can be well-represented by assuming the retained material has 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 material 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 (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

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

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

Seismic 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 = $(1.45-a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained material (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 $P_o = 0.5\ K_o\ \gamma\ H^2$, 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 Pavement Structure

Car only parking areas and access lanes are anticipated at the subject site. The proposed pavement structures are presented in Tables 2 and 3.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 3 - Recommended Pavement Structure - Access Lanes and Ramp

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 I or Type II material.

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 compaction equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. The sub-drain inverts should be approximately 300 mm below subgrade level and run longitudinal along the curblines. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Perimeter Foundation Drainage System

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by a minimum of 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

It is anticipated that underfloor drainage will be required to control water infiltration below the proposed basement level. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. It is recommended that 150 mm diameter sleeves, spaced at 3 m centres be cast in the footing or at the foundation wall/footing interface to direct flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

A waterproofing system should also be provided for any elevator shafts and sump pump pits (pit bottoms and walls) located within the lowest basement level.

Foundation Backfill

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 composite drainage system, such as Delta Drain 6000 or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or equivalent) should be provided in this regard.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for exterior unheated footings, not thermally connected to a heated space, such as exterior columns and/or wing walls.

It has been our experience that insufficient soil cover is typically provided to footings located in areas where minimal soil cover is available, such as entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided.

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 excavations to be undertaken by open-cut methods (i.e. unsupported excavations).

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A crushed stone should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's SPMDD.

Generally, it should be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

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

To reduce long term lowering of the groundwater level at this site, clay seals should be provided in the service trenches where a clay subgrade is encountered. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations low to moderate and 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.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which encounters the building's perimeter groundwater infiltration control system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e. less than 25,000 L/day with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

It is understood that one level of underground parking is planned for the proposed 6-storey building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures.

It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

The subsurface soil conditions contain 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 installation of straw, propane heaters and tarpaulins or other suitable means.

The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be constructed to avoid the introduction of frozen materials, snow or ice into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving during construction. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

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

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

8.0 Statement of Limitations

The recommendations 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 hole 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 DCR Phoenix Group, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Sept. 28-2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution

- ❏ DCR Phoenix Group (PDF copy)
- ❏ Paterson Group (1 copy)

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
1452-1470 Hunt Club Road & 1525-1531 Sieveright Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

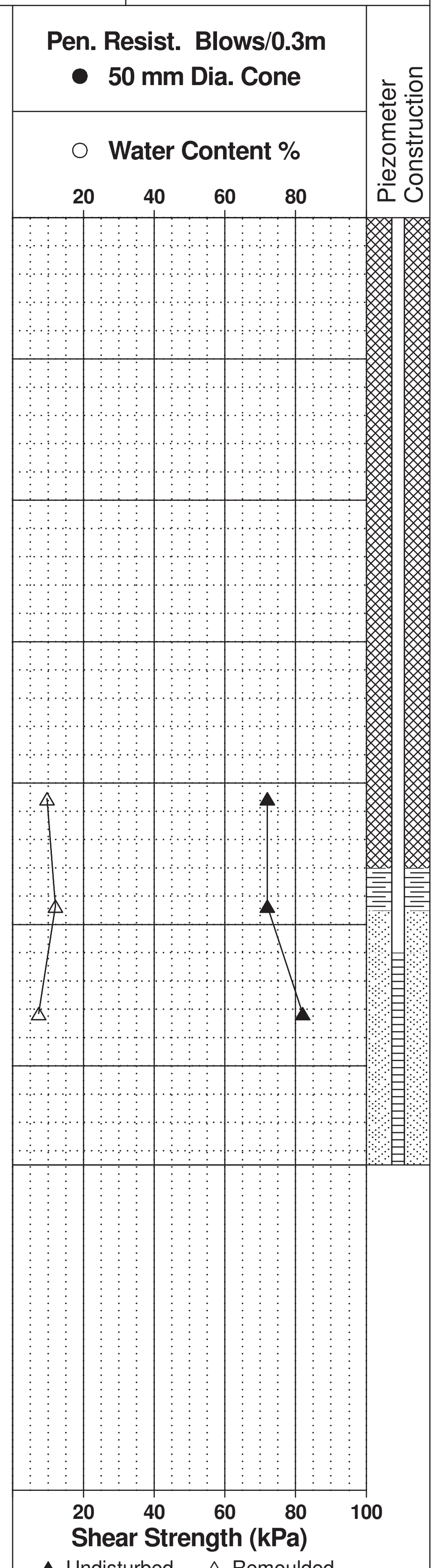
BORINGS BY CME-55 Low Clearance Drill

DATE August 25, 2020

FILE NO. PG5499

HOLE NO. BH 1-20

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.87
Asphaltic concrete	0.08						
FILL: Brown silty clay	0.74	AU	1				
Compact, brown **SILTY SAND**	2.29	SS	2	8	21	1	91.87
		SS	3	38	18	2	90.87
Loose, brown coarse to medium **SAND**	3.05	SS	4	54	4	3	89.87
Stiff, grey **SILTY CLAY**	5.94	SS	5	100	W		
		SS	6	100	P	4	88.87
		SS	7	100	P	5	87.87
		SS	8	100	P	6	86.87
Compact, grey **SILTY SAND**	6.70	SS	6	42	19		
End of Borehole							
(Piezometer damaged - Sept. 8, 2020)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1452-1470 Hunt Club Road & 1525-1531 Sieveright Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 25, 2020

FILE NO. PG5499

HOLE NO. BH 2-20

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.36
FILL: Brown sand with crushed stone, trace asphalt	0.51	AU	1				
FILL: Brown silty clay with sand and gravel	1.52	SS	2	79	11	1	92.36
Loose, brown coarse to medium **SAND**	2.06	SS	3	58	9	2	91.36
Stiff, grey **SILTY CLAY,** trace sand seams	3.66	SS	4	75	3	3	90.36
		SS	5	100	5		
Compact, grey **SANDY SILT** - some clay by 4.6m depth	6.10	SS	6	54	23	4	89.36
		SS	7	62	26	5	88.36
		SS	8	75	15	6	87.36
Compact, grey **SILTY SAND**	6.70	SS	9	46	24		
End of Borehole							

(GWL @ 4.61m - Sept. 2, 2020)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1452-1470 Hunt Club Road & 1525-1531 Sieveright Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 26, 2020

FILE NO. PG5499

HOLE NO. BH 3-20

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.31
Asphaltic concrete	0.05						
FILL: Brown silty sand with crushed stone	0.51	AU	1				
Loose, brown medium to coarse **SAND** with gravel	2.29	SS	2	42	4	1	92.31
		SS	3	54	7	2	91.31
Very stiff, grey **SILTY CLAY,** some gravel	3.05	SS	4	33	29	3	90.31
Dense to compact, grey **SANDY SILT**	4.57	SS	5	46	33	4	89.31
		SS	6	50	18		
Compact, grey **SILTY SAND,** some gravel	5.20	SS	7	33	28	5	88.31
Compact, grey **SANDY SILT** with clay	6.70	SS	8	29	12	6	87.31
		SS	9	33	21		

End of Borehole

(Piezometer damaged - Sept. 8, 2020)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1452-1470 Hunt Club Road & 1525-1531 Sieveright Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

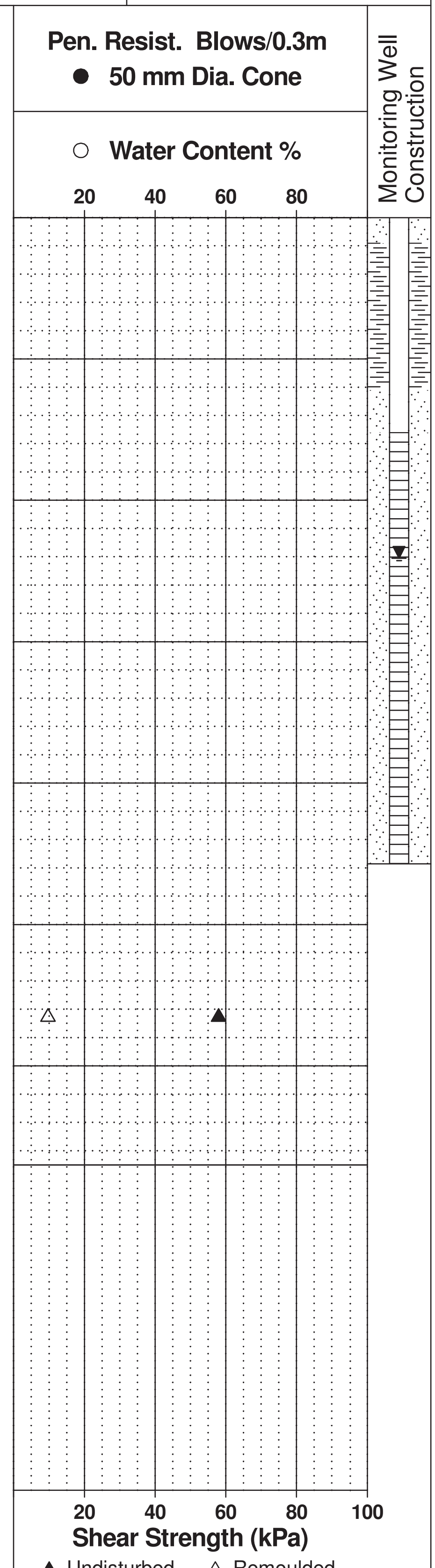
BORINGS BY CME-55 Low Clearance Drill

DATE August 25, 2020

FILE NO. PG5499

HOLE NO. BH 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.66
Asphaltic concrete 0.08							
FILL: Brown silty sand with crushed stone 0.60		AU	1				
Compact, brown medium to coarse **SAND**		SS	2	71	12	1	91.66
		SS	3	79	12	2	90.66
		SS	4	62	16		
		SS	5	67	16	3	89.66
4.11		SS	6	29	9	4	88.66
Stiff, grey **SILTY CLAY**		SS	7	83	W	5	87.66
						6	86.66
6.70		SS	8	100	2		
End of Borehole							
(GWL @ 2.41m - Sept. 2, 2020)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1452-1470 Hunt Club Road & 1525-1531 Sieveright Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 25, 2020

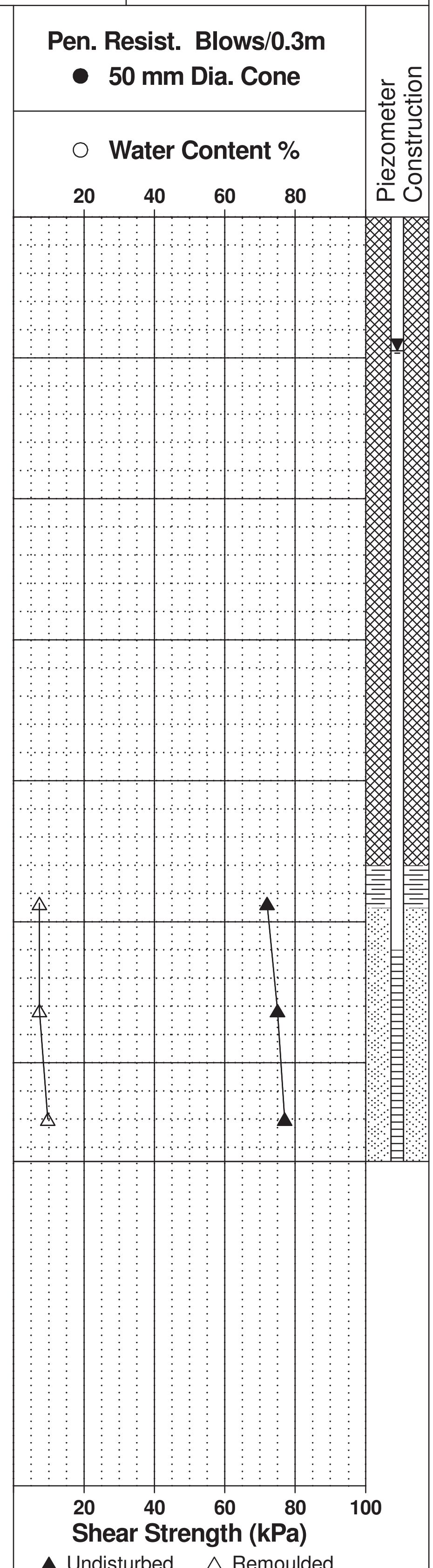
FILE NO. PG5499

HOLE NO. BH 5-20

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.40
TOPSOIL	0.20	AU	1				
Loose to compact, brown **SILTY SAND**		SS	2	75	5	1	92.40
		SS	3	79	16	2	91.40
	3.05	SS	4	71	27	3	90.40
Stiff, grey **SILTY CLAY,** trace sand		SS	5	88	2	4	89.40
		SS	6	100	W	5	88.40
		SS	7	100	P	6	87.40
		SS	8	100	P		
	6.70	SS	9	100	P		

End of Borehole

(GWL @ 0.95m - Sept. 8, 2020)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1452-1470 Hunt Club Road & 1525-1531 Sieveright Ave.
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5499

REMARKS

HOLE NO. BH 6-20

BORINGS BY CME-55 Low Clearance Drill

DATE August 25, 2020

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	92.91
FILL: Brown silty sand with crushed stone, trace asphalt (to 0.66)	AU	1				
FILL: Brown silty sand, trace gravel and clay (to 1.52)	SS	2	54	14	1	91.91
FILL: Dark brown silty clay with sand, gravel and organics (to 2.29)	SS	3	46	10	2	90.91
Dense to compact, grey **SANDY SILT,** trace clay (to 5.33)	SS	4	58	32	3	89.91
	SS	5	75	41	4	88.91
	SS	6	58	24		
	SS	7	71	23	5	87.91
Compact, grey **SAND** - with clay by 6.1m depth (to 7.62)	SS	8	38	26	6	86.91
	SS	9	33	15	7	85.91
End of Borehole (GWL @ 4.35m - Sept. 2, 2020)						

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

○ Water Content %
20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1452-1470 Hunt Club Road & 1525-1531 Sieveright Ave.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** August 25, 2020

FILE NO. PG5499

HOLE NO. BH 7-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.01
FILL: Brown silty sand with crushed stone, trace organics (to 0.56)		AU	1				
FILL: Brown silty clay, some sand (to 1.52)		SS	2	38	8	1	92.01
Stiff, brown **SILTY CLAY,** trace sand (to 3.05)		SS	3	21	7	2	91.01
		SS	4	38	7	3	90.01
Compact, brown **SILTY SAND** (to 4.57)		SS	5	29	13		
		SS	6	46	26	4	89.01
Compact, grey **SANDY SILT** (to 5.13)		SS	7	58	26	5	88.01
Compact, grey medium **SAND** (to 6.70)		SS	8	38	20	6	87.01
		SS	9	42	18		
End of Borehole							
(GWL @ 5.48m - Sept. 2, 2020)							

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

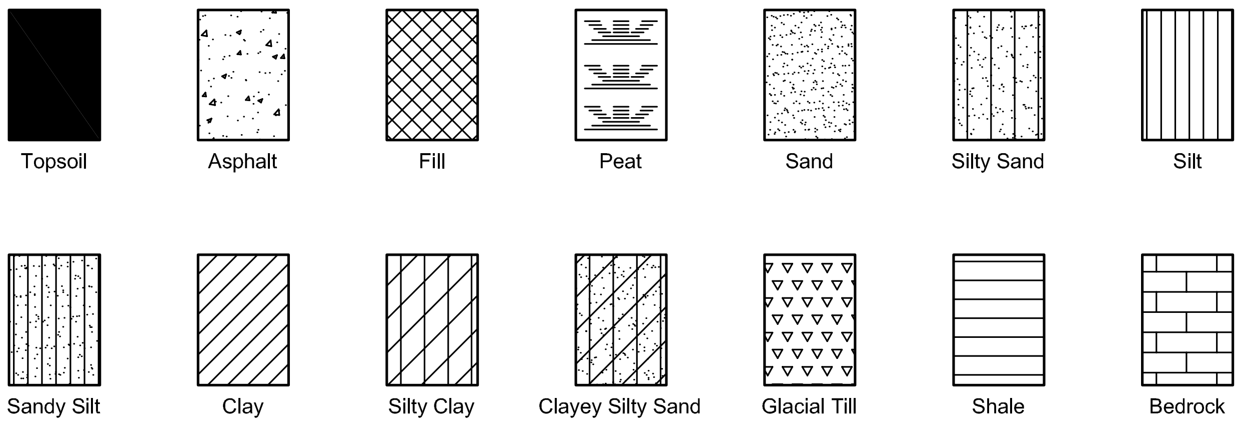
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

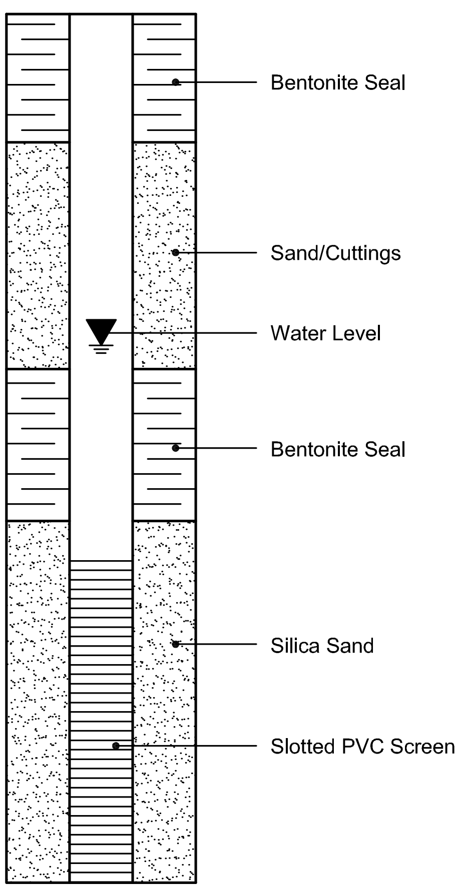
SYMBOLS AND TERMS (continued)

STRATA PLOT

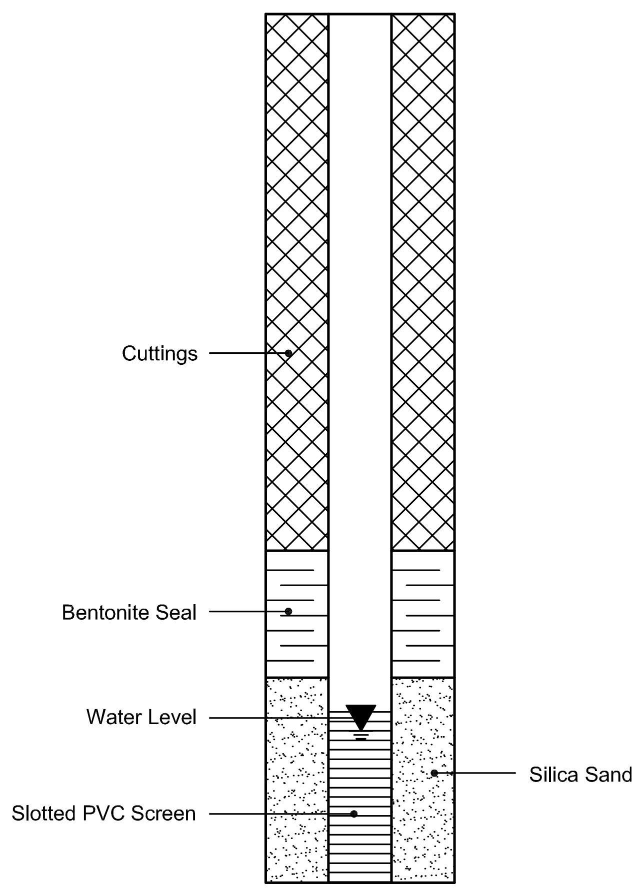


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 30698

Report Date: 02-Sep-2020

Order Date: 27-Aug-2020

Project Description: PE5015

	Client ID:	BH1-SS4	-	-	-
	Sample Date:	26-Aug-20 13:00	-	-	-
	Sample ID:	2035554-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	88.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.56	-	-	-
Resistivity	0.10 Ohm.m	19.7	-	-	-
Anions					
Chloride	5 ug/g dry	202	-	-	-
Sulphate	5 ug/g dry	147	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5499-1 - TEST HOLE LOCATION PLAN

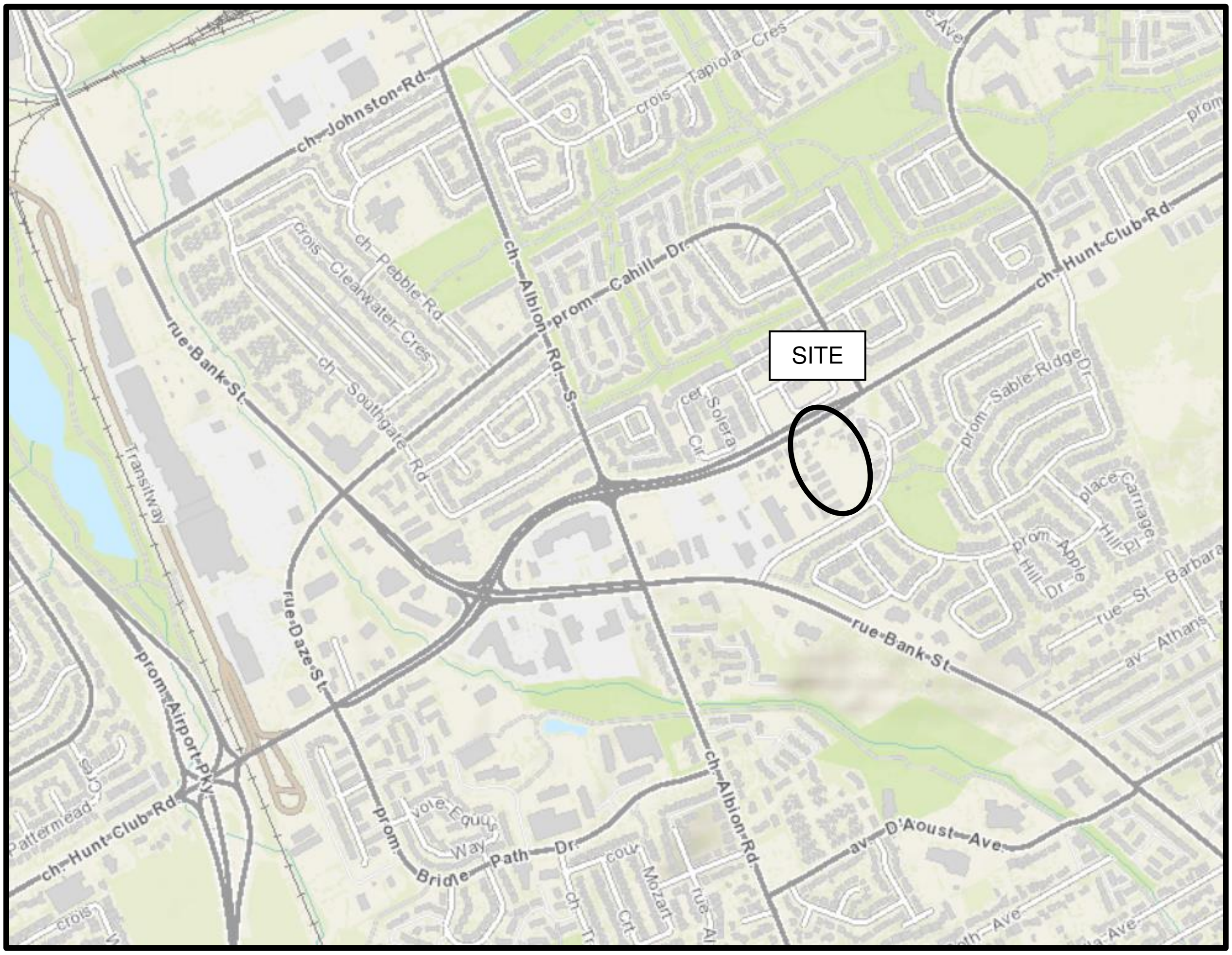
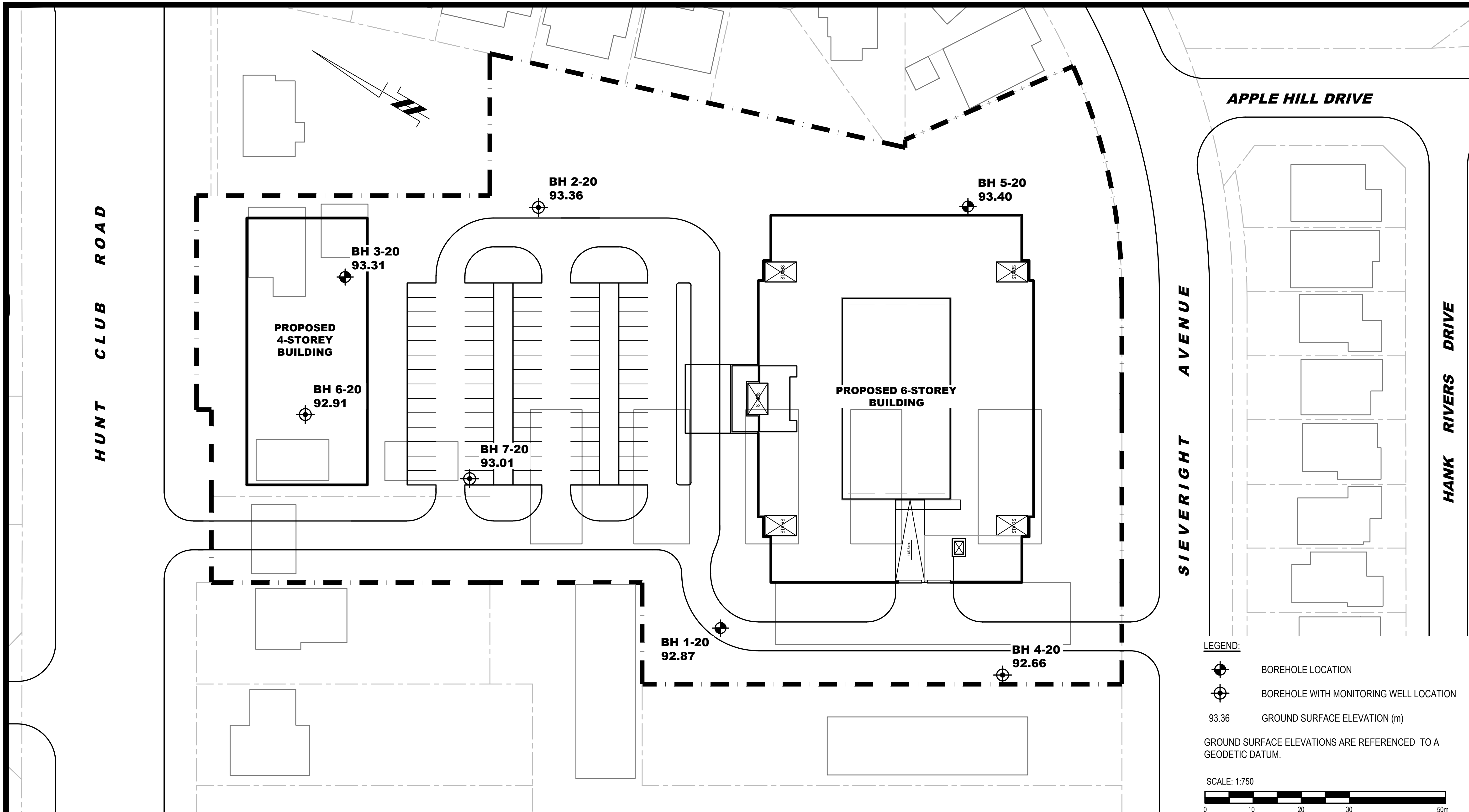


FIGURE 1

KEY PLAN

patersongroup



patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
0			

DCR PHOENIX GROUP

GEOTECHNICAL INVESTIGATION
1452-1470 HUNT CLUB ROAD AND 1525-1531 SIEVERIGHT AVENUE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 09/2020
Drawn by: MPG	Report No.: PG5499-1
Checked by: DP	Dwg. No.: PG5499-1
Approved by: DJG	Revision No.:

p:\autocad drawings\geotechnical\pg54xx\pg5499\pg5499-1 thlp.dwg

11x17