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Geotechnical Investigation

Proposed Multi-Storey Building
150 Laurier Avenue West
Ottawa, Ontario

Prepared For

Jadco Group

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Report PG5195-1

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

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Jadco Group to conduct a geotechnical investigation for the proposed multi-storey building, which is to be located at 150 Laurier Avenue West in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ❏ Determine the subsoil, groundwater, and bedrock conditions at this site by means of boreholes.

- ❏ Based on the results of the boreholes, provide geotechnical recommendations pertaining to the design of the proposed development, including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. A report addressing environmental issues for the subject site was prepared under a separate cover.

2.0 Proposed Development

Specific details regarding the proposed development were not known at the time of preparing this report. However, it is anticipated that the proposed development would consist of a high-rise building with up to 5 levels of underground parking. The footprint of the proposed parking garage is anticipated to occupy the entire site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out between January 9 to 15, 2020. At that time, 4 boreholes were advanced to a maximum depth of 21.2 m below the existing grade. The borehole locations were distributed in a manner to provide general coverage of the proposed development. The approximate locations of the boreholes are shown on Drawing PG5195-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill rig and geoprobe drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

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

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

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

The subsurface conditions observed in the boreholes 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.

Diamond drilling was carried out at BH 1, BH 2 and BH 4, to determine the nature of the bedrock and to assess its quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the quality of the bedrock.

Groundwater

A 32 or 51 mm diameter PVC groundwater monitoring well was installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations and elevations were surveyed in the field by Paterson. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located on the west side of Elgin Street in front of 150 Elgin Street. A Geodetic elevation of 70.16 m was previously provided for this TBM by Stantec Geomatics during a previous investigation for an adjacent site. This geodetic elevation was transferred to the fire hydrant in front of 160 Laurier Avenue West, which was used to survey the borehole locations.

The locations of the boreholes, TBM, and the ground surface elevation at the boreholes, are presented on Drawing PG5195-1 - 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.

4.0 Observations

4.1 Surface Conditions

The subject site currently consists of a 5 storey office building with asphalt covered parking surfaces on either side. The site is flat and at grade with Laurier Avenue West.

The site is bordered to the north by Laurier Avenue West, to the east by an old stone church building, to the south by a high rise office building and to the west by two 2-storey commercial buildings followed by a hi-rise office building.

4.2 Subsurface Profile

Overburden

Generally, the soil conditions encountered at the boreholes consisted of an asphalt pavement structure at ground surface overlying a fill layer, consisting of brown silty sand with trace gravel and/or clay. A stiff silty clay deposit was encountered below the fill layer extending to depths ranging from 12.2 to 14.6 m below ground surface. Glacial till, consisting of silty clay to silty sand with gravel, cobbles and boulders, was encountered below the silty clay deposit. Practical refusal to augering was encountered at depths between 14.5 and 17.6 m. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at the boreholes.

Bedrock

A very poor (RQD values ranging between to 25%) to an excellent (RQD values ranging between 90 to 100%) quality black shale bedrock was encountered at BH 1, BH 2 and BH 4 at depths ranging between 14.5 and 17.6 m.

Based on available geological mapping, the bedrock in this area consists of shale of the Billings formation and dark grey limestone.

4.3 Groundwater

Groundwater levels (GWLs) were measured in the monitoring wells installed at the borehole locations and the results are summarized in Table 1. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction and in the long term.

Table 1
Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	69.49	13.60	55.89	January 22, 2020
BH 2	69.07	7.45	61.62	January 22, 2020
BH 3	69.63	8.76	60.87	January 22, 2020
BH 4	68.82	7.94	60.88	January 22, 2020

Note: The ground surface elevation at each borehole was referenced to a temporary benchmark (TBM), which consists of the top spindle of the fire hydrant located on the west side of Elgin Street in front of 150 Elgin Street . A Geodetic elevation of 70.16 m was previously provided for this TBM.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed hi-rise development. It is expected that a raft foundation founded on the weathered bedrock will be used to support the building. The raft slab will also form part of the water suppression system to manage and minimize groundwater infiltration for the purpose of preventing long term dewatering of the surrounding areas.

It is expected that temporary shoring will be required for the excavation of the underground parking levels. It is further expected that the influence of the excavation and the selection of the shoring system should account for the effects to adjacent structures including dewatering control measures.

Bedrock excavation is expected for the construction of the lower underground levels of the proposed building.

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

5.2 Site Grading and Preparation

Stripping Depth

Based on the proposed depth of the building excavation, all the overburden will be removed from the footprint of the proposed building.

Bedrock Removal

It is expected that line-drilling in conjunction with controlled blasting and mechanical bedrock removal (hoe-ramming and rock grinding) will be required to remove bedrock.

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

As a general guideline, peak particle velocities (measured at the structures) should not exceed 50 mm/s during the blasting program to reduce the risks of damage to the existing structures. Typically, it's suggested that the peak particle velocity be maintained at 25 mm/s at the property line when possible.

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

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system. If the shoring system will include drilled piles into the bedrock extending to 1 to 1.5 m below the proposed excavation bottom, the 1 m horizontal ledge can be omitted.

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 equipments could be the source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of these equipments. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause of 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 several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines. 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 building.

5.3 Foundation Design

To minimize dewatering of adjacent properties, a raft slab foundation is recommended to be constructed over the bedrock surface in conjunction with a partial tanking system.

Bearing Resistance Values - Raft Foundation

It is expected that the proposed raft foundation will extend several meters below the shale bedrock surface to accommodate the five levels of underground parking.

A factored bearing resistance value at ultimate limit states (ULS) (contact pressure) of **2,000 kPa** could be used. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **50 MPa/m** for a contact pressure of **2,000 kPa**. The design of the raft foundation should consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the above assumptions for the raft foundation, the proposed structure could be designed with the above parameters and a total and differential settlement of 10 and 5 mm, respectively.

Pressure Relief Chamber

To prevent the long term dewatering of adjacent structures surrounding the site, at the founding level, a pressure relief chamber should be installed along with collection pipes within excavated or grinded trenches in the bedrock. The collection pipe trenching should extend along the proposed building perimeter and lead to the pressure relief chamber. It is suggested that the pressure relief chamber be incorporated in the lowest section of the P5 level within a utility room in close proximity to the proposed sump pits. Figure 2 - Pressure Relief Chamber in Appendix 2 provides an example of the required pressure relief chamber. Once the pressure relief chamber and associated piping is installed, the proposed raft slab can be constructed. The purpose of the pressure relief chamber will be as follows:

- ❏ manage any water infiltration along the bedrock surface during the excavation program.

- manage the water infiltration during the pouring of the raft slab to prevent water flow in the fresh concrete.
- manage water infiltration below the raft slab until sufficient load is applied to resist any potential hydrostatic uplift.
- regulate the discharge valve to control water infiltration once the raft slab is in place and over the long term to manage the hydrostatic pressure to permit any repairs associated with any water infiltration.
- Once the building is completed, the pressure relief valve will be fully closed to prevent any further dewatering.

Hydrostatic Pressure

With the fully closed valve within the pressure relief chamber and a perfectly watertight foundation, it is expected that a maximum hydrostatic pressure of **100 kPa** will be developed over the long term and should be incorporated in the design of the raft foundation and the foundation wall. Realistically, achieving a fully watertight foundation is not always possible due to minor water infiltration and, therefore, a realistic long term hydrostatic pressure will be closer to 60 to 70 kPa.

5.4 Design for Earthquakes

Since the building will be founded on bedrock, for design purposes, the site class for seismic site response is a **Class A** for the raft foundation. This higher seismic site classification must be confirmed by site specific shear wave velocity testing, which can be completed at a later date. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC 2012; Table 4.1.8.4.A) for a full discussion of the earthquake design requirements.

5.5 Basement Slab

It is expected that the lower basement slab will be placed over the raft foundation on a layer of clear stone or free draining granular backfill which will promote drainage to the sump pit. It is expected that the basement area will be mostly parking and that a concrete slab will be used. A rigid pavement structure is presented in Subsection 5.8. The thickness of the granular subfloor layer will be dependent on the proposed elevation of the P5 level floor slab. It is also expected that a sump pit will be incorporated in the design of the raft slab to drain any water which enters the granular layer via a breach in the raft slab or foundation wall waterproofing system.

The final basement floor slab and associated underfloor granular material should only be placed once the pressure relief chamber valve has been fully closed and no significant water infiltration is observed after hydrostatic pressure is applied.

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 soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m^3.

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

Static Earth Pressures

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

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

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 a_c \gamma H^2/g$ where:

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

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

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

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

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

5.7 Rock Anchor Design

If required in the structural design, rock anchors can be designed using the following geotechnical parameters:

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

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

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

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load with less deflection than a passive anchor.

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

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

Grout to Rock Bond

Generally, the unconfined compressive strength of shale ranges between about 60 and 90 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.2 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. A **Rock Mass Rating (RMR) of 44** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.183 and 0.00009**, respectively. For design purposes, we assumed that all rock anchors will be placed at least 1.2 m apart to reduce group anchor effects.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2.

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.2 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Fair quality Shale - Hoek and Brown parameters	44 m=0.183 and s=0.00009
Unconfined compressive strength - Shale	40 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm diameter hole are provided in Table 3. A detailed analysis for the anchorage system could be provided once the details of the loading for the proposed tower are known. It should be noted that the factored tensile resistance values given in Table 3 are based on a single anchor with no group influence effects.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor				
Diameter of Drill Hole (mm)	**Anchor Lengths (m)**			**Factored Tensile Resistance (KN)**
	Bonded Length	**Unbonded Length**	**Total Length**	
75	3.2	1.1	4.3	250
	3.8	2.2	5.8	450
	4.1	2.6	6.7	600
	5	2.5	7.5	750

Other considerations

The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request.

5.8 Pavement Structure

The proposed parking level slabs will be considered a rigid pavement structure. The following rigid pavement structure is recommended to support car parking only. A flexible asphaltic concrete pavement design is not expected for the subject site.

Table 4 - Recommended Rigid Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
125	Wear Course - Concrete slab
200	BASE - 20 mm clear stone
	SUBGRADE - Concrete transfer slab

6.0 Design and Construction precautions

6.1 Foundation Drainage and Backfill

Foundation Waterproofing and Drainage

It is anticipated that the building footprint will occupy the majority of the subject site. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be blind poured against a drainage system placed against the temporary shoring system and the vertical bedrock excavation face.

Since the lower basement levels will be located below the expected groundwater level, a waterproofing membrane (Paraseal LG) should be installed over the vertical surfaces from the bottom of the excavation (bedrock vertical face) up to 1 m above the long term groundwater level (to approximately 6.5 m below the existing finished grade). The bedrock vertical surface, where encounterered, will require bedrock grinding to create a smoother bedrock surface and lessen the potential of bedrock over breakage. By waterproofing the vertical excavation sides, it will be possible to lessen the groundwater volumes entering the excavation. A composite drainage system should be incorporated against the waterproofing membrane to act as a protection layer and to drain any water breaching the waterproofing membrane system.

For preliminary design purposes, the composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) should extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Raft Slab Construction Joints

It is expected that the raft slab will be poured in sections. The construction joint at each pour should incorporate a rubber water stop along with a chemical grout (Xypex or equivalent) applied to the entire vertical joint of the raft slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration below the basement floor slab. For design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at 6 to 9 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation program for a better assessment.

Adverse Effects of Dewatering on Adjacent Properties

Since the proposed development will be founded below the long term groundwater level, a waterproofing membrane system has been recommended to lessen the effects of water infiltration. Any long term dewatering of the site will be minimal and should have no adverse effect to the surrounding buildings or structures. The short term dewatering during the excavation program will be managed by the excavation contractor and an attempt will be made to grout or patch any areas with noticeable water infiltration.

Foundation Backfill

In areas where a doubled sided pour for the exterior foundation wall is being considered, backfill against the exterior side of the foundation wall 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 Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Pressure Relief Chamber

The purpose of the pressure relief chamber will be to control the groundwater infiltration and hydrostatic pressure created by fully or partially tanking the basement level. To avoid uplift on the raft foundation slab prior to having sufficient loading to resist uplift, it is recommended that the water infiltration be pumped via the pressure relief chamber during the construction program.

During the construction program, the valve of the pressure relief chamber can be gradually closed as the loading is applied to resist hydrostatic pressure. Once sufficient load is available to resist the full hydrostatic pressure, the valve of the pressure relief chamber can be adjusted and closed to minimize water infiltration volumes.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided in this regard. Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp, may be required to insulate against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Temporary Shoring

Temporary shoring will be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services.

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

The temporary system will consist of a combination of soldier pile and lagging system for open areas such as roadways and parking lots and interlocking steel sheet piling for areas adjacent or in close proximity to existing structures. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.

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

Table 5 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

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

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

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

Underpinning and Review of Existing Structures

The founding conditions of the existing adjacent structures should be reviewed by the geotechnical consultant prior to commencing excavation. Depending on the founding conditions, as well as, the proximity and depth of the proposed excavation, an underpinning program may be required for the existing building(s). It is further recommended that the condition of the existing foundations be reviewed by a structural engineer.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. 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 (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

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

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of the excavations. 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) Category 3 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, 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 breaching the waterproofing system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (less than 5,000 L/day) with higher volumes during peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

The installation of a temporary shoring system will disturb the soil immediately behind the shoring system which may cause some movement of adjacent structures. To lessen these effects, consideration should be given to adding brackets to the shoring system that could support the adjacent structures or an alternative support system for the adjacent structures. Furthermore, until the waterproofing is completed, temporary dewatering will also cause typical minor differential settlements to unsupported structures.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site mostly consist of frost susceptible materials. In the presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

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

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation services program including the following aspects be performed by the geotechnical consultant.

- ❑ Review the bedrock stabilization and excavation requirements.
- ❑ Review waterproofing and drainage system for foundation walls.
- ❑ Observe and approve the installation of the pressure relief chamber and associated piping.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations made in this report are in nature and in accordance with our present understanding of the project. A detailed investigation should be carried out to validate the recommendations presented in this report. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

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

The 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 Jadco 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.

Colin Belcourt, P.Eng.

David J. Gilbert, P.Eng.

Report Distribution

❑ Jadco Group (3 copies)
❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 150 Laurier Ave. West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 January 9

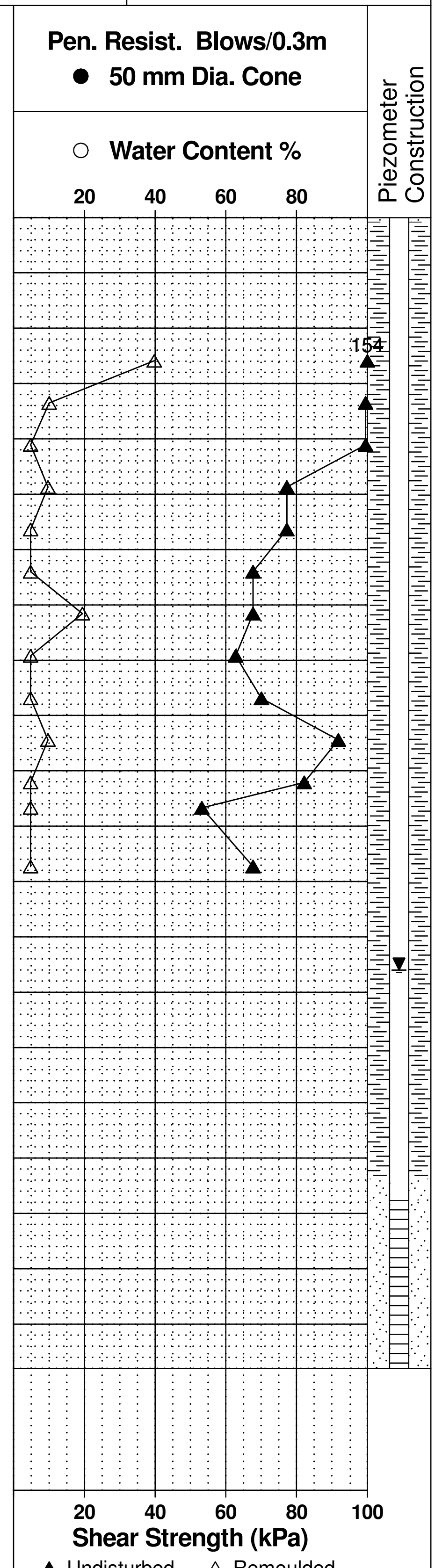
FILE NO. PG5195

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10	AU	1		
FILL: Brown silty sand with gravel	0.81	SS	2	54	14
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth	12.19	SS	3	62	12
		SS	4	100	P
		SS	5	100	P
		SS	6	100	P
		SS	7	100	P
		SS	8	100	P
			9	100	P
GLACIAL TILL: Grey silty clay with gravel, cobbles and boulders, trace sand	14.50	SS	10	100	3
		SS	11	100	5
BEDROCK: Fair quality, black shale	20.80	RC	1	100	53
		RC	2	100	15
		RC	3	100	68
		RC	4	100	77
		RC	5	88	54
End of Borehole					

(GWL @ 13.6m - Jan. 22, 2019)

DEPTH (m)	ELEV. (m)
0	69.49
1	68.49
2	67.49
3	66.49
4	65.49
5	64.49
6	63.49
7	62.49
8	61.49
9	60.49
10	59.49
11	58.49
12	57.49
13	56.49
14	55.49
15	54.49
16	53.49
17	52.49
18	51.49
19	50.49
20	49.49



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 150 Laurier Ave. West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 January 14

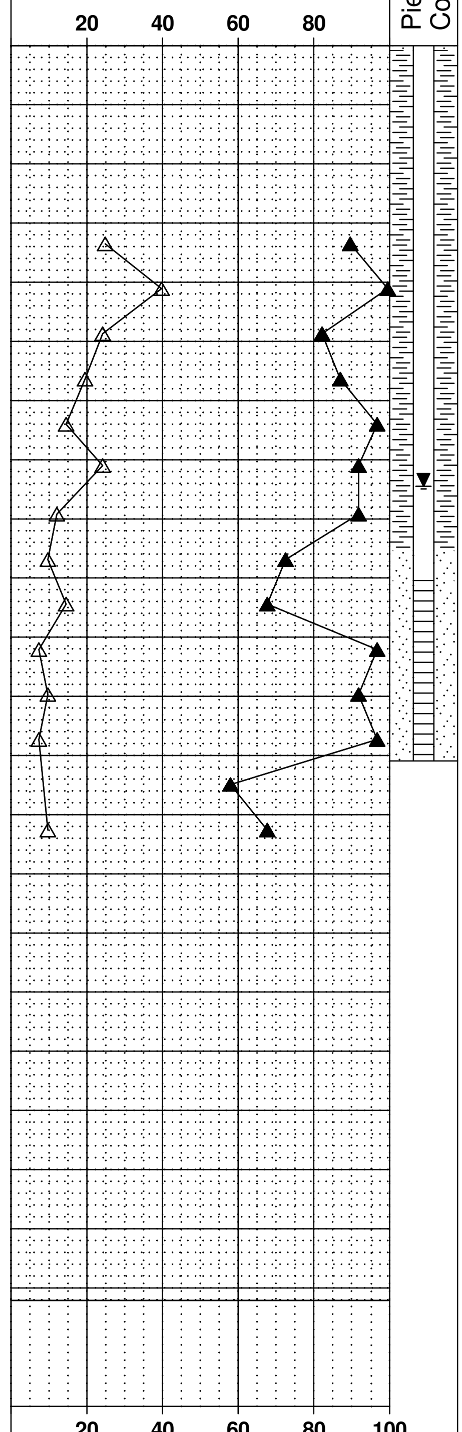
FILE NO. PG5195

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.07
Asphaltic concrete 0.10		AU	1				
FILL: Brown silty sand with gravel and clay 1.35		SS	2	54	19	1	68.07
Stiff, brown **SILTY CLAY**		SS	3	58	10	2	67.07
		SS	4	50	4		
- grey by 3.0m depth		SS	5	100	P	3	66.07
						4	65.07
		SS	6	100	P	5	64.07
						6	63.07
		SS	7	100	P	7	62.07
		SS	8	100	P	8	61.07
		SS	9	42	P	9	60.07
						10	59.07
		SS	10	62	P	11	58.07
						12	57.07
13.72		SS	11	100	P	13	56.07
GLACIAL TILL: Grey clayey silt with gravel, cobbles and boulders 15.24		SS	12	100	P	14	55.07
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders, some clay		SS	13	21	3	15	54.07
						16	53.07
17.58		SS	14	0	5	17	52.07
BEDROCK: Very poor to poor quality, grey shale		RC	1	100	0	18	51.07
		RC	2	100	27	19	50.07
		RC	3	100	41	20	49.07
21.21						21	48.07
End of Borehole							
(GWL @ 7.45m - Jan. 22, 2019)							

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 150 Laurier Ave. West
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5195**

REMARKS

HOLE NO. **BH 3**

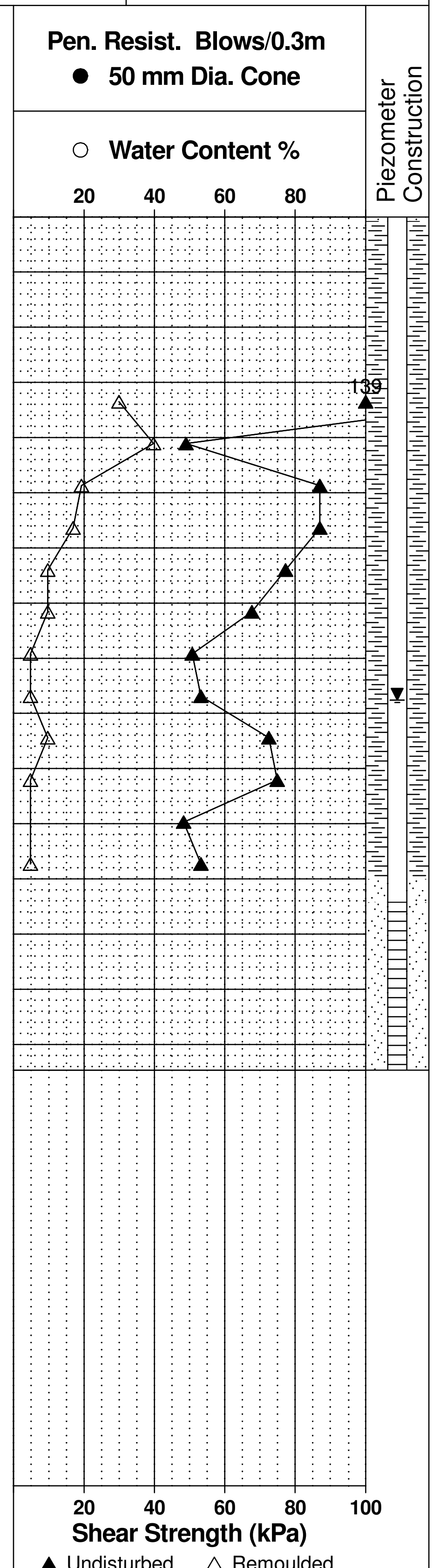
BORINGS BY CME-55 Low Clearance Drill **DATE** 2020 January 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.65
Asphaltic concrete 0.13		AU	1				
FILL: Brown silty sand with gravel, trace clay 1.52		SS	2	17	12	1	68.65
Very stiff to stiff, brown **SILTY CLAY**		SS	3	100	13	2	67.65
		SS	4	100	6		
		SS	5	100	P	3	66.65
- grey by 3.0m depth						4	65.65
		SS	6	100	P	5	64.65
		SS	7	100	P	6	63.65
						7	62.65
		SS	8	100	P	8	61.65
		SS	9	71	P	9	60.65
						10	59.65
12.19		SS	10	100	P	11	58.65
GLACIAL TILL: Grey silty clay to clayey silt with sand, gravel, cobbles and boulders 13.72		SS	11	83	10	12	57.65
		SS	12	75	3	13	56.65
GLACIAL TILL: Grey silty sand with clay, gravel, cobbles and boulders 15.47		SS	13	58	53	14	55.65
		SS	14	89	50+	15	54.65

End of Borehole

Practical refusal to augering at 15.47m depth

(GWL @ 8.76m - Jan. 22, 2019)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 150 Laurier Ave. West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2020 January 15

FILE NO. PG5195

HOLE NO. BH 4

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10	AU	1		
FILL: Brown silty sand with gravel, some clay and concrete debris	1.35	SS	2	50	9
Stiff, brown **SILTY CLAY**		SS	3	100	5
		SS	4	8	4
		SS	5	100	P
- grey by 3.0m depth		SS	6	58	P
		SS	7	71	P
	14.63	SS	8	100	P
GLACIAL TILL: Grey silty sand with gravel, cobbles and boulders, some clay		SS	9	25	24
	17.27	SS	10	29	24
BEDROCK: Good to fair quality, black shale		RC	1	100	84
		RC	2	96	91
	20.90	RC	3	100	68
End of Borehole					
(GWL @ 7.94m - Jan. 22, 2019)					

DEPTH (m)	ELEV. (m)
0	68.82
1	67.82
2	66.82
3	65.82
4	64.82
5	63.82
6	62.82
7	61.82
8	60.82
9	59.82
10	58.82
11	57.82
12	56.82
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16	52.82
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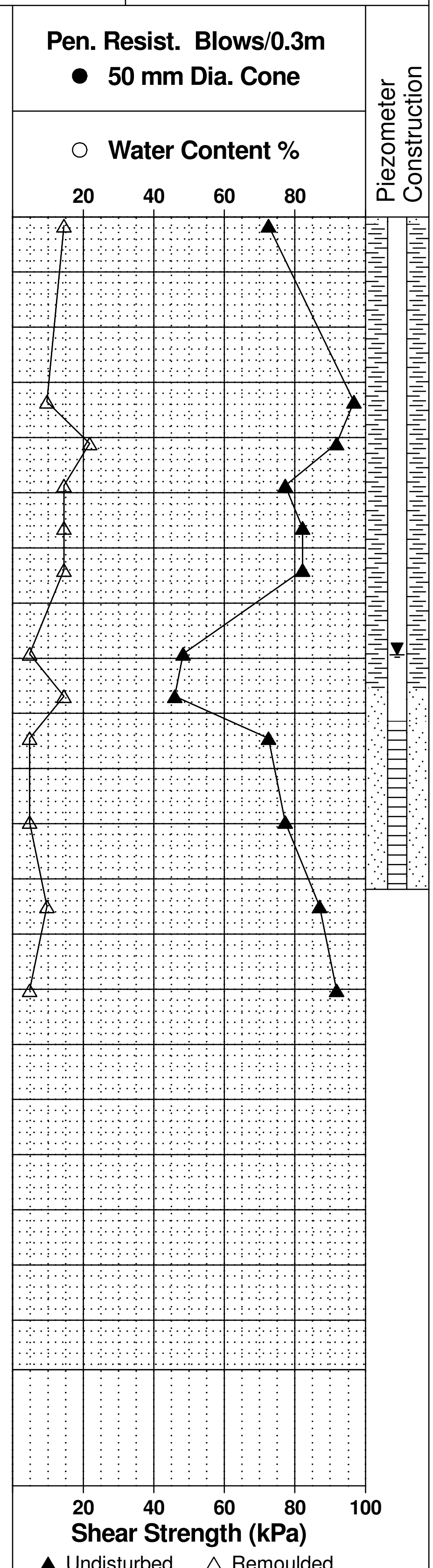
18	50.82
19	49.82
20	48.82



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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

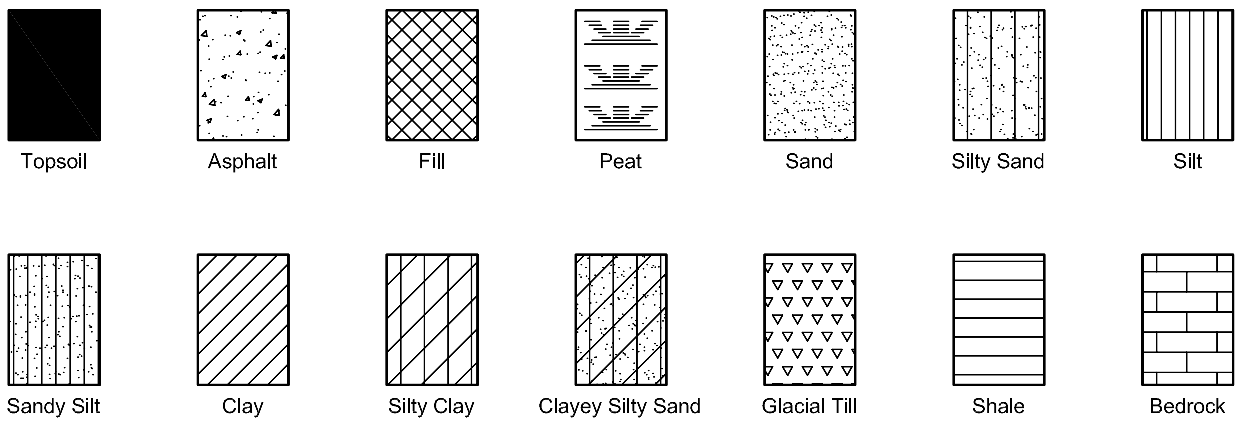
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

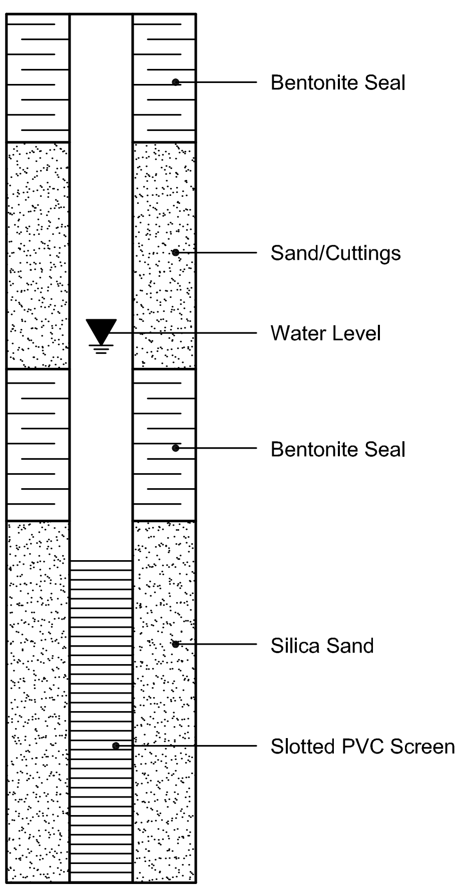
SYMBOLS AND TERMS (continued)

STRATA PLOT

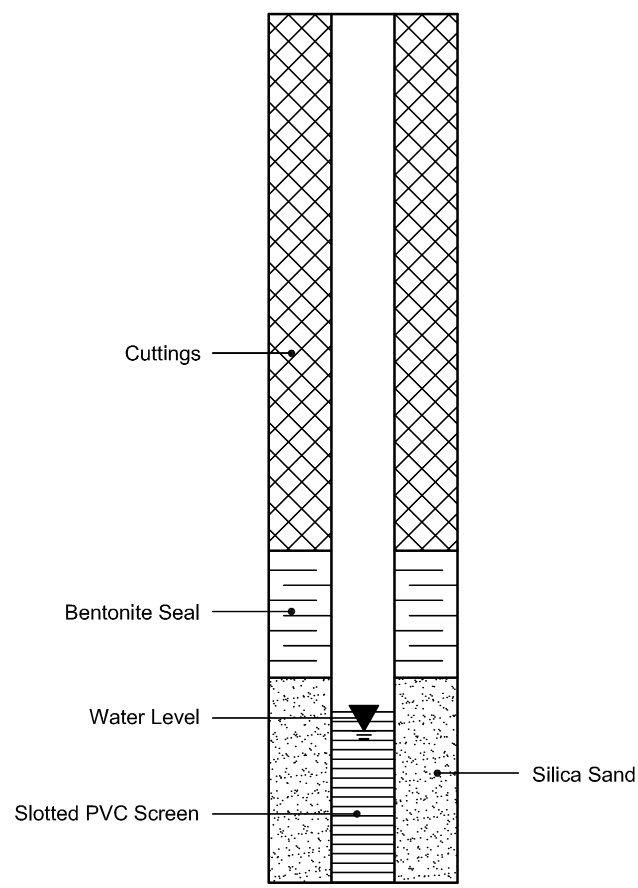


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION

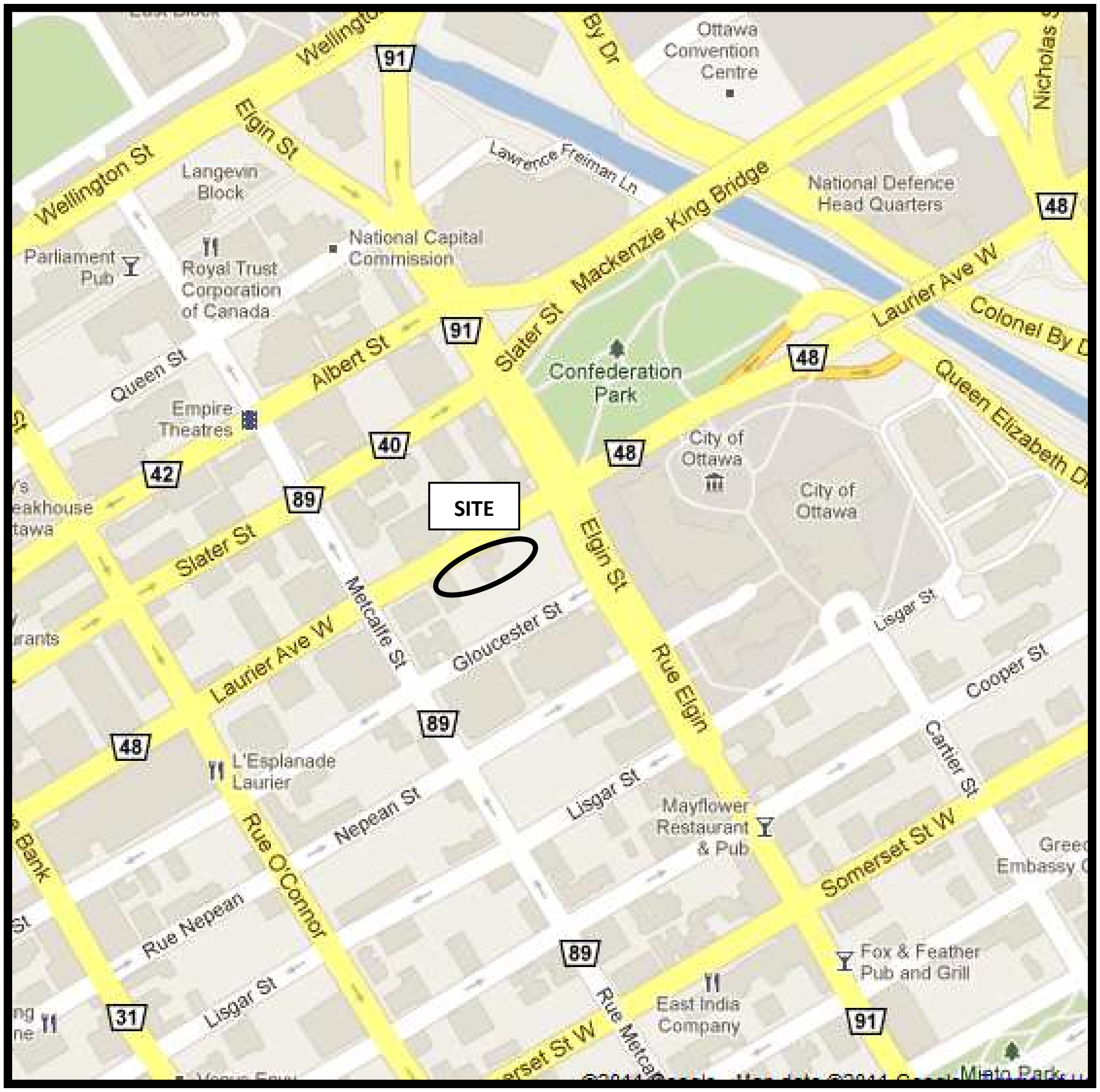


APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - TYPICAL PRESSURE RELIEF CHAMBER

DRAWING PG5195-1 - TEST HOLE LOCATION PLAN



Source: Google Maps

FIGURE 1

KEY PLAN

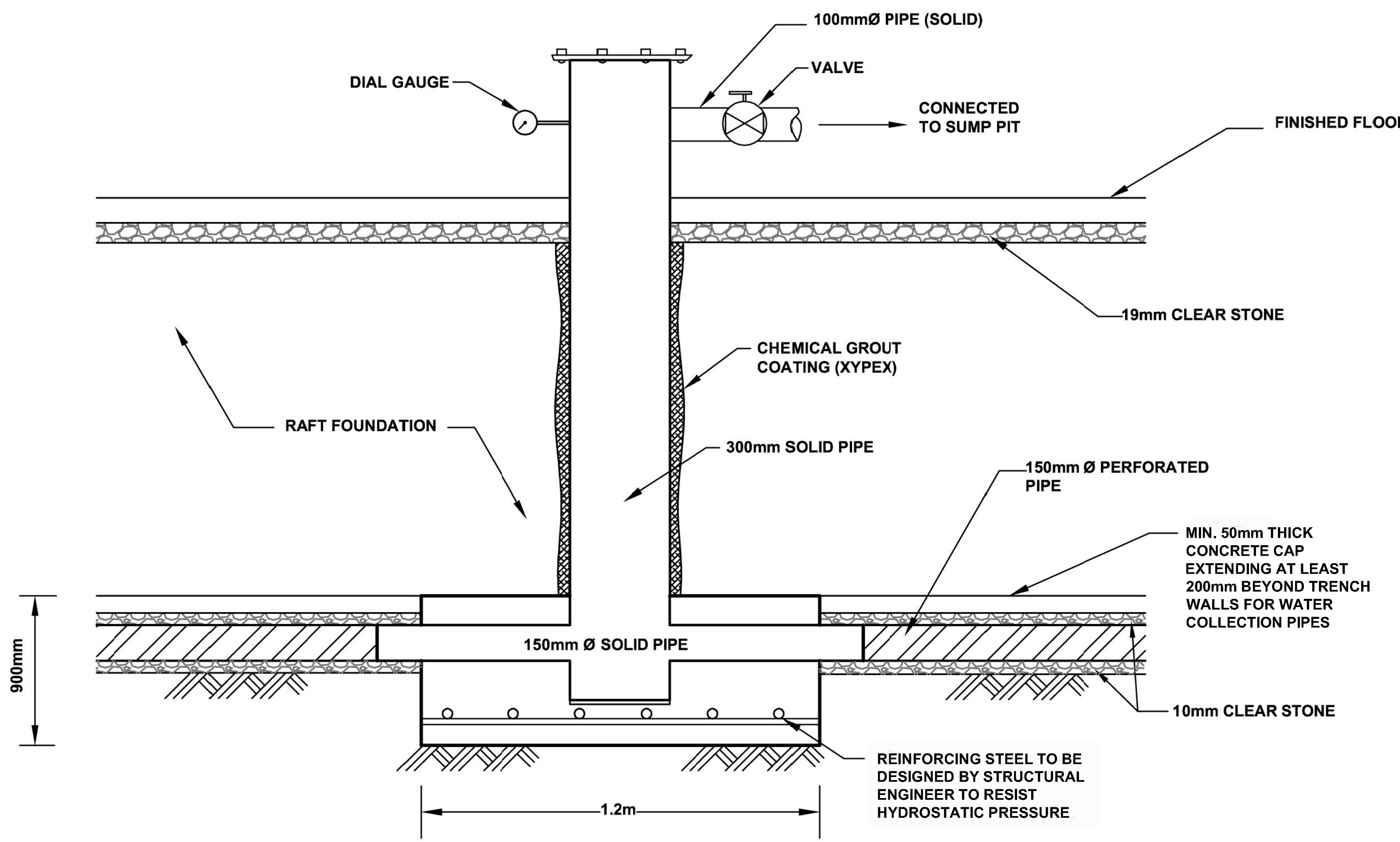
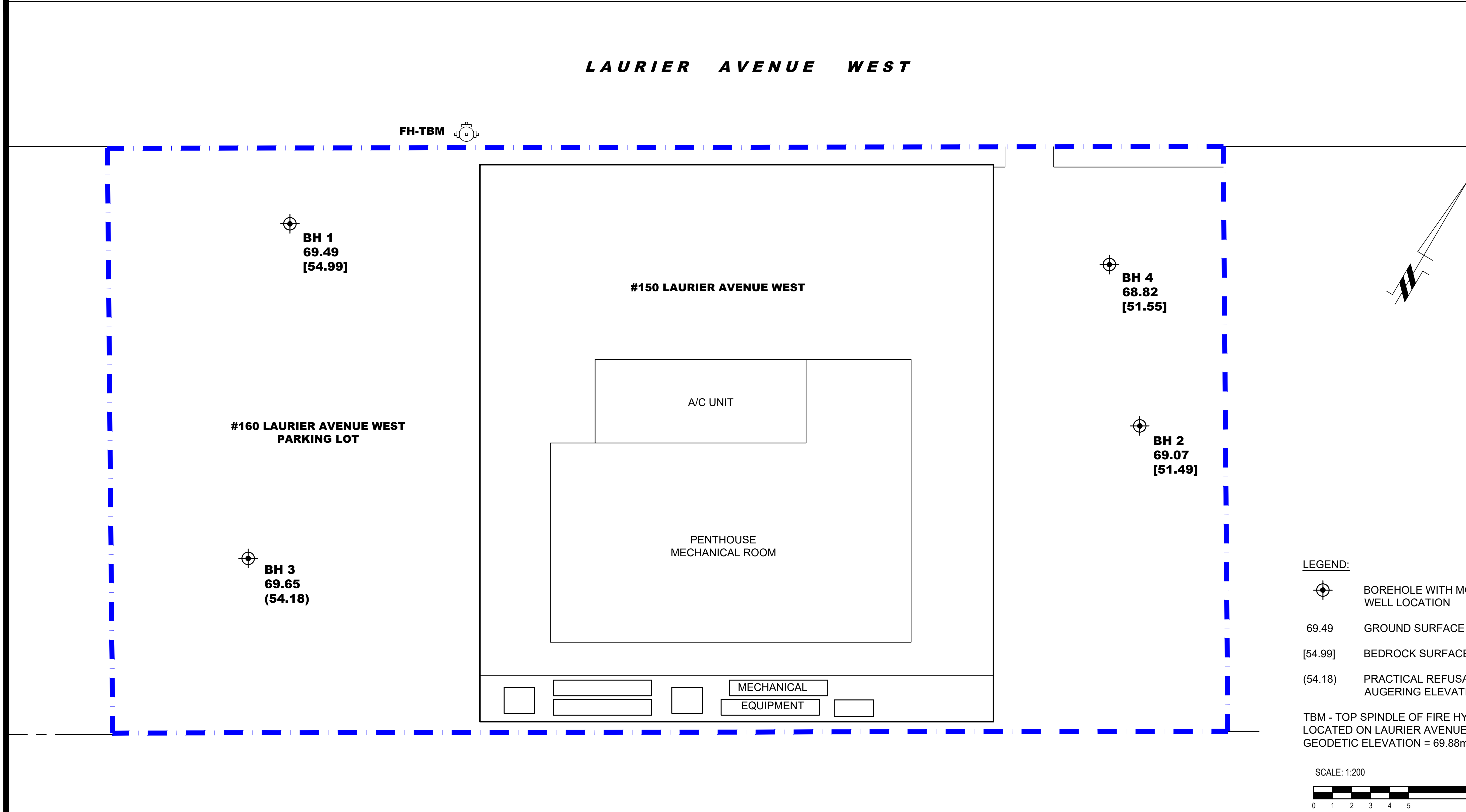


FIGURE 2 - PRESSURE RELIEF CHAMBER



LEGEND:

BOREHOLE WITH MONITORING WELL LOCATION

69.49 GROUND SURFACE ELEVATION (m)

[54.99] BEDROCK SURFACE ELEV. (m)

(54.18) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT LOCATED ON LAURIER AVENUE WEST. GEODETIC ELEVATION = 69.88m

SCALE: 1:200

0 1 2 3 4 5 10m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

JADCO GROUP

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 150 LAURIER AVENUE WEST

OTTAWA, ONTARIO

Title: **TEST HOLE LOCATION PLAN**

Scale: 1:200	Date: 01/2020
Drawn by: NFRV	Report No.: PG5195-1
Checked by: CB	Dwg. No.: PG5195-1
Approved by: DJG	Revision No.:

p:\autocad drawings\geotechnical\pg51xx\pg5195\pg5195-1 test hole location plan.dwg