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# Geotechnical Investigation

Proposed Expansion
Chateau Laurier Hotel
1 Rideau Street
Ottawa, Ontario

Prepared For

Larco Investments Ltd.

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Report PG0708-2

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# Appendices

# 1.0 Introduction

Paterson Group (Paterson) was commissioned to conduct a geotechnical investigation for the proposed addition to the Fairmont Chateau Laurier located located at 1 Rideau Street in the City of Ottawa, Ontario (refer to Figure 1, Key Plan in Appendix 3).

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 current and historical 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.

# 2.0 Proposed Development

It is our understanding that the proposed development will consist of a 12 storey building with 5 levels of underground parking. The footprint of the proposed parking garage will occupy the entire site. It is our understanding that the proposed expansion will require the demolition of the current parking structure servicing the Chateau Laurier hotel. Based on the previous report, the subject site is a former quarry.

The subject site is a rectangular-shaped parcel of land bordered to the east by Mackenzie Avenue, to the south by the existing Chateau Laurier building, to the west by the Canadian Museum of Contemporary Photography and to the north by Major’s Hill Park. The Rideau Canal, including the Ottawa locks which are the northern entrance to the canal from the Ottawa River (8 locks with a total drop of about 24 m), is located to the west of the museum.

# 3.0 Method of Investigation

## 3.1 Field Investigation

### Field Program

The field program for the investigation was carried out in the interim of October 17 to November 24, 2005. At that time, 5 boreholes (BHs 1, 2, 3, 5 and 6) were advanced to depths ranging from 2.8 to 12.1 m below existing grade. The borehole locations were selected by the structural engineer and were determined in the field by Paterson and the structural engineer taking into consideration existing services and site features. The locations of the boreholes are shown in Drawing PG0708-1 - Test Hole Location Plan included in Appendix 3.

The boreholes were put down using portable drilling equipment operated by a crew of two. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical division. The drilling procedure consisted of coring through the surficial asphalt and/or concrete layers and continually sampling the overburden. The overburden, where required, and the bedrock, at all borehole locations, were cored using diamond drilling procedures.

### Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler. The soil samples were classified on site, placed in sealed plastic bags and transported to our laboratory. Rock samples were recovered from all boreholes using a core barrel and portable diamond drilling techniques. The rock samples were classified on site, placed in hard cardboard core boxes and transported to our laboratory. The depths at which the split-spoon and rock core samples were recovered from the boreholes are shown as SS and RC, 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 a third the value of the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 21.2 kg hammer falling from a height of 760 mm. It should be noted that the N values reported on the Soil Profile and Test Data sheets are corrected to account for the fact that the weight of the hammer used with the portable drill rig is only one third of a standard hammer weight (i.e. the number of blows recorded in the field is divided by three).

Diamond drilling using portable equipment was carried out in all boreholes to determine the nature of the bedrock. The rock core samples recovered have a diameter of 38 mm (1.5 inches). The recovery value and the rock quality designation value (RQD) were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of sound rock pieces longer than 100 mm in one core run over the length of the core run. Both values are indicative of the quality of the bedrock.

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

**Groundwater**

A flexible polyethylene standpipe was installed in each borehole 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 borehole locations were determined in the field by the structural engineer and Paterson personnel, taking into consideration underground services and site features. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), which consists of the cut cross at the north end of the top flight of the stairs located on the east side of Mackenzie Avenue, opposite the existing parking structure. An elevation of 70.93 m was assigned to the TBM as provided by Novatech Engineering Consultants Ltd.

The locations of the boreholes and the ground surface elevation at each borehole are presented on Drawing PG0708-1 - Test Hole Location Plan in Appendix 3.

## 3.3 Laboratory Testing

Soil samples and bedrock cores were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

## 3.4 Analytical Testing

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

# 4.0 Observations

## 4.1 Surface Conditions

The subject site is currently being used as a parking structure. The ground surface at the lowest level of the structure slopes down to the west. A difference in grade of 1.4 m was noted between the boreholes. The ground surface of the parking garage is paved with asphaltic concrete.

## 4.2 Subsurface Profile

The soil profile at this site consists primarily of asphaltic concrete over cementitious concrete underlain by fill overlying limestone bedrock. The bedrock/inferred bedrock was encountered at all boreholes at depths ranging from 0.8 to 6.2 m below the garage floor slab. Reference should be made to the borehole logs in Appendices 1 and 2 for the details of the soil profile encountered at each borehole location.

**Asphaltic Concrete**

Asphaltic concrete was encountered at ground surface at all borehole locations. The thickness of the asphaltic concrete varies between 30 and 80 mm. A 40 mm thick asphaltic concrete layer was encountered at a depth of 0.4 m, beneath the concrete slab at BH 1.

**Cementitious Concrete**

A cementitious concrete slab was observed beneath the surficial asphaltic concrete layer at all boreholes. The thickness of the concrete slab ranges from 100 to 410 mm.

**Fill/Possible Fill**

Fill was encountered beneath the asphaltic/cementitious concrete at all boreholes. The fill extends to depths ranging from 0.8 m at BH 6 to 6.2 m at BH5. The fill consists of crushed stone, silty sand with gravel and cobbles and/or boulders. Clay pockets were encountered within the fill layer at BH 1.

A 3.4 m thick layer of silty sand with some clay was encountered at a depth of 2.8 m at BH 5. This material is possibly fill as well. Gravel, cobbles and boulders were encountered in this layer.

### Bedrock

The limestone bedrock surface was encountered at all borehole locations, at depths ranging from 0.8 m at BH 6 to 6.2 m at BH 5. Generally, the bedrock surface slopes downward from east to west.

Based on the core samples, the bedrock in this area consists of fresh grey limestone with shale partings.

The recovery values and RQD values for the bedrock cores were calculated. The recovery values range from 50 to 100%, with sixteen out of the twenty cores with a recovery value greater than 90%. The RQD values range from 0 to 93%. The rock cores with an RQD of 0% were recovered at BHs 3 and 6 in the upper portion of the bedrock, indicating the presence of fractured/weathered limestone. Generally, the RQD values vary between 36 and 93%. Out of twenty rock core samples, eight values range from 50 to 75% and another eight values are equal or greater than 75%. The quality of the bedrock ranges from very poor to excellent, and is, generally, fair to good.

Based on available geological mapping, the bedrock consists of limestone with interbeds of calcarenite and shale of the Lindsay Formation.

## 4.3 Groundwater

Groundwater levels (GWL) were measured in the standpipes installed in all boreholes on December 9, 2005. The measured groundwater levels ranged from 1.9 to 9.0 m below existing ground surface. The measured GWL readings at each borehole location are presented in Table 1, below, and on the Soil Profile and Test Data Sheets in Appendix 1.

It should be noted that these groundwater levels may be representative of coring drill water trapped in the standpipes and may not represent site conditions. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could be higher at the time of construction.

**Table 1**
**Summary of Groundwater Levels**

| Borehole Number | Measured Groundwater Level (m) | | Recording Date |
|---|---|---|---|
| | Depth | Elevation | |
| BH 1 | 8.39 | 63.90 | December 9, 2005 |
| BH 2 | Blocked | -- | December 9, 2005 |
| BH 3 | 1.88 | 69.82 | December 9, 2005 |
| BH 5 | 6.36 | 64.50 | December 9, 2005 |
| BH 6 | 9.00 | 63.24 | December 9, 2005 |

Note: The groundwater elevations are referenced to a temporary benchmark (TBM), which consists of a cut cross at the north end of the top flight of stairs located on the east side of MacKenzie Avenue, opposite the parking structure. An elevation of 70.93 m was assigned to the TBM as provided by Novatech Engineering Consultants Ltd.

# 5.0 Discussion

## 5.1 Geotechnical Assessment

The proposed development is to consist of a building with 5 levels of underground parking and 12 floors above grade. Based on information provided by the structural engineer, the founding elevation is to be approximately 15 to 16 m below the existing grade (approximate elevation 57 m) at MacKenzie Avenue. It is also understood that the lowest floor slab level of Chateau Laurier is at approximate elevation 64.8 m while that of the museum is at approximate elevation 64.6 m.

From a geotechnical viewpoint, the subject site is suitable for the proposed development. Bedrock was encountered at depths ranging from 0.8 to 6.2 m (elevations 71.4 to 64.4 m). It is anticipated that the building will be founded on footings placed directly on the limestone bedrock surface.

Bedrock excavation is expected for the construction of the underground parking levels. Line drilling, controlled blasting and rock grinding are expected for the removal of the bedrock.

Considering that the site is bound by streets and nearby institutional/commercial buildings, and that an approximately 15 to 16 m deep excavation is required, it is expected that temporary shoring above unsupported almost vertical bedrock slopes will be used to advance the proposed excavation.

Rock anchors may be required to stabilize the temporary shoring and the bedrock excavation walls. The requirement for rock anchors to stabilize the bedrock excavation walls can be established at the time of construction. For budget purposes, it can be considered that two rows of rock anchors placed at 4 m intervals will be required along the unsupported bedrock excavation walls. Additional rock anchors will be required to ensure lateral stability of the bedrock faces below the founding levels of the existing buildings.

Construction debris from the demolition of the existing on-site structure and the possible presence of the remnants of former structures could result in conditions requiring special considerations. Old foundation walls, construction debris, etc., are to be properly removed from the site, where necessary.

In view of the substantial volume of bedrock to be excavated at this site, and considering the presence of historic buildings in the subject area, the control of vibrations should be an integral part of the proposed works. The blasting operations should be planned and supervised by a professional engineer.

The above and other considerations are further discussed in the following sections. From a geotechnical perspective, the subject site is considered satisfactory for the proposed hi-rise development. 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 be accounted in the effects to adjacent structures.

Bedrock excavation is expected for the construction of the lower underground levels of the proposed building. A combination of line drilling, blasting, mechanical bedrock removal and rock grinding operations are expected for the removal of the bedrock.

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

## 5.2 Site Grading and Preparation

### Stripping Depth

Due to the depth of the bedrock at the subject site and the expected founding level for the proposed garage to accommodate 5 levels of underground parking along with the footprint of the garage that will occupy the entire boundary of the subject site, it is anticipated that all overburden material will be excavated from the subject site. Bedrock excavation will be required for the construction of the underground parking garage.

### Bedrock Removal

Based on the volume of the bedrock encountered in the area, 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 the 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 25 mm per second during the blasting program to reduce the risks of damage to the existing structures, especially the multi-storey building to the west.

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.

### Horizontal Rock Anchors

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

## 5.3 Foundation Design

### Concrete Mud Slab and Pressure Relief Chamber

To prevent the long term dewatering of the surrounding the site, at the founding level, a pressure relief chamber will be installed along with collection pipes within excavated trenches in the bedrock. It is suggested that the pressure relief chamber be incorporated in a large footing within a utility room next to the proposed sump pits. A figure illustrating an example of the required pressure relied chamber is enclosed in Appendix 3 of this report. Once the pressure relief chamber and associated piping is installed, a concrete mud slab will be placed over the entire area of the excavation.

For the concrete mud slab, it is recommended that a 300 mm thick concrete layer be used consisting of 25 MPa concrete. It is recommended that the bottom excavation be as level as possible to have a continuous pour and avoid abrupt vertical elevation changes.

### Bearing Resistance Values

Based on the subsurface profile encountered, it is expected that limestone bedrock will be encountered at the founding level and conventional spread footings founded on the bedrock surface.

A factored bearing resistance value at ULS of **4,000 kPa**, incorporating a geotechnical resistance factor of 0.5 could be used if founded on the black limestone or very sound limestone bedrock where the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within selected major footings and shear walls. At least one drill hole should be completed per major footing and shear wall. The drill hole inspection should be carried out by the geotechnical consultant.

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

**Settlement**

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

## 5.4 Design for Earthquakes

Shear wave velocity testing will need to be completed for the subject site to accurately determine the applicable seismic site classification for the proposed building from Table 4.1.8.4.A of the Ontario Building Code 2012. Based on the local knowledge of the immediate area, for preliminary design purposes, a **Site Class A** can be considered. The soils underlying the subject site are not susceptible to liquefaction.

## 5.5 Basement Slab

It is expected that most of the basement area will be used for parking. Since the foundation will consist of a conventional spread footing foundation, a minimum 150 mm layer of OPSS Granular A crushed stone material or 19 mm clear stone (if better drainage is required) will be used over the footings. The final floor slab will be poured over the compacted OPSS Granular A or 19 mm clear stone.

## 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, in our opinion, 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 ($\Delta P_{AE}$).

**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
$\gamma$ = 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 ($\Delta P_{AE}$) can be calculated using the earth pressure distribution equal to $0.375a_c\,\gamma H^2/g$ where:

$a_c$ = $(1.45\text{-}a_{max}/g)a_{max}$
$\gamma$ = 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 -<br>Hoek and Brown parameters | 44<br>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 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.

It is should be noted that due to the intended use of the rock anchors and nature of the passive rock anchor design, proof testing is not required provided that the grout installation is adequately completed to the satisfaction of the geotechnical consultant. It is recommended that compressive strength testing be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

# 6.0 Design and Construction precautions

## 6.1 Foundation Drainage and Backfill

### Foundation Waterproofing

It is understood that the building footprint will occupy the entire boundary 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 on the bedrock excavated face.

Since the lower basement levels will be located below the expected groundwater level, consideration will be given to installing a waterproofing membrane or coating for the vertical surfaces from the bottom of the excavation (bedrock vertical face) up to 1 m above the long term groundwater level (approximately 9 m below the existing grade). The bedrock vertical surface will require bedrock grinding to create a smoother bedrock surface and lessen the potential of bedrock over breakage. By waterproofing the vertical excavation sides in combination with the concrete mud slab, 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 100 or 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.

### Underfloor Drainage

Underfloor drainage will be required to control water infiltration that breaches the concrete mud slab. For design purposes, we recommend that 100 or 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.

**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 tanking the basement level. To avoid uplift on the concrete mud slab and 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 to minimize water pressure build up and 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.

## 6.3 Temporary Shoring

### Temporary Shoring Requirements

Temporary shoring may 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 4 - 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**

The necessity for underpinning adjacent structures will depend on the locations and depths of the proposed excavation and the locations and foundation depths of adjacent structures.

The lowest floor elevations at the Chateau Laurier and the Canadian Museum are 64.8 and 64.6 m, respectively. The lowest floor level of the proposed addition will be at about elevation 56.7 m. It is anticipated that the existing buildings are founded on bedrock at an elevation of approximately 64 m. Therefore, it is expected that the proposed excavation will be approximately 8 to 9 m deeper than the assumed founding levels.

Underpinning will be required, for adjacent structures founded on soil, where a line drawn up and out, at 45 degrees, from 0.3 m below the bedrock surface at the excavation wall closest to the adjacent structure, passes below the bottom of the adjacent structure founding level. Temporary shoring should not be considered as an appropriate underpinning means unless it is adequately designed to reduce potential movements to a tolerable magnitude.

Based on the information provided, the Chateau Laurier is founded, at least partially, on caissons. It is expected that underpinning and/or additional support will be required, below the existing building founding levels, at the proposed excavation walls adjacent to the Chateau Laurier and to the Canadian Museum of Contemporary Photography. A minimum setback of 1 m should be provided between the exterior side of the founding units and the wall of the proposed excavation. Also, a pattern of rock anchors could be provided to ensure the lateral stability of the founding bearing medium.

If the upper part of the caissons are above the excavation level, it will be necessary to support the caissons laterally to provide full support, from a column design perspective as well as to resist the earth pressure (at rest) behind the caissons. This support is in addition to the required support/retention of the soil between the caissons. Where a shoring system is used to support the overburden, care should be taken to locate the rock anchors away from the caissons.

Adjacent structures founded on bedrock at a higher level than the excavation may also require additional support depending on the quality of the bedrock. This should be evaluated at the time of construction.

## 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 low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow 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 and Climate Change (MOECC) 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 MOECC.

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 MOECC review of the PTTW application.

**Long-term Groundwater Control**

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater 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**

Based on our observations, the long term groundwater level is expected at a depth ranging from 7 to 9 m depth and within the bedrock. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site as a result of the proposed shoring system and waterproofing required adjacent to any neighbouring structures. No significant 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

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.

## 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 the subject site. The results of the chloride content, pH and resistivity indicate the presence of a moderate to severely aggressive environment for exposed ferrous metals at this site.

# 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 system for foundation walls.
- ❑ 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 soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request 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 Larco Investments 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.**

Carlos P. Da Silva, P.Eng.

**Report Distribution**

❑ Larco Investments (3 copies)
❑ Paterson Group (1 copy)

# APPENDIX 1

**SOIL PROFILE AND TEST DATA SHEETS**

**SYMBOLS AND TERMS**

**ANALYTICAL TEST RESULTS**

**patersongroup** Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

# SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Addition, Fairmont Chateau Laurier
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY Portable Drill

DATE 18 OCT 05

FILE NO. PG0708

HOLE NO. BH 1

| SOIL DESCRIPTION | Depth to base (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 72.29 |
| Asphaltic concrete | 0.08 | | | | | | |
| Concrete slab | 0.36 | SS | 1 | 56 | 11 | | |
| Asphaltic concrete | 0.40 | | | | | | |
| FILL: Crushed stone | 0.91 | SS | 2 | | 33+ | 1 | 71.29 |
| FILL: Boulders with pockets of silty clay with sand and gravel | | | | | | 2 | 70.29 |
| | | SS | 3 | 50 | 4 | | |
| | 3.00 | SS | 4 | | 20+ | 3 | 69.29 |
| BEDROCK: Fresh, grey limestone with shale partings | | RC | 1 | 100 | 76 | 4 | 68.29 |
| | | | | | | 5 | 67.29 |
| | | RC | 2 | 100 | 83 | 6 | 66.29 |
| | | RC | 3 | 94 | 66 | 7 | 65.29 |
| | | | | | | 8 | 64.29 |
| | | RC | 4 | 98 | 74 | 9 | 63.29 |
| | | | | | | 10 | 62.29 |
| | | RC | 5 | 97 | 74 | 11 | 61.29 |
| | 12.04 | RC | 6 | 100 | 73 | 12 | 60.29 |
| End of Borehole | | | | | | | |
| (GWL @ 8.39m-Dec. 9/05) | | | | | | | |

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

**patersongroup** Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

# SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Addition, Fairmont Chateau Laurier
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY Portable Drill

DATE 20 OCT 05

FILE NO. PG0708

HOLE NO. BH 2

| SOIL DESCRIPTION | | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 71.42 |
| Asphaltic concrete | 0.04 | | | | | | |
| Concrete slab | 0.44 | | | | | | |
| FILL: Brown silty sand with gravel and boulders | | SS | 1 | 42 | 14 | 1 | 70.42 |
| | 1.83 | SS | 2 | 24 | 41 + | | |
| BEDROCK: Fresh, grey limestone with shale partings | | RC | 1 | 98 | 57 | 2 | 69.42 |
| | 3.91 | RC | 2 | 93 | 78 | 3 | 68.42 |
| End of Borehole | | | | | | | |
| (Standpipe blocked @ 1.1m depth - Dec. 9/05) | | | | | | | |

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

**patersongroup** Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

# SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Addition, Fairmont Chateau Laurier
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY Portable Drill

DATE 1 NOV 05

FILE NO. PG0708

HOLE NO. BH 3

| SOIL DESCRIPTION | Depth to base (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 71.70 |
| 25mm Asphaltic concrete over concrete slab | 0.36 | | | | | | |
| FILL: Brown silty sand with gravel, cobbles and boulders | 0.97 | | | | | 1 | 70.70 |
| | | RC | 1 | 83 | 0 | | |
| BEDROCK: Fresh, grey limestone with shale partings. Heavily fractured in upper 0.7m depth | 2.80 | | | | | 2 | 69.70 |
| | | RC | 2 | 100 | 65 | | |
| End of Borehole | | | | | | | |
| (GWL @ 1.88m-Dec. 9/05) | | | | | | | |

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

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

# SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Addition, Fairmont Chateau Laurier
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY Portable Drill

DATE 24 NOV 05

FILE NO. PG0708

HOLE NO. BH 5

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 70.86 |
| Asphaltic concrete 0.05 | | | | | | | |
| Concrete slab 0.41 | | | | | | | |
| FILL: Cobbles and boulders | | | | | | 1 | 69.86 |
| | | | | | | 2 | 68.86 |
| 2.82 | | | | | | 3 | 67.86 |
| Compact to dense, brown-grey **SILTY SAND**, some clay with gravel, cobbles and boulders (possible fill) | | SS | 1 | 46 | 11 | | |
| | | SS | 2 | 83 | 20 | 4 | 66.86 |
| | | SS | 3 | 44 | 23 | | |
| | | SS | 4 | 82 | 33+ | 5 | 65.86 |
| | | RC | 1 | 25 | 0 | | |
| 6.18 | | | | | | 6 | 64.86 |
| **BEDROCK**: Fresh, grey limestone with shale partings | | RC | 2 | 67 | 58 | | |
| | | RC | 3 | 100 | 63 | 7 | 63.86 |
| 7.80 | | | | | | | |
| End of Borehole | | | | | | | |
| (GWL @ 6.36m-Dec. 9/05) | | | | | | | |

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

**patersongroup** Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

# SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Addition, Fairmont Chateau Laurier
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY Portable Drill

DATE 18 OCT 05

FILE NO. PG0708

HOLE NO. BH 6

| SOIL DESCRIPTION | Depth to base (m) | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | | 0 | 72.24 |
| Asphaltic concrete | 0.05 | | | | | | |
| Concrete slab | 0.15 | SS | 1 | 67 | 17+ | | |
| FILL: Crushed stone with gravel, cobbles and boulders | 0.81 | RC | 1 | 50 | 0 | 1 | 71.24 |
| BEDROCK: Fresh, grey limestone with shale partings. Heavily fractured in upper 0.4m. | | RC | 2 | 51 | 40 | 2 | 70.24 |
| | | | | | | 3 | 69.24 |
| | | RC | 3 | 100 | 93 | 4 | 68.24 |
| | | RC | 4 | 96 | 76 | 5 | 67.24 |
| | | | | | | 6 | 66.24 |
| | | RC | 5 | 98 | 75 | 7 | 65.24 |
| | | | | | | 8 | 64.24 |
| | | RC | 6 | 100 | 86 | 9 | 63.24 |
| | | | | | | 10 | 62.24 |
| | | RC | 7 | 100 | 83 | 11 | 61.24 |
| | 12.04 | RC | 8 | 95 | 36 | 12 | 60.24 |
| End of Borehole | | | | | | | |
| (GWL @ 9.00m-Dec. 9/05) | | | | | | | |

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

# 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 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 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 |

## 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 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 |

Sand 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

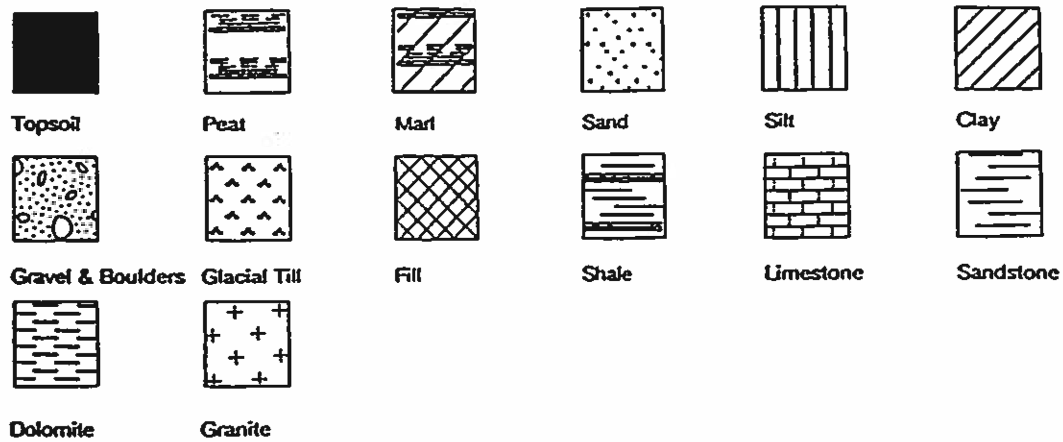
| | | |
|---|---|---|
| $p'_o$ | - | Present effective overburden pressure at sample depth |
| $p'_c$ | - | Preconsolidation pressure of (maximum past pressure on) sample |
| Ccr | - | Recompression index (in effect at pressures below $p'_c$) |
| Cc | - | Compression index (in effect at pressures above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

## PERMEABILITY TEST

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

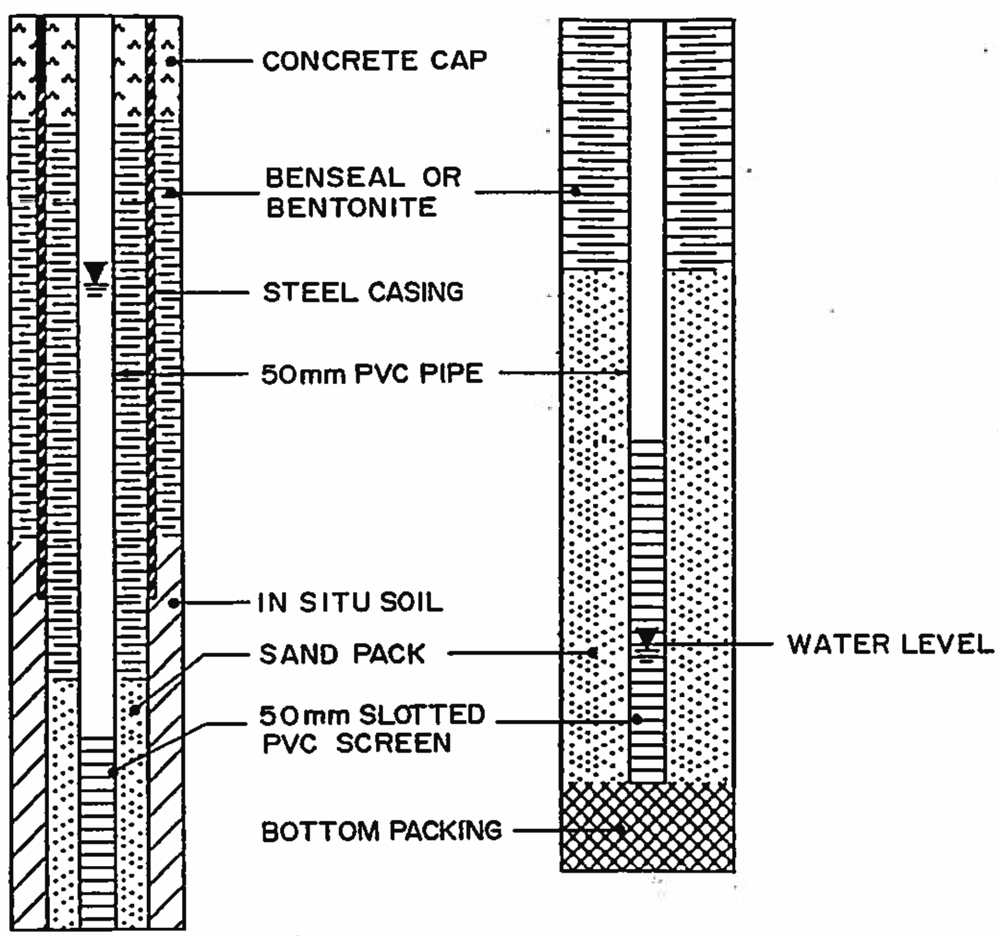
# SYMBOLS AND TERMS (continued)

## STRATA PLOT




## MONITORING WELL AND PIEZOMETER CONSTRUCTION

**Monitoring Well Construction** **Piezometer Construction**



Paracel Laboratories Ltd.

**Order #: K3337**

*Certificate of Analysis*

Report Date: 12-Dec-2005
Order Date: 07-Dec-2005

Client: **Paterson Group Inc.**
Client PO: **2096**

Project: **PG0708**

**Matrix: Soil**

| | Sample ID: | BH1 SS3 |
|---|---|---|
| | Sample Date: | 17/10/2005 |
| Parameter | MDL/Units | K3337.1 |
| Chloride | 5 ug/g | 15 |
| Sulphate | 5 ug/g | 480 |
| pH | 0.05 pH units | 7.66 |
| Resistivity | 0.1 ohm.m | 20 |

# APPENDIX 2

**PREVIOUS INVESTIGATION**
**MCROSTIE, GENEST, ST. LOUIS & ASSOCIATES LTD., REPORT NO. SF-4643**

**BOREHOLE LOGS - BH 00-1 AND BH 00-2**

**PLATE NO. 1- BOREHOLE LOCATION PLAN**

| | | |
|---|---|---|
| CHATEAU LAURIER PARKING GARAGE | B.M. (ELEV 71.16m) geodetic: Mon. # 88602 on ext. wall of Chateau Laurier, north of entrance on MacKenzie ave. | BOREHOLE NO: 00-1 |
| | | PROJECT NO: E-8003 |
| START DATE: 00/08/08 | | ELEVATION: 70.92 m |

SAMPLE TYPE: REMOULDED–AUGER / SHELBY TUBE / SPLIT-SPOON / BW-CASING / NO RECOVERY / BX CORE

| DEPTH (m) | SMALL PEN. (kPa) | SPT (N) | SAMPLE TYPE | SAMPLE NO | % CORE RECOVERY | SOIL / ROCK DESCRIPTION | ELEVATION (m) |
|---|---|---|---|---|---|---|---|
| 0.0 | | | | | | CONCRETE SLAB | |
| | | | | | | 70.52 | 70.0 |
| 1.0 | | | | | | VOID | |
| | | | | | | 69.27 | |
| | | 2 / 6 | SPLIT-SPOON | 1 | | FILL fine sand with pieces of crushed stone | 69.0 |
| 2.0 | | | | | | 68.67 | |
| | | 18 / 18 | SPLIT-SPOON | 2 | | | 68.0 |
| 3.0 | | | | | | | |
| | | 17 / 18 | SPLIT-SPOON | 3 | | POSSIBLE FILL dense to very dense sandy TILL | 67.0 |
| 4.0 | | 15 / 19 | SPLIT-SPOON | 4 | | | |
| | | 15 / 15 | SPLIT-SPOON | 5 | | | 66.0 |
| 5.0 | | 29 / 21 | SPLIT-SPOON | 6 | | | |
| 6.0 | | 21 / 12 | SPLIT-SPOON | 7 | | | 65.0 |
| | split barrel refusal, all water lost | 8/7cm | SPLIT-SPOON | 8 | | 64.35 | |
| | | | BX CORE | | 98 | LIMESTONE with shale partings | 64.0 |
| 7.0 | | | | | | 63.71 | |
| | | | BX CORE | | 95 | LIMESTONE with shale partings Borehole continued 62.92 | 63.0 |
| 8.0 | | | | | | | |

VANE Cu (kPa): 80 160 240 320
VANE Cu REMOULDED (kPa): 80 160 240 320
PLASTIC M.C. LIQUID: 20 40 60 80

| | | |
|---|---|---|
| McROSTIE GENEST ST-LOUIS Ottawa, Canada | LOGGED BY: JML | COMPLETION DEPTH: 9.75 m |
| | REVIEWED BY: JGM | COMPLETE: 00/08/08 |
| | Fig. No: 2 | Page 1 of 2 |

| CHATEAU LAURIER PARKING GARAGE | B.M.(ELEV 71.16m)geodetic; Mon. # 886G2 on ext. wall of Chateau Laurier, north of entrance on MacKenzie ave. | BOREHOLE NO: 00-1 |
|---|---|---|
| START DATE: 00/08/08 | | PROJECT NO: E-8003 |
| | | ELEVATION: 70.92 m |

SAMPLE TYPE: REMOULDED-AUGER / SHELBY TUBE / SPLIT-SPOON / BW-CASING / NO RECOVERY / BX CORE

| DEPTH (m) | SMALL PEN. (kPa) | SPT (N) | SAMPLE TYPE | SAMPLE NO | % CORE RECOVERY | SOIL / ROCK DESCRIPTION | ■ VANE Cu (kPa) ■ 80 160 240 320 / ▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320 / PLASTIC M.C. LIQUID 20 40 60 80 | ELEVATION (m) |
|---|---|---|---|---|---|---|---|---|
| 8.0 | | | BX CORE | | | LIMESTONE with shale partings | | |
| | | | | | | 62.31 | | 62.0 |
| 9.0 | | | | | 100 | LIMESTONE with shale partings | | |
| | | | | | | Bottom of hole 61.17 | | 61.0 |
| 10.0 | | | | | | | | 60.0 |
| 11.0 | | | | | | | | 59.0 |
| 12.0 | | | | | | | | 58.0 |
| 13.0 | | | | | | | | 57.0 |
| 14.0 | | | | | | | | 56.0 |
| 15.0 | | | | | | | | 55.0 |
| 16.0 | | | | | | | | |

| McROSTIE GENEST ST-LOUIS Ottawa, Canada | LOGGED BY: JML | COMPLETION DEPTH: 9.75 m |
|---|---|---|
| | REVIEWED BY: [signature] | COMPLETE: 00/08/08 |
| | Fig. No: 35 | Page 2 of 2 |

| | | |
|---|---|---|
| CHATEAU LAURIER PARKING GARAGE | B.M.(ELEV 71.16m) geodetic: Mon.# 888G2 on ext. wall of Chateau Laurier, north of entrance on MacKenzie ave. | BOREHOLE NO: 00-2 |
| | | PROJECT NO: E-8003 |
| START DATE: 00/08/07 | | ELEVATION: 71.36 m |

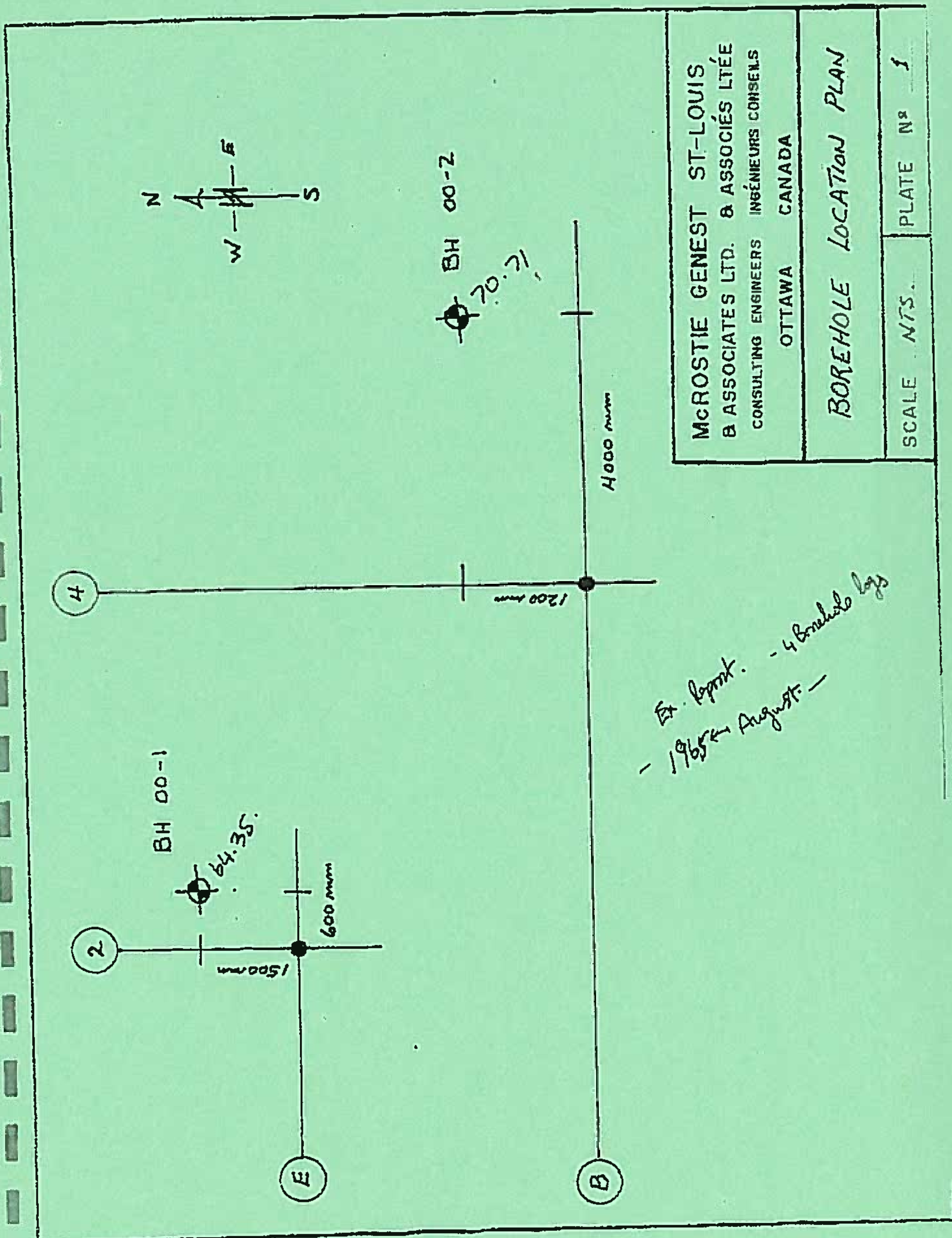
SAMPLE TYPE: REMOULDED-AUGER / SHELBY TUBE / SPLIT-SPOON / BW-CASING / NO RECOVERY / BX CORE

■ VANE Cu (kPa) ■ 80 160 240 320
▲ VANE Cu REMOULDED (kPa) ▲ 80 160 240 320
PLASTIC M.C. LIQUID 20 40 60 80

7.4 m

| DEPTH(m) | SMALL PEN. (kPa) | SPT (N) | SAMPLE TYPE | SAMPLE NO | % CORE RECOVERY | SOIL / ROCK DESCRIPTION | ELEV. | ELEVATION(m) |
|---|---|---|---|---|---|---|---|---|
| 0.0 | | | | | | CONCRETE SLAB | | 71.0 |
| | | | | | | FILL – crushed limestone | 70.91 | |
| | | | | | 91 | LIMESTONE with shale partings | 70.71 * | |
| 1.0 | | | | | 100 | LIMESTONE with shale partings | 70.39 | 70.0 |
| | | | | | 93 | LIMESTONE with shale partings | 69.99 | |
| | | | | | 100 | LIMESTONE with shale partings | 69.71 | |
| 2.0 | | | | | | | 69.46 | 69.0 |
| 3.0 | | | | | 100 | LIMESTONE with shale partings | | 68.0 |
| | | | | | | Bottom of hole | 67.60 | |
| 4.0 | | | | | | | | 67.0 |
| 5.0 | | | | | | | | 66.0 |
| 6.0 | | | | | | | | 65.0 |
| 7.0 | | | | | | | | 64.0 |
| 8.0 | | | | | | | | |

| | | |
|---|---|---|
| McROSTIE GENEST ST-LOUIS Ottawa, Canada | LOGGED BY: JML | COMPLETION DEPTH: 3.75 m |
| | REVIEWED BY: | COMPLETE: 00/08/07 |
| | Fig. No.: 4 | Page 1 of 1 |


McROSTIE GENEST ST-LOUIS
& ASSOCIATES LTD. & ASSOCIÉS LTÉE
CONSULTING ENGINEERS INGÉNIEURS CONSEILS
OTTAWA CANADA
BOREHOLE LOCATION PLAN
SCALE NTS
PLATE Nº 1
N
S
W
E
BH 00-2
70.71
BH 00-1
64.35
4000 mm
1200 mm
600 mm
1500 mm
4
2
E
B
Ex. Report. - 4 Borehole logs
- 1965 August. -

# APPENDIX 3

**FIGURE 1 - KEY PLAN**

**FIGURE - EXAMPLE OF PRESSURE RELIEF CHAMBER**

**DRAWING PG0708-1 - TEST HOLE LOCATION PLAN**

FIGURE 1
KEY PLAN

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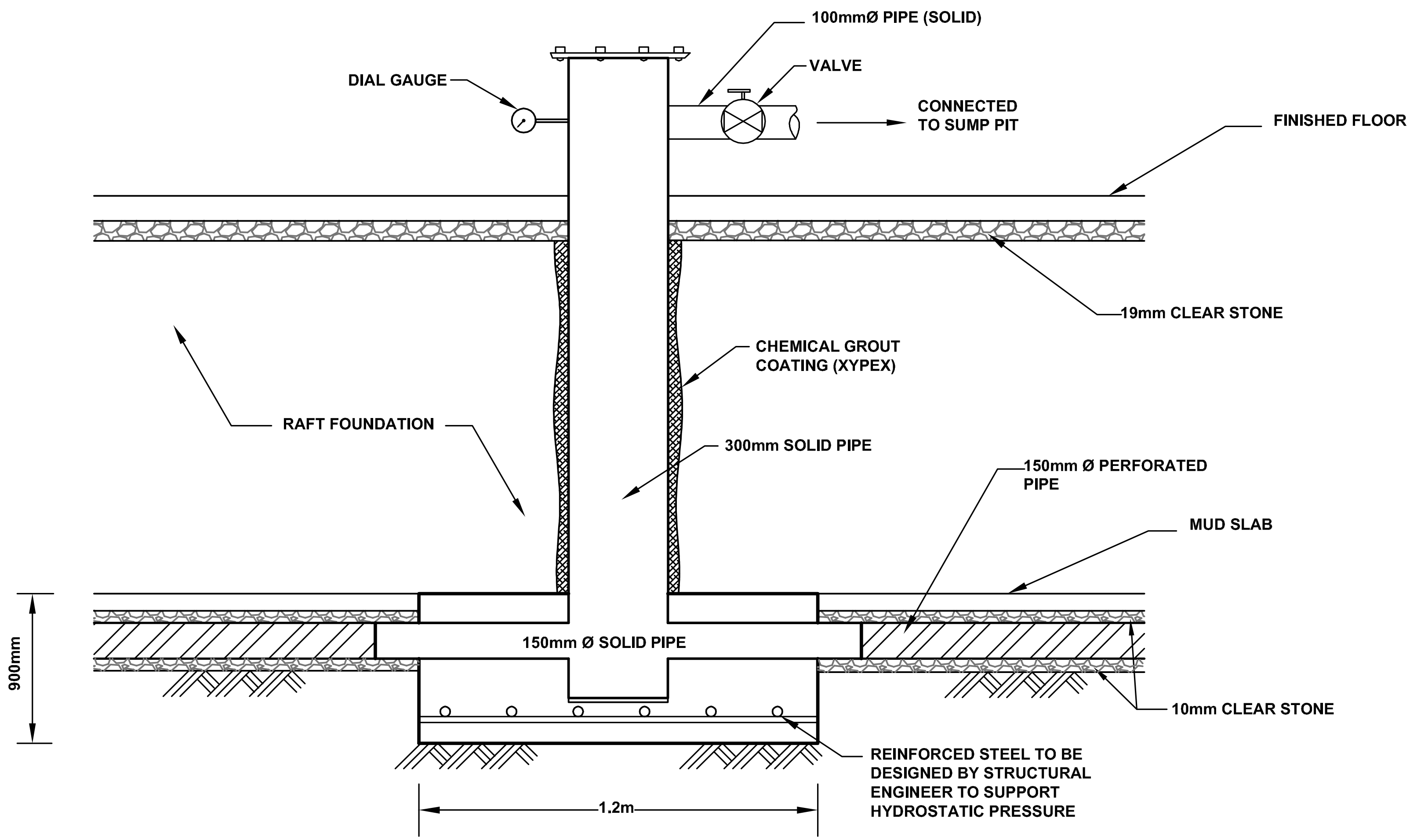


# PRESSURE RELIEF CHAMBER

patersongroup
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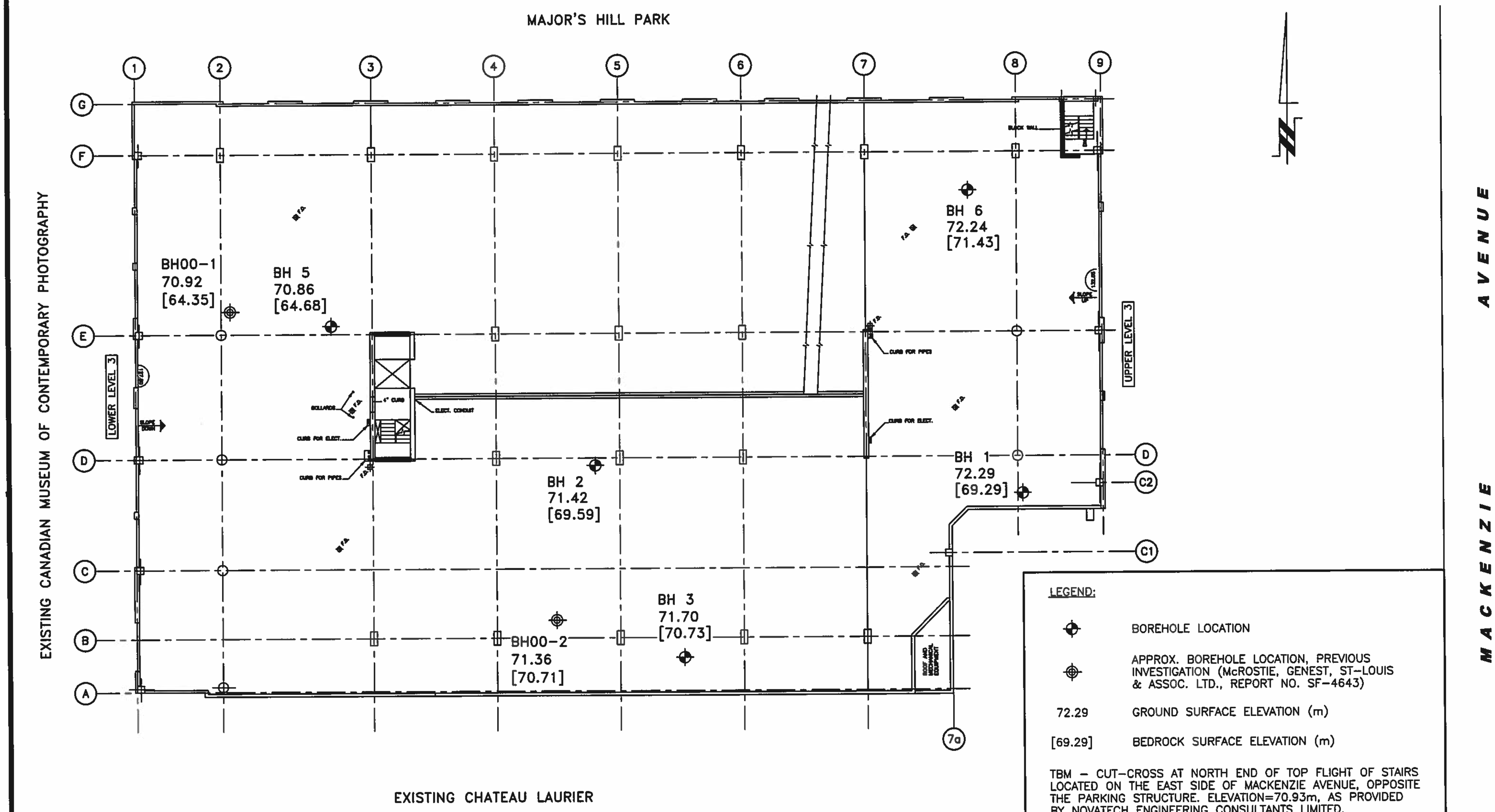


LEGEND:

| | |
|---|---|
| ⊕ | BOREHOLE LOCATION |
| ⊕ | APPROX. BOREHOLE LOCATION, PREVIOUS INVESTIGATION (McROSTIE, GENEST, ST-LOUIS & ASSOC. LTD., REPORT NO. SF-4643) |
| 72.29 | GROUND SURFACE ELEVATION (m) |
| [69.29] | BEDROCK SURFACE ELEVATION (m) |

TBM – CUT-CROSS AT NORTH END OF TOP FLIGHT OF STAIRS LOCATED ON THE EAST SIDE OF MACKENZIE AVENUE, OPPOSITE THE PARKING STRUCTURE. ELEVATION=70.93m, AS PROVIDED BY NOVATECH ENGINEERING CONSULTANTS LIMITED.

BASE PLAN PROVIDED BY JOHN G. COOKE & ASSOC. LTD.


**paterson group**
consulting engineers
28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7


| | |
|---|---|
| Scale: | 1:250 |
| Des.: | DC |
| Dwn: | JN |
| Chkd: | GC |

JOHN G. COOKE AND ASSOCIATES LTD.
GEOTECHNICAL INVESTIGATION
PROPOSED ADDITION, FAIRMONT CHATEAU LAURIER
OTTAWA, ONTARIO

# TEST HOLE LOCATION PLAN

| | |
|---|---|
| Dwg. No. | PG0708-1 |
| Report No.: | PG0708-01 |
| Date: | 12/2005 |