Geotechnical Engineering

Environmental Engineering

Hydrogeology

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Materials Testing

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Geotechnical Investigation

Proposed Multi-Storey Building
261, 269, 277 King Edward Avenue
Ottawa, Ontario

Prepared For

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Claude Lauzon Group to conduct a geotechnical investigation for a proposed multi-storey building to be located at 261, 269, 277 King Edward Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical study was:

- to review existing borehole logs and available information.
- to provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Project

It is understood that the proposed project will consist of a 5 storey residential building with one level of underground parking. It is further understood that the proposed building will encompass the majority of the subject site.

3.0 Background Information

A geotechnical investigation was completed by Inspec-Sol on February 6, 2007. At that time, two (2) boreholes were completed to provide general coverage of the proposed development. The boreholes were advanced to depths of 11.2 to 13.1 m. The locations of the test holes are presented in Drawing PG3597-1 - Test Hole Location Plan included in Appendix 2.

Diamond drilling using BQ size coring equipment was completed at borehole 2 to confirm the depth to bedrock and quality of bedrock.

The soil profiles are presented on the Borehole Records by Others in Appendix 1.

Groundwater

Perforated plastic standpipes were installed in one of the boreholes to permit monitoring of the groundwater level subsequent to the completion of the sampling program. The groundwater results are discussed in Subsection 4.3.

4.0 Observations

4.1 Surface Conditions

The property was formerly occupied by several residential buildings with either slag-on-grade construction or single basement levels. The ground surface across the site and adjacent properties is level. The site is bordered to the north partially by Murray Street and an existing residential dwelling, to the east by low rise residential buildings, to the south by Clarence Street and to the west by King Edward Street.

4.2 Subsurface Profile

The boreholes completed by others within the subject site indicate that the subsoil conditions consist of a granular fill layer at ground surface followed by a silty clay to sand fill layer. Native clayey silt with some sand was found beneath the layer of fill. The clayey silt is grey, stiff and moist to wet (below 3.2m). A compact, grey glacial till was encountered beneath the silt deposit.

Based on the RQD values from the recovered rock core, a limestone bedrock of excellent quality was encountered below the glacial till.

Specific details of the soil profile at each test hole locations are presented on the Soil Profile and Test Data sheets in Appendix 1.

4.3 Groundwater

Groundwater levels were measured in the standpipe installed at BH 1. The results are summarized in Table 1. Groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	58.3	3.50	54.80	February 26, 2007
BH 1	58.3	3.44	54.86	March 7, 2007

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed development. Based on the proposed founding depth of approximately 3 to 4 m below the existing grade and anticipated building loads, a raft foundation or conventional style footings placed on undisturbed, stiff silty clay bearing surface should be considered for the proposed building.

For two or more underground parking levels, consideration could also be given to extending the footings to the underlying inferred bedrock at a depth of 11 to 12 m below the existing grade.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

Based on the anticipated excavation depth, all topsoil and fill materials will be removed from within the perimeter of the proposed building.

Protective Mud Slab

The excavation bottom will consist of a clayey silt/silty clay deposit and should be protected from disturbance due to worker traffic. It is recommend that a minimum 50 mm thick lean concrete mud slab should be poured on the undisturbed clay surface once exposed. The lean concrete can consist of minimum 15 MPa compressive strength concrete.

Fill Placement

Fill placed for grading beneath the proposed building, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery. The granular material should be placed in maximum 300 mm thick lifts and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building should be compacted to at least 98% of the Standard Proctor Maximum Dry Density.

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed under the subgrade level for areas to be paved, the material should be compacted in thin lifts to a minimum density of 95% of the SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values (one level of Underground Parking)

Footings placed on an undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**.

Footings designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to an engineered fill or stiff silty clay bearing medium when a plane extending, a minimum of 1.5H:1V, from the bottom edge of the footing to the founding soil/engineered fill.

Lean Concrete Filled Trenches (two or more levels of Underground Parking)

For improving the bearing resistance values while remaining with two levels of underground parking, consideration should be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**20 MPa** 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, a test pit should be undertaken to assess the water infiltration issues and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the limestone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa** incorporating a geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS and a bearing resistance value at serviceability limit states (SLS) of **1,500 kPa**.

Bearing Resistance Values (3 Levels of Underground Parking)

Footings placed on a clean surface sounded limestone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa** incorporating a geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS and a bearing resistance value at serviceability limit states (SLS) of **1,500 kPa**.

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.

Raft Foundation

Consideration should be given to a raft foundation to found the proposed structure.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. A bearing resistance value at serviceability limit states (SLS) (contact pressure) of **100 kPa** could be used. The loading conditions for the contact pressure are based on sustained loads, that are generally 100% dead load and 50% live load. The factored bearing resistance at ultimate limit states (ULS) is calculated to be **200 kPa**. 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 **5 MPa/m** for a contact pressure of **100 kPa**. The design of the raft foundation should consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modelling the soil structure interaction should consider the bearing medium to be elastic and to assign a subgrade modulus. However, sensitive silty clay is not elastic and limits have to be placed on the stress ranges of a particular modulus. This value can be re-evaluated once detail of the structural design becomes available.

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 25 and 15 mm, respectively. The base of the raft foundation (approximately 0.8 to 1 m thick) is to be located at a depth of approximately 4 m below the current ground surface. The long-term groundwater level (LGWL) is estimated to be at a depth of 4 to 5 m below the existing grade. The raft slab is impervious and the basement walls will be provided with a perimeter foundation drainage system.

Deep Foundation

Deep foundation methods could be considered for the proposed structure. Concrete filled steel pipe piles to refusal on a bedrock surface are the most common deep foundation option constructed in the Ottawa area. An alternative would be the use of augered caissons socketed into bedrock.

If a deep foundation option is considered, more information can be provided upon request. Caissons should bearing surface should be free of any loose material and inspected by competent geotechnical personnel prior to pouring concrete.

5.4 Design for Earthquakes

The site class for seismic site response is a **Class D** for the footings or raft foundation founded on the silty clay deposit. A **Class A** can be given for the footings extending to bedrock. However, the higher seismic site classification must be confirmed by site specific shear wave velocity test based on the current ontario building code. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Wall

There are several applicable combinations of backfill materials and retaining soils for the basement walls of the subject structure. However, the conditions should be designed by assuming the retaining soil consists of a material with an angle of internal friction of 30 degrees and a dry 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) should be calculated by 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}) should 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:

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

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

5.6 Basement Slab

It is expected that the lower basement slab will be placed over shallow footings or a raft foundation. It is expected that the basement area will be mostly parking and that a concrete slab with a sub-floor granular layer will be incorporated in the design to accommodate services. A rigid pavement structure is presented in Subsection 5.8. The thickness of the granular sub-floor layer will be dependent on what services are incorporated in the design. It is also expected that a sump pit will be incorporated to drain any water which enters the granular layer via a breach in the raft slab or waterproofing membrane.

A concrete mud slab should be placed to protect the native soil from worker traffic and equipment before pouring the raft slab.

5.7 Rock Anchor Design

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

A third failure mode of shear failure along the grout/steel interface should 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.

The centre to centre spacing between bond lengths should be at least 1.2 m or a minimum of four times the anchor hole diameter to ensure the group influence effects are minimized. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout fluid does not flow from one hole to an adjacent empty one.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor is provided with a fixed anchor length at the anchor base, which will provide the anchor capacity, and an free anchor length between the rock surface and the top of the bonded length. As the depth at which the apex of the shear failure cone develops 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 cementitious grout. The free anchor length is provided by installing a sleeve to act as a bond break, with the sleeve filled with grout. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, should be provided. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

The rock anchor capacity depends on the dimensions of the rock anchors and the anchorage system configuration. Based on bedrock information, a **Rock Mass Rating (RMR) of 44** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.128 and 0.00009**, respectively.

Recommended Grouted Rock Anchor Lengths

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

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	44 m=0.128 and s=0.00009
Unconfined compressive strength - Limestone	50 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths are provided in Table 3. The factored tensile resistance values provided 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	2	3.4	5.4	450
	2.6	3.6	6.2	600
	3.2	3.8	7	750
	4	3.9	7.9	900
125	1.2	3.7	4.9	450
	1.6	4.1	5.7	600
	1.9	4.4	6.3	750
	2.4	4.7	7.1	900

Other considerations

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter. The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie pipe is recommended to place grout from the bottom to top of the anchor holes.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on test procedures 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.

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

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

Asphalt pavement is not anticipated to be required at the subject site. However, should pavement be reconsidered for the project, the recommended pavement structures shown in Tables 5 and 6 would be applicable.

Table 5 - Recommended Flexible Pavement Structure - Car Only Parking Areas

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

**Table 6 - Recommended Flexible Pavement Structure
Access Ramp/Lanes and Heavy Truck Parking Areas**

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is understood that the building foundation walls will be placed in close proximity to all the boundaries. It is expected that insufficient room will be available for most of the exterior backfill along these walls and, therefore, the foundation walls will be blind poured against a drainage system placed against the shoring face. It is understood that a portion of the east foundation wall may be provided with sufficient room to allow for a two sided pour and the drainage and waterproofing system can be placed directly against the concrete foundation wall.

A waterproofing membrane will be required to lessen the effect of water infiltration for the basement levels starting at 4 or 5 m below finished grade if 2 or more underground parking levels are considered. The waterproofing membrane can be placed over a suitably prepared surface and should extend to the footing founding depth. The vertical membrane should extend horizontally at least 1 m beneath the footings to ensure a suitable seal with the horizontal waterproofing membrane layer.

It is recommended that the composite drainage system (such as Miradrain G100N or equivalent) be placed directly against the foundation wall and 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.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where space is available for conventional wall construction, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

6.3 Excavation Side Slopes

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

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

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

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

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

Temporary Shoring

The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's representative prior to implementation.

The temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels. A periodic survey should be completed by the project surveyors to verify movement, if any, of the shoring system if a raker style support system is required.

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

Table 7 - 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 used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in a maximum lift thickness of 300 mm and compacted to a minimum of 95% of the SPMDD. The bedding material should extend at least to the spring line of the pipe.

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

Clay Seals

Trenches located below the groundwater table should be provided with clay seals to reduce long-term groundwater lowering. The clay seals should be as per Standard Drawing No. S8 of the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. The seals should be at least 1.5 m long (in the trench direction), as compared to the 1 m minimum in the detail, and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material.

The barriers should consist of relatively dry and compactable stiff silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

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) 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 encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e.- less than 10,000 L/day) with 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 the excavation depth of the proposed underground parking (one level) and long term groundwater level, negligible dewatering is anticipated at the subject site. Therefore, no adverse effects from temporary (during construction) and long term dewatering to the surrounding buildings or properties are expected for the subject site.

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 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.

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

7.0 Recommendations

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

- ❏ 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.
- ❏ Field density tests to determine the level of compaction achieved.
- ❏ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. Also, our recommendations should be reviewed when the project 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 that we be notified immediately in order 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 Claude Lauzon Group or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

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Report Distribution:

- ❏ Claude Lauzon Group (3 copies)
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APPENDIX 1

BOREHOLE LOGS BY OTHERS

REFERENCE No.: T020191-A1

ENCLOSURE No.: 1

INSPEC-SOL

BOREHOLE No.: BH1
ELEVATION: 58.3 m

BOREHOLE LOG

Page: 1 of 1

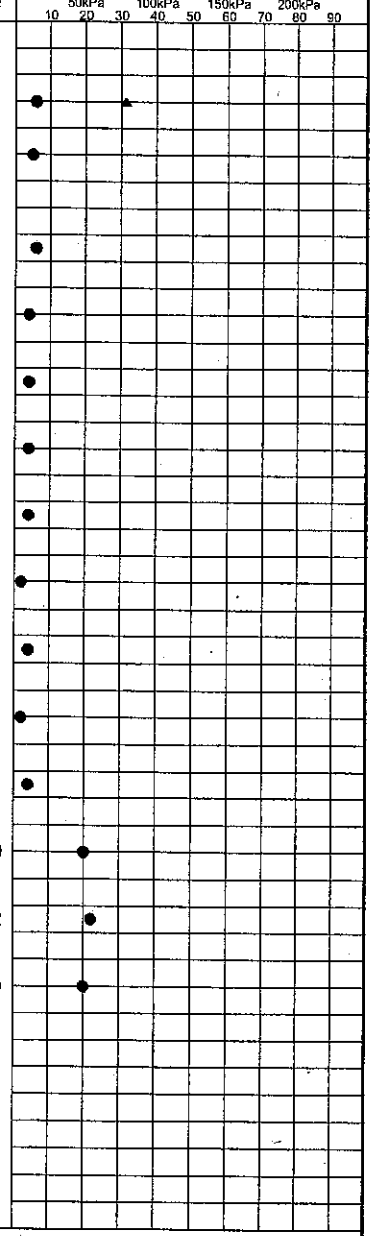
CLIENT: Claude Lauzon
PROJECT: Proposed Five Storey Apt Building
LOCATION: 260 Murray St, Ottawa, Ontario
DESCRIBED BY: M. Patterson   CHECKED BY: C.Kamichaitis
DATE (START): February 6, 2007   DATE (FINISH): February 6, 2007

LEGEND

- SS Split Spoon
- ST Shelby Tube
- RC Rock Core
- Water Level
- o Water content (%)
- Atterberg limits (%)
- ● N Penetration Index based on Split Spoon sample
- • N Penetration Index based on Dynamic Cone sample
- △ Cu Shear Strength based on Field Vane
- □ Cu Shear Strength based on Lab Vane
- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (metres)	Elevation (m)	Description of Soil and Bedrock	Type and Number	Recovery (%)	Unit Weight (kg/m³)	Penetration Index / RQD (N)
	58.3	GROUND SURFACE				
		Granular foundation, crushed stone (20mm), brown, frozen				
1.0		Fill - Sand, some silt, loose, brown, moist	SS1	50		6
		Silty clay, grey, moist and stiff	SS2	50		5
2.0		Silt, some clay and sand, loose, grey, very moist	SS3	63		6
3.0	WL3.4	Becoming wet	SS4	88		4
4.0			SS5	100		4
5.0		Becoming saturated	SS6	100		4
6.0			SS7	50		4
			SS8	100		2
7.0		Becoming sandy, trace gravel	SS9	100		4
8.0			SS10	100		2
9.0			SS11	100		4
		Sandy Silt, some clay and gravel, compact, grey, saturated	SS12	54		20
10.0			SS13	17		22
11.0			SS14	67		20
12.0		End of borehole with auger refusal at 11.3m				
13.0						

SCALE FOR TEST RESULTS



NOTES:

REFERENCE No.: T020191-A1

ENCLOSURE No.: 2

INSPEC-SOL

BOREHOLE No.: BH2
ELEVATION: 58.5 m

BOREHOLE LOG

Page: 1 of 1

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PROJECT: Proposed Five Storey Apt Building
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DESCRIBED BY: M. Patterson CHECKED BY: C.Kamichaitis
DATE (START): February 6, 2007 DATE (FINISH): February 6, 2007

LEGEND

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- S Sensitivity Value of Soil
- ▲ Shear Strength based on Pocket Penetrometer

Depth BGS (metres)	Elevation (m)	Description of Soil and Bedrock	Type and Number	Recovery (%)	Unit Weight (kg/m³)	Penetration Index / RQD (N)
	58.5	GROUND SURFACE				
		Granular foundation, crushed stone (20mm), brown, frozen				
1.0		Some silt, very loose, topsoil inclusions, brown, moist	SS1	25		3
		Silty clay, some sand, some silt and brick debris, brown, moist	SS2	42		6
2.0			SS3	100		10
		Silt, some clay and sand, loose, grey, very moist	SS4	100		6
3.0		Becoming wet	SS5	100		4
4.0			SS6	100		4
5.0		Becoming saturated	SS7	100		4
6.0			SS8	100		5
			SS9	100		4
7.0			SS10	100		4
8.0		Becoming sandy, trace gravel	SS11	100		4
9.0			SS12	100		4
		Sandy silt, some clay and gravel, compact, grey, saturated	SS13	88		13
10.0			SS14	67		23
11.0			SS15	25		24
12.0		Limestone with interbedded shale layers, grey, excellent quality	RC16	91		91
13.0		Borehole terminated at 13.1m				

SCALE FOR TEST RESULTS: 50kPa 100kPa 150kPa 200kPa; 10 20 30 40 50 60 70 80 90

NOTES:

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG3597-1 - TEST HOLE LOCATION PLAN

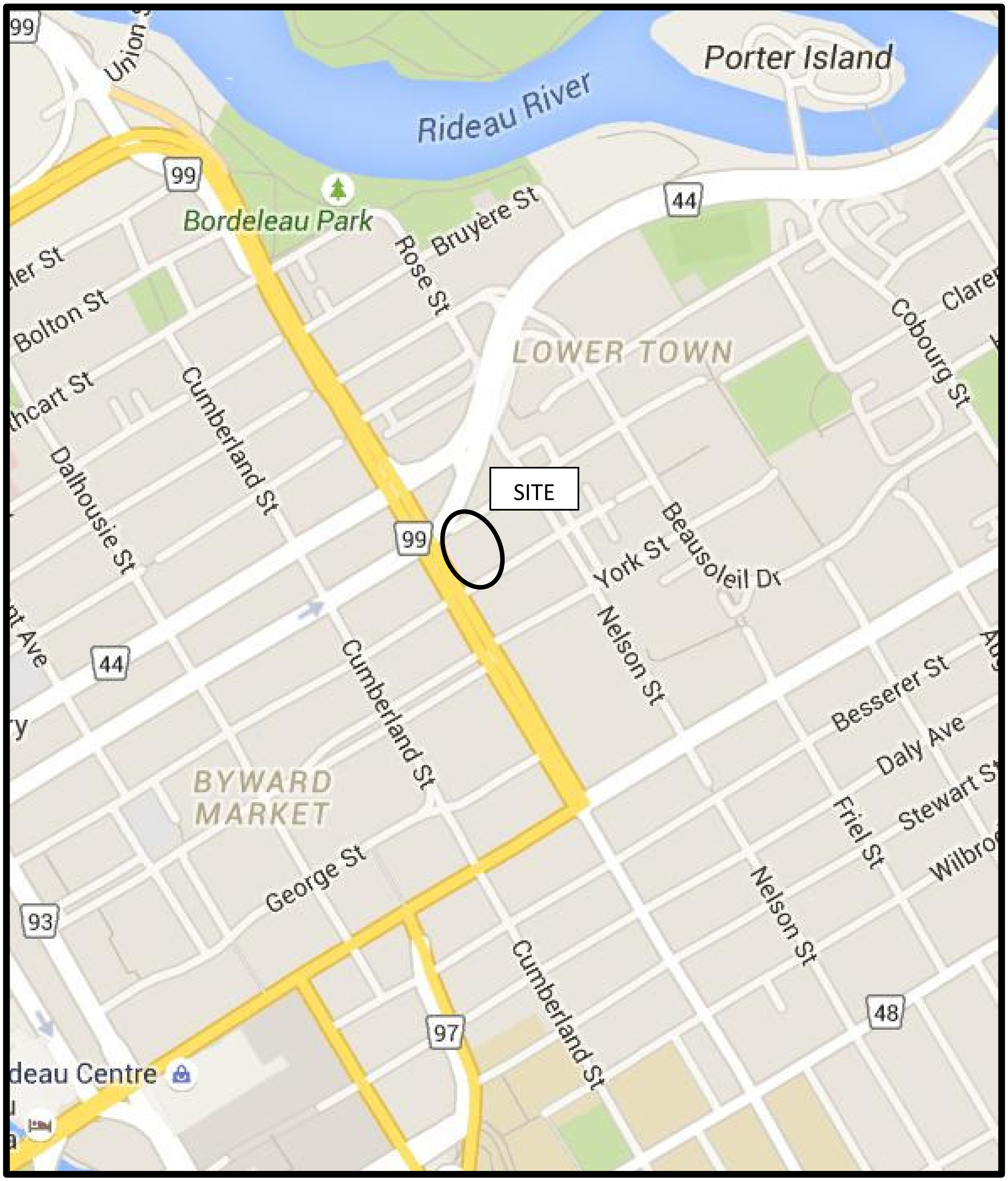
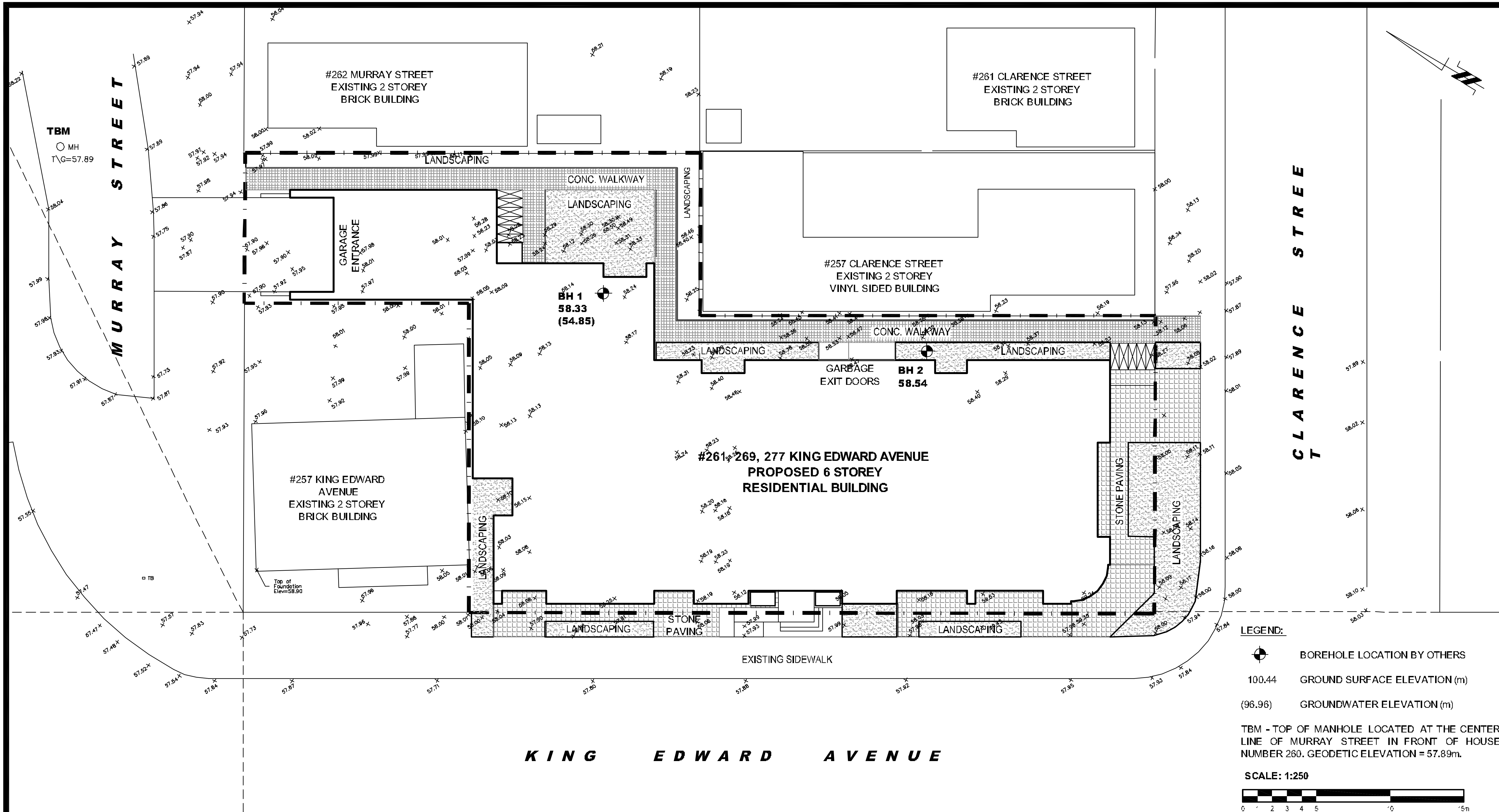


FIGURE 1

KEY PLAN



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NO.	REVISIONS	DATE	INITIAL

CLAUDE LAUZON GROUP

GEOTECHNICAL INVESTIGATION

PROP. MULTI-STOREY BUILDING - 261, 269 & 277 KING EDWARD AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 09/2015
Drawn by: CPB	Report No.: PG3597-1
Checked by: CPB	Drawing No.: PG3597-1
Approved by: DG	Revision No.:

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