765 MONTREAL ROAD - GEOTECHNICAL REPORT

Project No.: CP-17-0480-01

Prepared for:

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FOUNDATION DESIGN RECOMMENDATION REPORT
765 Montreal Road, Ottawa, Ontario

1.0 INTRODUCTION

This report presents the factual findings obtained from a geotechnical investigation performed at the above-mentioned site, for the proposed construction of a low-rise apartment building, expected to be four-storey above ground level and a walk-out basement, in the neighborhood of Vanier in Ottawa, Ontario. It is understood the existing residential home will be removed as part of the proposed construction. The field work was carried out between February 5, 2018 and February 8, 2018, it was comprised of five boreholes advanced to a maximum depth of 7.9 m below existing ground surface.

The purpose of the investigation was to explore the subsurface conditions at this site and to provide anticipated geotechnical conditions influencing the design and construction of the proposed building.

McIntosh Perry Consulting Engineers Ltd (McIntosh Perry) carried out the investigation at the request of Shepard's of Good Hope.

2.0 SITE DESCRIPTION

The property under consideration for proposed development is located at 765 Montreal Road east of the Vanier neighbourhood of Ottawa. Access to the property is granted via Lang's Road, a low density access to residential properties North of Montreal Road. Entry to the front door of the property is gained via a stone stairway which ascends to the East from the driveway off of Lang's Road. A significant grade difference exists between the top of fill at the South West corner of the property to the elevation at the intersection of Montreal Road and Lang's Road. Top of fill grade in the yard is relatively flat generally matching the grade at the front step, however within 0.75 m out from the East and North walls of the building, a grade change of 0.75 m is observed relative to the general yard grade. General grade around the property rises to the North East significantly and gradually to the South and West. Fill thickness on the property drops significantly to the South and West to match the grade of the surrounding roads. Vegetation is dense with a mix of large to medium trees and brush.

It is understood the proposed structure will be a four-storey building. Location of the property is shown on Figure 1, included in Appendix B.

3.0 FIELD PROCEDURES

Staff of McIntosh Perry Consulting Engineers (McIntosh Perry) visited the site before the drilling investigation to mark out the proposed borehole locations. Utility clearance was carried out by USL-1 on behalf of McIntosh

Perry. Public and private utility authorities were informed and all utility clearance documents were obtained before the commencement of drilling work.

The equipment used for drilling was owned and operated by OGS INC Geotechnical/Environmental Drilling of Ottawa, Ontario. Boreholes were advanced using portable drilling equipment. Boreholes were advanced to a maximum depth of 7.9 m below the ground level. Soil samples were obtained at 0.6 m intervals of depth in boreholes using a 50 mm outside diameter split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. In boreholes BH18-1 and 18-2, were refusal was met boreholes were advanced through diamond coring. Rock cores were sealed with bentonite, and boreholes were backfilled with auger cuttings. All boreholes were restored to match the original surface. Borehole locations are shown on Figure 2, included in Appendix B.

No topographic information was provided at the time of the investigation. Ground surface elevations shown in the borehole logs, used a local benchmark assumed to be El. 100.0 m, at the top of the front step by the entrance to the existing house.

4.0 LABORATORY TEST PROCEDURES

Laboratory testing on representative SPT samples was performed at McIntosh Perry geotechnical lab included moisture content. Sieve grain-size analysis, and rock core compression tests of retrieved samples, was tested by LRL Ltd. The laboratory tests to determine index properties were performed in accordance with Ministry of Transportation Ontario (MTO) test procedures, which follow American Society for Testing Materials (ASTM) test procedures.

The rest of the soil and rock samples recovered will be stored in McIntosh Perry storage facility for a period of one month after submission of the final report. Samples will be disposed after this period of time unless otherwise requested in writing by the Client.

Laboratory tests are included in Appendix C.

5.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Site Geology

Based on published physiography maps of the area (Ontario Geological Survey) the site is located within the Ottawa Valley Clay Plains. Surficial geology maps of southern Ontario identify the property as fine-textured glaciomarine deposits.

The Ottawa Valley between Pembroke and Hawkesbury, Ontario consists of clay plains interrupted by ridges of rock or sand. It is naturally divided into two parts, above and below Ottawa, Ontario. Within the valley, the bedrock is further faulted so that some of the uplifted blocks appear above the clay beds. The sediments

themselves in the valley are deep silty clay. Although the clay deposits are grey in color like the lime stones that underlies them in part, they are only mildly calcareous and likely derived from the more acidic rock of the Canadian Shield.

5.2 Subsurface Conditions

In general, the site stratigraphy consists of a topsoil, underlain by fill material, followed by a mudstone which transitioned to a shale bedrock. The soils encountered at this site can be divided into two different zones.

a) Fill
b) Bedrock

The soils encountered during the course of the investigation, together with the field and laboratory test results are shown on the Record of Borehole sheets included in Appendix C. Description of the strata encountered are given below.

5.2.1 Fill

A layer of topsoil was found at the surface of all boreholes with the exception of BH18-4. The topsoil was found to range in thickness from 0.2 m to 0.6 m. below the topsoil was observed to be a sand fill with varying portions of silt and gravel. The fill was observed to be compact to very dense, brown and moist to wet. SPT 'N' values were observed to range from 13 to 50 blows/300mm. One representative sample of the fill underwent sieve grain-size analysis and was found to contain 29% gravel, 47% sand, and 24% silt and clay. The results of the fill were compared with OPSS requirements for fill and conformed to SSM requirements. Below the topsoil in borehole BH18-4 the material was observed to transition to mudstone. Total depth of overburden material was observed to range from 0.9 m to 3.8 m (El. 99.1 m to 95.4 m), before transitioning to mudstone.

5.2.2 Bedrock

Refusal was met in all boreholes, indicating a transitioning between the topsoil and fill to bedrock. The top of the bedrock was observed to be a weathered mudstone, which in some boreholes SPT samplers and augers were capable of penetrating. In BH18-1 and 18-2, boreholes were advanced beyond auger refusal through diamond coring to confirm bedrock. The cores were measured and logged both on site and in the office, and were observed to be mud stone transitioning to shale. The rock was observed to be dark grey to black, highly to moderately weathered and highly fractured near the surface. Within the mudstone recovery was limited, core recovery measured from the retrieved shale rock cores varied between 49% and 98%. RQD measured from the retrieved shale rock cores varied between 0% and 68%, increasing with depth. Based on the RQD values, the bedrock quality may be classified as very poor to fair.

Selected rock core samples were tested in the laboratory to determine the uniaxial compressive strength. The results show an average compressive strength of 23 MPa. The results of rock core samples are included in Table 5.1 below.

Table 5-1: Compressive Strength Test

Borehole	Run No	Depth (m)	L/D Ratio	Strength (MPa)	Description of Failure
BH18-1	RC-11	5.61-5.77	2.13:1	**16.1**	Blocky, vertical and horizontal breaks
BH18-2	RC-09	4.69-4.83	2.11:1	**16.7**	Blocky, vertical and horizontal breaks
BH18-2	RC-09	5.26-5.49	2.05:1	**38.0**	Blocky, vertical and horizontal breaks

5.3 Groundwater

Water was observed in boreholes which were cored, water level elevations may be as a result of water used during coring process and not representative of groundwater elevation. Groundwater level may be expected to fluctuate due to seasonal changes.

5.4 Chemical Analysis

The chemical test results conducted by Paracel Laboratories in Ottawa, Ontario, to determine the resistivity, pH, sulphate and chloride content of representative soil sample are shown in Table 5-1 below:

Table 5-2: Soil Chemical Analysis Results

Borehole	Sample	Depth / El. (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (Ohm-cm)
BH18-1	SS-4	1.8-2.4	7.51	0.0217	0.0039	2,880

6.0 DISCUSSIONS AND RECOMMENDATIONS

6.1 General

This section of the report provides recommendations for the design of the proposed construction of a low-rise apartment building, expected to be four-storey above ground level above a walk-out basement, at 765 Montreal Road in the neighborhood of Vanier in Ottawa, Ontario. The recommendations are based on interpretation of the factual information obtained from the boreholes advanced during the subsurface investigation. The discussions and recommendations presented are intended to provide sufficient information to the designer of the proposed building to select the suitable types of foundation to support the structure.

The comments made on the construction are intended to highlight aspects which could have impact or affect the detailed design of the building, for which special provisions may be required in the Contract Documents. Those who requiring information on construction aspects should make their own interpretation of the factual

data presented in the report. Interpretation of the data presented may affect equipment selection, proposed construction methods, and scheduling of construction activities.

6.2 Project Design

6.2.1 Existing Site Condition

Detailed site condition is provided in Section 2. At the time of investigation the property contained an existing building at the north of the property and a densely vegetated yard at the south.

A significant grade difference exists between the top of the yard fill at the South West corner of the property to the elevation at the intersection of Montreal Road and Lang's Road. Top of fill grade in the yard is relatively flat generally matching the grade at the front step, however within 0.75 m out from the East and North walls of the building, a grade change of 0.75 m is observed relative to the general yard grade. General grade around the property rises to the North East significantly and gradually to the South and West. Vegetation is dense with a mix of large to medium trees and brush. The height difference is supported by a retaining wall with cobblestone face.

6.2.2 Proposed Development

It is understood the proposed structure will be a four-storey building above a walk-out basement. Location of the property is shown on Figure 1, included in Appendix B. The existing building is proposed to be demolished. Ground elevation is currently sloping about 0.8 m from north to south.

6.3 Frost Protection

Based on applicable building codes, a minimum earth cover of 1.8 m, or the thermal equivalent of insulation, should be provided for all exterior footings to reduce the effects of frost action.

6.4 Site Classification for Seismic Site Response

Selected spectral responses in the general vicinity of the site for 10% chance of exceedance in 50 years (475 years return period) are as indicated in Table 6-1, shown below and in Appendix E;

Table 6-1: Selected Seismic Spectral Responses (10% in 50 Yrs)

Sa(0.2)	Sa(0.5)	Sa(2.0)	PGA	PGV
0.164	0.089	0.021	0.104	0.069

The above noted spectral responses are for reference only and it may not indicate the critical spectrum for the proposed structure. The structural engineer shall consider deriving design specific spectral responses. The PGA for 2% probability of exceedance in 50 years is 0.284 g.

The site can be classified as a Site Class "C" based on the encountered elevation of the rock surface for the purposes of site-specific seismic response to earthquakes based on Table 4.1.8.4.A OBC 2012.

6.5 Shallow Foundations

Considering the order of structural loads and the subgrade capacity expected at the foundation level, provision of conventional strip footings and isolated pad footings may be adequate. Footings are expected to be buried to resist overturning and sliding and also to provide protection against frost action.

The encountered shale bedrock slopes from north to south. Top of shale at the southeast is approximately 2 m lower than the north and at the southwest is approximately 4 m lower than the north. The walk-out basement as indicated on the preliminary architectural sketches is only proposed for the southern half of the building which seems to be in agreement with the inferred rock slope.

In any case to minimize the risk of differential settlement, it is important to found all footings on a subgrade of similar properties. For the proposed structure, all footings shall be extended to a competent layer of shale.

The shale in some areas is overlain by a layer of mudstone which is an unstable rock of low strength. Mudstone has to be removed from the influence zone of all footings where encountered, unless otherwise approved by a geotechnical engineer. Shale at the top is also extremely weathered. All weathered and broken pieces shall be removed from the influence zone of the footings or to be stabilized by grout where considered possible. A geotechnical engineer shall review the subgrade upon excavation and provide directions to the contractor. If the rock had to be over-excavated due to unsuitable shale quality, and within the influence zone of the footings, the grade can be raised to the designated founding level by lean concrete with compressive strength not less than the bearing capacity of good quality shale as indicated in this report.

In areas outside of footings' influence zone the grade can be raised with engineered fill. The influence zone of footings are defined by a 1H:2V slope drawn downward and outward from the edge of the footing.

It should be noted that shale is typically prone to weathering and the subgrade once approved shall be covered as soon as possible to reduce the risk of shale degradation.

If adequate frost cover is not provided, the deficit of earth cover should be compensated by application of synthetic insulation material adequately projecting beyond foundation walls.

The engineered fill shall be placed in horizontal lifts of uniform thickness of no more than 300 mm before compaction. It should be placed at appropriate moisture content and compacted to a 100% standard Proctor density. The requirements for fill material and compaction may be addressed with a note on the structural drawing for foundation or grading drawing and/or with a Non-Standard Special Provision (NSSP).

6.5.1 Bearing Capacity

Bearing capacity calculations are prepared based on the assumption that all footings are extended to the shale substrate. Factored capacities for Ultimate Limit State (ULS) are provided for two different footing shapes, for strip footings and isolated pad footings. Foundations resting on rock usually experience relatively small serviceability deformations for the conventional building loads, therefore Serviceability Limit State (SLS) capacities are not relevant.

ULS bearing capacities for intact shale with estimated Rock Mass Rating (RMR) of 44;

- Strip footings with minimum 1 m width; ULS= 450 kPa
- Pad footing with minimum 1 m in shorter dimension and aspect ratio of less than 2; ULS=600 kPa

ULS bearing capacities for weathered shale with estimated Rock Mass Rating (RMR) of 29 are also provided. The weathered shale has to be approved by a geotechnical engineer to ensure the rock integrity is acceptable;

- Strip footings with minimum 1 m width; ULS= 150 kPa
- Pad footing with minimum 1 m in shorter dimension and aspect ratio of less than 2; ULS=200 kPa

6.6 Slabs-on-Grade

Free-floating Slabs-on-grade should be supported on minimum 200 mm of Granular A compacted to 100% SPMDD. In case the subgrade needs to be raised, Granular B type II or granular A needs to be compacted to minimum 96% SPMDD. If the slab-on-grade is designed to support internal columns, the fill used for the grade raise shall be compacted to minimum 100% SPMDD.

All subgrades should be approved under the supervision of a geotechnical representative prior to placement of the Granular “A” and slab-on-grade.

6.7 Lateral Earth Pressure

Free draining material shall be used as backfill material for foundation walls. If the proper drainage is provided “at rest” condition may be assumed for calculation of earth pressure on foundation walls. The following parameters are recommended for the granular backfill.

Table 6-1: Backfill Material Properties

Borehole	Granular “A”	Granular “B”
Effective Internal Friction Angle, ϕ'	35°	30°
Unit Weight, γ (kN/m^3)	22.8	22.8

7.0 CONSTRUCTION CONSIDERATIONS

Any organic material and existing fill material of any kind, should be removed from the footprint of the footings and all structurally load bearing elements. If grade raise above the native shale subgrade is required, suitable fill material to conform to specifications of OPSS Granular criteria shall be used. Grade raise above intact shale, and within the influence zone of the footing, shall be done using lean concrete with compression strength not less than the shale bearing capacity. The Structural Fill, if directly supporting the load of the structure, should be free from any recycled or deleterious material, it should not be placed in lifts thicker than 300 mm and should be compacted to 100% Standard Proctor Maximum Dry Density (SPMDD).

Based on the construction season the founding level may fall below the groundwater table. The existing weathered rock is considered permissive. Hydraulic conductivity of the intact or weathered rock was not measured as part of the scope of work. For high level estimations a hydraulic conductivity of 10^(-5) cm/s can be considered for weathered shale. The contractor shall examine the water table at a given excavation time and calculate the exposed area under groundwater table and apply for a Permit to Take Water if the extracted groundwater is suspect to exceed the allowable threshold as indicated by environmental regulations.

A geotechnical engineer or technician should attend the site to confirm the subgrade and footing preparation, as well as the type of the material and level of compaction.

Foundation walls should be backfilled with free-draining material such as OPSS Granular types A or B. The native fill might be a suitable material for backfilling (see Figure 3 in Appendix B). However the scope of work was done through limited sampling and the contractor shall examine/test bulk sample upon excavation if material re-sue is considered. Sub-drains with positive of drainage to the City sewer should be provided at foundation level.

8.0 SITE SERVICES

At the subject site, the burial depth of water-bearing utility lines is typically 2.4 m below ground surface. If this depth is not achievable due to design restrictions, equivalent thermal insulation should be provided. The contractor should retain a professional engineer to provide detailed drawings for excavation and temporary support of the excavation walls during construction.

Utilities should be supported on minimum of 150 mm bedding of Granular A compacted to minimum 96% of SPMDD. Utility cover can be Granular A or Granular B type II compacted to 96% SPMDD. All covers are to be compacted to 100% SPMDD if intersecting structural elements. The engineer designing utilities shall ensure the proposed utility pipes can tolerate compaction loads.

9.0 CEMENT TYPE AND CORROSION POTENTIAL

A soil sample was submitted to Parcel laboratories for testing of chemical properties relevant to exposure of concrete elements to sulphate attacks as well as potential soil corrosivity effects on buried metallic structural element. Test results are presented in Tables 5-1.

The potential for sulphate attack on concrete structures is moderate. Type GU Portland cement is expected to be adequate to protect buried concrete elements in the subsurface conditions encountered.

The corrosion potential for buried steel elements was determined as 'non-aggressive'.

10.0 CLOSURE

We trust this geotechnical investigation and foundation design report meets requirements of your project. The "Limitations of Report" presented in Appendix A are an integral part of this report. Please do not hesitate to contact the undersigned should you have any questions or concerns.

McIntosh Perry Consulting Engineers Ltd.

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11.0 REFERENCES

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