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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL BUILDING
3443 INNES ROAD
INNES WARD
CITY OF OTTAWA, ONTARIO

Submitted to:

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DISTRIBUTION

1 PDF copy – Mr. George Elias
1 copy – Morey Associates Ltd.

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for a proposed commercial building located at 3443 Innes Road, at the northwest corner of Innes Road and Pagé Road intersection, in Innes Ward, City of Ottawa, Ontario (see Key Plan, Figure 1 and Air Photograph, Figure 2).

The purpose of the investigation was to identify the general subsurface conditions at the site by means of a limited number of test pits and based on an interpretation of the factual information obtained and available project details to provide engineering guidelines on the geotechnical engineering aspects of the design of the project including construction considerations, which could influence design decisions.

This report has been prepared in consideration of the terms and conditions noted in the report and with the assumption that the design of the project will satisfy any applicable codes and standards. Should there be any changes in the design features outlined below, which may relate to the geotechnical considerations, Morey Associates Ltd. should be advised in order to review the report recommendations.

2.0 BACKGROUND INFORMATION AND SITE GEOLOGY

The site consists of an rectangular shaped parcel of land some 0.32 hectares in plan area. It is understood that preliminary plans are being prepared to construct a three storey commercial building with some residential apartment units, with possibly a one level heated, underground parking garage. The proposed building is likely to be of reinforced concrete, steel and wood frame construction, supported by conventional spread footing foundations. It is understood that the proposed building will be provided with a private access driveway and parking lot and possibly provided with a concrete surfaced ramp for access to an underground parking garage. Surface drainage at the site for the proposed building will likely be by means of swales, catch basins and storm sewers. The site is currently serviced by municipal water main, sanitary sewer and possibly storm sewer.

For discussion purposes Innes Road is considered to be at the south side of the site. The site is bordered on the north by an existing partly wooded single family dwelling lot with residential development beyond, on the south by Innes Road followed by commercial and residential development, on the east by Pagé Road followed by an Ultramar service station and commercial plaza with existing single family dwelling development beyond and on the west by an existing single family dwelling lot with residential development beyond.

The ground cover at the site consists, in general, of grass with some mature to young trees. An existing single storey, single family dwelling with basement and attached garage accessed by an asphaltic concrete surfaced private driveway exists within about the southwest portion of the site. It is understood that the existing dwelling at the site including the foundations will be removed. The ground surface at the site is relatively flat, with a gentle slope from the northeast portion of the site down towards the southwest portion of the site.

A review of the existing surficial geology map for the site area indicates that the site is in an area of surficial bedrock with areas thinly veneered by sediments up to 1 metre thick. Bedrock geology maps indicate that the bedrock underlying the site consists mainly of limestone, dolomite, sandstone and shale of the Ottawa Formation.

3.0 PROCEDURE

The field work for this investigation was carried out between June 15 and 20, 2017 during which time five test pits, TP1 to TP5, were put down at the site. The test pits were advanced to depths ranging from about 0.4 to 2.1 metres below the existing ground surface using a rubber tire mounted excavator supplied and operated by a local excavating contractor. The soil conditions encountered in the test pits were observed and classified by visual and tactile examination of the materials exposed on the walls and bottom of the test pits. Groundwater conditions in the test pits were noted at the time of excavating.

All of the field work was supervised by a member of our engineering staff who located the test pits in the field, logged the subsurface conditions encountered within each test pit and cared for the samples obtained. The test pits were loosely backfilled with the excavated material upon

completion. A description of the subsurface conditions encountered at each of the test pits is given in the attached Record of Test Pits sheets. The approximate locations of the test pits are shown on the attached Site Sketch Plan, Figure 3.

The ground surface elevations at the test pits were measured by members of our engineering staff relative to a temporary benchmark with an assigned elevation of 100.00 metres local datum (see Site Sketch Plan, Figure 3).

A sample of soil obtained from the site was delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack and soil corrosion on buried concrete and steel. The results of that testing are provided in the attached Appendix A.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, a description of the subsurface conditions encountered at the test pits is provided in the attached Record of Test Pit sheets. The test pit logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at other locations than the test pit locations may vary from the conditions encountered at the test pits.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and Morey Associates Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The groundwater conditions described in this report refer only to those observed at the location and date of the observations noted in the report, and on the test pit logs. Groundwater conditions may vary seasonally, or may be affected by construction activities on or in the vicinity of the site.

The following is a brief overview of the subsurface conditions encountered at the test pits.

4.2 Fill and Topsoil

From the surface at test pits 2 and 3 about a 0.9 metre thickness of fill material consisting of topsoil, sand, silt, clay and crushed stone was encountered.

From the surface at test pits 1, 4 and 5 and below the fill at test pits 2 and 3 about a 0.15 to 0.4 metre thickness of topsoil was encountered. Test pit 1 and 2 were terminated below the fill and/or topsoil material on refusal to advancement on large boulders or the upper surface of the bedrock at depths of about 0.4 and 1.3 metres below the existing ground surface, respectively.

The material was classified as topsoil based on colour and the presence of organic materials and is intended as identification for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustaining plant growth.

4.3 Silty Sand

Beneath the fill and/or topsoil at test pits 3, 4 and 5 a relatively thin layer of yellow brown to red brown to grey brown silty sand was encountered. Some gravel was observed within the silty sand material at test pit 3. Test pit 5 was terminated below the silty sand material on refusal to advancement on large boulders or the upper surface of the bedrock at a depth of about 0.6 metres below the existing ground surface. The silty sand layer was fully penetrated at test pits 3 and 4 and found to be some 0.4 and 0.15 metres in thickness, respectively. Based on tactile examination and the difficulty to advance the excavation at the test pits it is considered that the silty sand material is in a compact to dense state of packing.

4.4 Silty Clay

Beneath the silty sand at test pit 4 about a 0.5 metre thick deposit of grey brown silty clay was encountered. Test pit 4 was terminated below the silty clay material on refusal to advancement on large boulders or the upper surface of the bedrock at a depth of about 0.8 metres below the existing

ground surface. Based on tactile examination and the difficulty to advance the excavation at test pit 4, it is considered that the silty clay material is stiff to very stiff in consistency.

4.5 Glacial Till

Beneath the silty sand at test pit 3 about a 0.6 metre thick deposit of grey brown glacial till was encountered. The glacial till consists of silty sand with gravel and cobbles and a trace to some clay. The glacial till likely contains boulders, some of which could be large. Test pit 3 was terminated below the glacial till material on refusal to advancement on large boulders or the upper surface of the bedrock at a depth of about 2.1 metres below the existing ground surface. Based on tactile examination and the difficulty to advance the excavation at test pit 3, it is considered that the glacial till material is in a compact to dense in state of packing.

4.6 Groundwater

All of the test pits were dry upon completion of excavating on June 15, 2017. It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year such as the early spring.

5.0 PROPOSED COMMERCIAL BUILDING

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the information from the test holes and the project requirements. It is stressed that the information in the following sections is provided for the guidance of the designers and is intended for this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from offsite sources are outside the terms of reference for this report and have not been addressed.

5.2 Foundations for Proposed Commercial Building

With the exception of the topsoil the subsurface conditions encountered at the test pits advanced for this investigation are suitable for support of the proposed building on spread footing foundations. The excavations for the foundations should be taken down through the topsoil, fill and any deleterious material to expose the native, undisturbed compact to dense silty sand and/or stiff to very stiff silty clay and/or compact to dense glacial till and/or bedrock.

Based on the results of the investigation, strip and pad footings a minimum 0.6 metres in width and a minimum 0.6 metres square (0.6 metres by 0.6 metres), respectively, bearing on the native undisturbed silty sand and/or silty clay and/or glacial till and/or engineered fill and above the groundwater level may be designed using a maximum allowable bearing pressure of 100 kilopascals for serviceability limit states and 150 kilopascals for the factored ultimate bearing resistance.

Footings founded all on the bedrock and above the groundwater level may be designed using a maximum allowable bearing pressure of 500 kilopascals for serviceability limit states and 750 kilopascals for the factored ultimate bearing resistance.

There is no requirement for a landscape grade raise restriction adjacent to the proposed building foundations for this site.

Provided that any loose and/or disturbed soil is removed from the bearing surfaces prior to pouring concrete, the total and differential settlement of the footings, founded on the native undisturbed silty sand and/or silty clay and/or glacial till, should be less than 25 millimetres and 20 millimetres, respectively. Provided that all soil is removed from the bearing surfaces prior to pouring concrete, the total and differential settlement of the footings founded all on the bedrock should be negligible.

In view of the presence of the silty sand, silty clay and glacial till at the site, it is strongly suggested that footings for the proposed building should be founded all on the silty sand and/or silty clay and/or glacial till and/or engineered fill or all on the bedrock subgrade to minimize the potential for foundation wall cracks in overburden to bedrock subgrade transition areas. Where footings are founded on both the silty sand and/or silty clay and/or glacial till and/or engineered fill and the bedrock a suitable footing subgrade transition treatment, such as shown in Figure 4, should be used in conjunction with an engineered structural design to resist potential distress for up to 25 millimetres of differential settlement across the transition.

Any fill required to raise the footings for the proposed building to founding level should consist of imported granular material (engineered fill). The engineered fill should consist of material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular A or Granular B Type II and should be compacted in maximum 250 millimetre thick loose lifts to 100 percent of the standard Proctor maximum dry density. To allow the spread of load beneath the footings, the engineered fill should extend a minimum 0.3 metres out beyond the edge of the footings and then down and out from the top edge of the engineered fill at 1 horizontal to 1 vertical, or flatter. The excavations for the proposed building should be sized to accommodate this fill placement. Currently, OPSS documents allow recycled asphaltic concrete to be used in Granular A and Granular B Type II materials. Since the source of recycled material cannot be determined, it is suggested that any granular materials used below the founding level be composed of virgin materials only.

Groundwater inflow from the native soils into the excavations during construction, if any should be handled by pumping from sumps within the excavation.

5.3 Foundation Walls and Retaining Walls

The underground parking foundation walls should be designed to resist lateral earth pressures, P, acting against the walls at any depth, h, calculated using the following equation.

$$P = k_0 (\gamma h + q)$$

Where P = the pressure, at any depth, h, below the finished ground surface

k_0 = earth pressure at-rest coefficient, 0.5. For garage ramp retaining walls replace k_0 with $k_a = 0.35$

γ = unit weight of soil to be retained, estimated at 22 kN/m^3

q = surcharge load (kPa) above backfill material

h = the depth, in metres, below the finished ground surface at which the pressure, P, is being computed

The above equation assumes that the water level will be maintained at the founding level by means of a perimeter drainage system and foundation wall backfill requirements, as described below.

All exterior foundation elements and those in any unheated parts of the proposed building should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated foundation elements adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum 1.8 metres of earth cover. Where less than the required depth of soil cover can be provided, the foundation elements could be protected from frost by using a combination of earth cover and extruded polystyrene rigid insulation. A typical footing frost protection insulation detail could be provided, if required.

To prevent possible foundation frost jacking of unheated walls and/or isolated walls, backfill material against foundation walls should consist of free draining, non-frost susceptible material such as sand, or sand and gravel, meeting OPSS Granular B Type I grading requirements. Alternatively, foundations could be backfilled with native material in conjunction with the use of an approved proprietary drainage layer system against the foundation wall. It is pointed out that there is potential for possible frost jacking of the upper portion of some types of these drainage layer systems if frost susceptible material is used as backfill. This could be mitigated by backfilling the upper approximately 0.6 metres with non-frost susceptible granular material.

The garage access ramp retaining walls should be backfilled horizontally a distance 1.2 metres back of the wall using sand, or sand and gravel, meeting OPSS Granular B Type I grading requirements. To prevent frost heave of the garage access ramp, it is considered that the ramp

should be underlain by a minimum 50 millimetre thickness of DOW HI 40 rigid insulation or approved equivalent.

If the underground parking garage is unheated and frost susceptible material is used as backfill, measures should be taken to prevent frost penetration through the foundation walls into the backfill material. In that case suitable rigid insulation could be used on the exterior or the interior of the below grade portion of the foundation walls. A suitable foundation wall rigid insulation frost protection detail could be provided, if required.

Where the foundation backfill material will ultimately support a pavement structure or walkway, it is suggested that the foundation wall backfill material be compacted in 250 millimetre thick loose lifts to at least 95 percent of the standard Proctor dry density value. In that case any native material proposed for foundation backfill should be inspected and approved by the geotechnical engineer.

For the proposed building and retaining walls a conventional, perforated perimeter drain should be provided at founding level, leading by gravity flow to a storm sewer. The drain should be provided with a 150 millimetre thick surround of 20 millimetre crushed stone.

5.4 Underground Parking Slab on Grade

For predictable performance of the proposed building main floor slab or the underground parking concrete floor slab all soft/loose and any deleterious material should be removed from within the proposed building area. The exposed native subgrade surface should then be inspected and approved by geotechnical personnel. Any soft areas evident should be subexcavated and replaced with suitable engineered fill.

Engineered fill materials provided to support the concrete floor slab should consist of sand, or sand and gravel meeting the OPSS grading requirements for Granular B Type I or crushed stone meeting OPSS grading requirements for Granular B Type II. A minimum 150 millimetre thickness of crushed stone meeting OPSS Granular A should be provided immediately beneath the concrete floor slab. The engineered fill materials should be compacted in maximum 250 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage of the concrete. The saw cut depth should be about one quarter of the thickness of the slab. The crack control cuts should be at a grid spacing not exceeding about 5 metres.

If the underground parking garage is unheated, the floor slab will require protection from frost effects. Details for suitable frost protection using rigid insulation can be provided, if required.

5.5 Seismic Design for Proposed Commercial Building

For Seismic design purposes, in accordance with the 2012 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class C. That site class has been determined based on the limited investigation carried out at the site. A higher site class designation may result from in situ seismic velocity testing of the subsurface materials.

5.6 Potential for Soil Liquefaction

As indicated above the results of the test pits put down across the site indicate that the native deposits underlying the site consist of compact to dense silty sand, stiff to very stiff silty clay, compact to dense glacial till and shallow bedrock. Based on the relatively thin layer of silty sand that could exist beneath the proposed building footings and on the underlying stiff to very stiff silty clay, compact to dense glacial till and shallow bedrock, it is considered that no damage to the proposed building should occur due to liquefaction of the native subgrade for the expected seismic conditions in the Ottawa region.

5.7 Bedrock Excavation

In view of the relatively shallow bedrock at the site and the potential founding levels of the proposed buidling below the existing grade, excavation of bedrock may be required. It is considered that small amounts of bedrock removal could be carried out using heavy excavating equipment, but that removal of relatively large amounts of bedrock could be facilitated by means of a hoe ramming and/or blasting operation and should be carried out under the supervision of a vibration specialist engineer and/or blasting specialist engineer. Both horizontal and vertical overbreak of the bedrock

excavation face/bottom can be expected due to the hoe ramming/blasting operation. If control of potential bedrock overbreak is required, line drilling at the proposed excavation face could be carried out. The smaller the distance between the drill holes the less overbreak can be expected. It is considered that line drilling at 150 millimetre horizontal spacing to the full depth of the excavations should control overbreak to an acceptable level.

In view of the existing residential and commercial developments in close proximity to the subject site and the potential for vibration during the removal of bedrock by hoe ramming and/or blasting, it is considered that monitoring of the hoe ramming and/or blasting should be carried out throughout the operation to ensure that the limiting vibration criteria established by the specialist engineer is met. Pre-construction surveys of nearby structures and existing utilities should be carried out prior to any hoe ramming and/or blasting operations.

5.8 Potential for Sulphate Attack/Corrosion on Buried Concrete and Steel

The results of chemical laboratory testing of the above mentioned soil sample (see attached Appendix A) gave a percent sulphate (SO_4) for the soil sample of less than 0.01 and an ohm-cm resistivity and pH of 12,500 and 8.0, respectively.

The National Research Council of Canada (NRC) recognizes four categories of potential sulphate attack of buried concrete based on percent sulphate in soil. From 0 to 0.10 percent the potential is negligible, from 0.10 to 0.20 percent the potential is mild but positive, from 0.20 to 0.50 percent the potential is considerable and 0.50 percent and greater the potential is severe. Based on the above the soil sample is considered to have a negligible potential for sulphate attack on buried concrete.

The results of the laboratory testing of a soil sample for resistivity and pH indicates the soil sample tested has an underground corrosion rate of about 0.5 loss-oz./ft^2/yr. Based on the findings of Fischer and Bue (1981) underground corrosion rates (loss-oz./ft^2/yr) of 0.30 and less are considered nonaggresive, from 0.30 to 0.75 the rate is considered slightly aggressive, from 0.75 to 2.0 the rate is considered aggressive and 2.0 and greater the rate is considered very aggressive. Accordingly, the above mentioned soil sample is considered to have a slightly aggresive corrosion rate on buried bare steel. The slightly aggressive corrosion rate on buried bare steel (ie: exposed

ferrous metal) should be considered in the design of substructures and in the selection of pipe materials.

6.0 SITE SERVICES

6.1 Excavation

The excavations for the site services will be carried out through fill, topsoil and silty sand and/or silty clay and/or glacial till and, depending on the depth of the services, likely bedrock. The sides of the excavations in overburden materials should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Ontario Occupational Health and Safety Act. That is, open cut excavations with overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box. It is not considered likely due to the groundwater level measurements indicated on the test hole logs, however if some of the services excavations extend below the water table in overburden material some loss of ground and groundwater inflow may occur, requiring side slopes as flat as 3 horizontal to 1 vertical to be used. Excavation in the bedrock may be carried out with near vertical side walls.

Small amounts of bedrock removal, if required, can most likely be carried out by the use of a large excavator. If larger amounts of bedrock removal are required it may be more economically feasible to use a hoe ramming and/or blasting operation and should be carried out under the supervision of a vibration specialist engineer and/or blasting specialist engineer. In view of the existing residential and commercial developments in close proximity to the subject site and the potential for vibration during the removal of the bedrock by hoe ramming and/or blasting it is considered that monitoring of the hoe ramming and/or blasting should be carried out throughout the operation to ensure that the limiting vibration criteria established by the specialist engineer is met. Pre-construction surveys of nearby structures and existing utilities should be carried out prior to any hoe ramming and/or blasting operations.

Groundwater seepage into the excavations, if any, should be handled by pumping from sumps in the excavation.

6.2 Pipe Bedding and Cover Material

It is suggested that the service pipe bedding material consist of at least 150 millimetres of granular material meeting OPSS requirements for Granular A. A provisional allowance should, however, be made for subexcavation of any existing fill or disturbed material encountered at subgrade level. Granular material meeting OPSS specifications for Granular B Type II could be used as a sub-bedding material. The use of clear crushed stone as a bedding or sub-bedding material should not be permitted.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The sub-bedding, bedding and cover materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

6.3 Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches.

In areas where the service trench will be located below or in close proximity to the proposed access driveway and/or parking lot areas, acceptable native materials should be used as backfill between the driveway/parking lot subgrade level and the depth of seasonal frost penetrations (i.e. 1.8 metres below finished grade) in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of driveway/parking lot. Where native backfill is used, it should match the native materials exposed on the trench walls. Well shattered and graded blast rock with maximum size less than 300 millimetres could be used as trench backfill material. It is not considered likely due to the groundwater level measurements indicated on the test hole logs, however some of the native materials from the lower part of the trench excavations may be wet of optimum for compaction. Depending on the weather conditions encountered during construction, some drying of materials and/or recompaction may be required. Any wet materials that cannot be compacted to the required density should either be wasted from the site or should be used outside

of driveway/parking lot areas. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the driveway, sidewalks, parking areas etc., the trench backfill should be compacted in maximum 250 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. The specified density may be reduced where the trench backfill is not located within or in close proximity to future roadways, driveways, sidewalks, parking areas or any other type of permanent structure.

7.0 PRIVATE ACCESS DRIVEWAY AND PARKING AREA PAVEMENTS

In preparation for pavement construction at this site, all fill, topsoil and any soft, wet or deleterious materials should be removed from the proposed driveway and parking areas. The exposed subgrade should be inspected and approved by geotechnical personnel and any soft areas evident should be subexcavated and replaced with suitable earth borrow approved by the geotechnical engineer. The subgrade should be shaped and crowned to promote drainage of the proposed driveway and parking area granulars. Following approval of the preparation of the subgrade, the pavement granulars may be placed.

For areas of the site that require the subgrade to be raised to the proposed driveway and/or parking areas subgrade level, it is considered that some of the drier native materials could be used for this purpose or the material could consist of OPSS select subgrade material or OPSS Granular B Type I or Type II. Any materials proposed for this use should be approved by the geotechnical engineer before placement within the proposed driveway and parking area. Materials used for raising the subgrade to proposed driveway and parking area subgrade level should be placed in maximum 300 millimetre thick loose lifts and be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

For the proposed private access driveway and any areas of heavy truck traffic use the pavement should consist of:

40 millimetres of hot mix asphaltic concrete (HL3 or Superpave12.5) over
40 millimetres of hot mix asphaltic concrete (HL8 or Superpave19) over
150 millimetres of OPSS Granular A base over
350 millimetres of OPSS Granular B, Type II subbase

For proposed light vehicle parking areas the pavement should consist of:

50 millimetres of hot mix asphaltic concrete (HL3 or Superpave12.5) over
150 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase

Compaction of the granular pavement materials should be carried out in maximum 300 millimetre thick loose lifts to 100 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in acceptance with Table 10 of OPSS 310.

The above pavement structures will be adequate on an acceptable subgrade, that is, one where any driveway and parking area fill/services trench backfill has been adequately compacted. If the proposed driveway and parking area subgrade is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or incorporate a non-woven geotextile separator between the proposed driveway and parking area subgrade surface and the granular subbase material.

The design of asphaltic concrete is the responsibility of the asphaltic concrete designer and/or supplier employed by the owner and is outside of the scope of this report. The above pavement design has been determined to ensure that the combined strength of the pavement components (asphaltic concrete and granulars) is adequate to protect the subgrade from the expected repeated vehicle loading. It is understood that the future pavements at the subject site are to be privately

owned and as such it is at the discretion of the owner whether the additional cost of superpave is warranted.

The design life for a flexible pavement as described above is generally taken as about 11 years to first major maintenance.

8.0 CONSTRUCTION CONSIDERATIONS

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions within the proposed building area do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

All footing areas and any engineered fill areas for the proposed building should be inspected by Morey Associates Ltd. to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundations should be inspected and in situ compaction testing should be carried out to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for any site services should be inspected and approved by Morey Associates Ltd. In situ compaction testing should be carried out on the service pipe bedding and backfill to ensure the materials meet the specifications from a compaction point of view.

The native soils at this site will be sensitive to disturbance and softening from construction operations, from rainwater or snow melt, and possibly frost. In order to minimize disturbance, construction traffic operating directly on the subgrade should be kept to an absolute minimum and the subgrade should be protected from below freezing temperatures.

Any native material proposed to be used as earth fill below the pavement/sidewalks/etc. areas should be inspected and approved by Morey Associates Ltd. prior to use.

Any depressions at the footing subgrade level due to removal of boulders from within the glacial till should be filled with concrete or well compacted suitable granular material.

The existing single family dwelling and associated existing access driveway and possible previous sewage system servicing the existing dwelling is located within the area of the proposed building, and/or private access driveway and/or parking areas. The excavation for the proposed building should be extended, if necessary, to ensure all of the existing foundations and any fill materials associated with the existing dwelling (ie: sewage system, service trenches, etc.) are removed from within the proposed building area. The excavations for the proposed private access driveway and parking areas should be taken down to a native, undisturbed subgrade and as such any below grade foundations and any fill materials associated with the existing dwelling should be removed from within the private access driveway and parking area subgrades.

Bedrock removal may be required at this site for the proposed building and/or services excavations. Small amounts of bedrock removal, if required, can most likely be carried out by the use of a large excavator. If larger amounts of bedrock removal are required, it may be more economically feasible to use hoe ramming and/or blasting techniques and should be carried out under the supervision of a vibration specialist engineer and/or blasting specialist engineer. Monitoring of the hoe ramming and/or blasting should be carried out throughout the hoe ramming and/or blasting period to ensure that the bedrock removal meets the limiting vibration criteria established by the specialist engineer. Pre-construction surveys of nearby structures and existing utilities should be carried out prior to any hoe ramming and/or blasting operations.

9.0 REPORT CONDITIONS AND LIMITATIONS

It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document is neither intended nor authorized by Morey Associates Ltd. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The engineering guidelines provided in this report are based on subsurface data obtained at the specific test locations only. Boundaries between zones on the logs are often not distinct but transitional and were interpreted. Experience indicates that the subsurface soil and groundwater conditions can vary significantly between and beyond the test locations. For this reason, the engineering guidelines given in this report are subject to a field verification of the subsurface soil conditions at the time of construction.

The report guidelines are applicable only to the project described in the report. Any changes in the scope of the project will require a review by Morey Associates Ltd., to ensure compatibility with the engineering guidelines contained in this report.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we may be of further service to you, please do not hesitate to contact our office.

Yours truly,
Morey Associates Ltd.

D. G. Morey, B.A.Sc. (Civil Eng.), P.Eng.
Director/Civil Engineer

C. R. Morey, M. Sc. (Eng.), P. Eng
Senior Consulting Engineer

File: 017209

KEY PLAN	FIGURE 1

NOT TO SCALE

MOREY ASSOCIATES LTD.
CONSULTING ENGINEERS

Project No. 017209

Date June 2017

AIR PHOTOGRAPH	FIGURE 2

NOT TO SCALE

MOREY ASSOCIATES LTD.
CONSULTING ENGINEERS

Project No. 017209

Date June 2017

DRAWING NOTES

1. All dimensions and elevations are in metres. Do not scale drawing.
2. This site sketch plan should be read in conjunction with the accompanying Morey Associates Ltd. report for file No. 017209.
3. This drawing is not a legal survey plan.
4. The existing ground surface elevations shown on sketch plan are relative to a local temporary benchmark, TBM = Northwest bolt (spray painted) at base of existing lightstand, located as indicated on drawing, local elevation 100.00 metres.
5. Any changes made to this plan must be verified and approved by Morey Associates Ltd.

LEGEND:

TP1 — Approximate Morey Associates Ltd. Test Pit

99.99 — Existing Ground Surface Elevation, Local Datum (see Note 4)

REFERENCE:

Base plan referenced from Google satellite images. Approximate property boundaries referenced from City of Ottawa geomaps website.

DRAWING	PROJECT
SITE SKETCH PLAN - FIGURE 3	GEOTECHNICAL INVESTIGATION PROPOSED COMMERCIAL BUILDING
LOCATION 3443 INNES ROAD INNES WARD CITY OF OTTAWA ONTARIO	CLIENT Bob, Nehme & Walid Elias

DATE	DRAWING No.	DRAWN BY	APPROX. SCALE	FILE NO.
June 2017	1 of 1	DGM	1:600	017209

MOREY ASSOCIATES LTD.
CONSULTING ENGINEERS

2672 HWY.43, PO BOX 184
KEMPTVILLE, ONTARIO
K0G 1J0

T:613.215.0605
F:613.258.0605
info@moreyassociates.com

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL BUILDING

LOCATION

3443 INNES ROAD
INNES WARD
CITY OF OTTAWA, ONTARIO

MOREY ASSOCIATES LTD.
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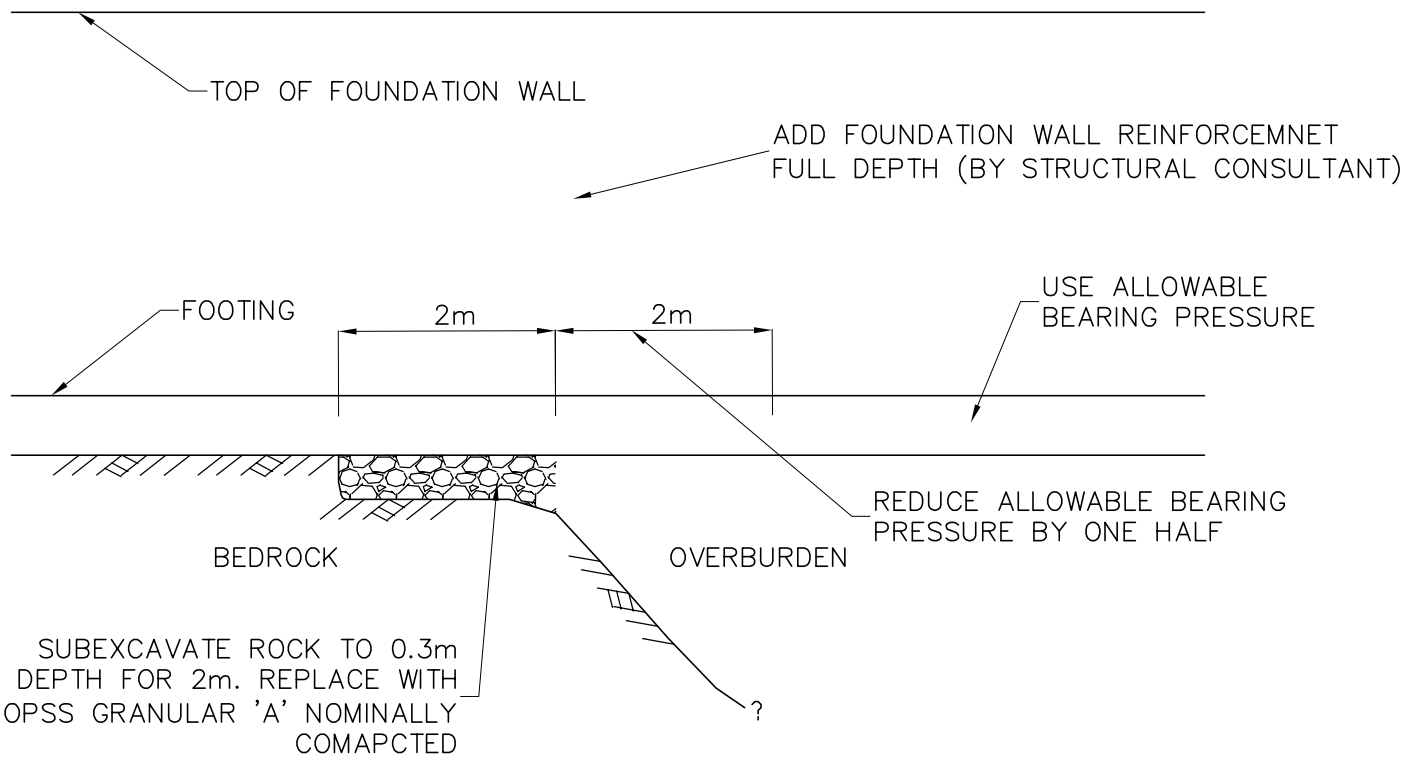
CLIENT

Bob, Nehme & Walid Elias

DATE	FILE	DRAWING
June 2017	017209	FIGURE 4

TYPICAL BEDROCK/OVERBURDEN FOOTING TRANSITION DETAIL



NOT TO SCALE

PROJECT: 017209 - Geotechnical Investigation

RECORD OF TEST PIT 1

SHEET: 1 of 1

LOCATION: See Site Sketch Plan, Figure 3

DATUM: Local

TEST PITTING DATE: June 15, 2017

DEPTH SCALE (m)	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (o, 15 30 45 60)	SHEAR STRENGTH nat. Cu (kPa) (▼, 20 40 60 80 100) / SHEAR STRENGTH rem. Cu (kPa) (▽, 20 40 60 80 100)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		99.91							
	Rubber Tire Mounted Excavator	TOPSOIL		0.00							
				99.56							
		End of test pit. Refusal to advance excavator on possible large boulder or bedrock. Test pit dry upon completion of excavating.		0.35							
1											
2											

DEPTH SCALE: 1:15

MOREY ASSOCIATES LTD. CONSULTING ENGINEERS

LOGGED: MP

CHECKED: DGM

PROJECT: 017209 - Geotechnical Investigation

LOCATION: See Site Sketch Plan, Figure 3

RECORD OF TEST PIT 2

SHEET: 1 of 1

DATUM: Local

TEST PITTING DATE: June 15, 2017

DEPTH SCALE (m)	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (o, 15 30 45 60)	SHEAR STRENGTH nat. Cu (kPa) (▼, 20 40 60 80 100) / SHEAR STRENGTH rem. Cu (kPa) (▽, 20 40 60 80 100)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		99.84							
	Rubber tire mounted excavator	Topsoil, clay, crushed stone (FILL)		0.00							
		TOPSOIL		98.94 / 0.90							
1				98.54 / 1.30							
		End of test pit. Refusal to advance excavator on possible large boulder or bedrock. Test pit dry upon completion of excavating.									
2											

DEPTH SCALE: 1:15

MOREY ASSOCIATES LTD. CONSULTING ENGINEERS

LOGGED: MP

CHECKED: DGM

PROJECT: 017209 - Geotechnical Investigation

LOCATION: See Site Sketch Plan, Figure 3

RECORD OF TEST PIT 3

SHEET: 1 of 1

DATUM: Local

TEST PITTING DATE: June 15, 2017

DEPTH SCALE (m)	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (o, 15 30 45 60)	SHEAR STRENGTH nat. Cu (kPa) (▼, 20 40 60 80 100) / SHEAR STRENGTH rem. Cu (kPa) (▽, 20 40 60 80 100)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		99.84							
	Rubber tire mounted excavator	Topsoil, sand, silt (FILL)		0.00							
		TOPSOIL		98.94 / 0.90							
1		Red brown SILTY SAND, some gravel		98.74 / 1.10							
		Grey brown silty sand, some gravel and cobbles, trace clay (GLACIAL TILL)		98.34 / 1.50							
2		End of test pit. Refusal to advance excavator on possible large boulder or bedrock. Test pit dry upon completion of excavating.		97.74 / 2.10							

DEPTH SCALE: 1:15

MOREY ASSOCIATES LTD. CONSULTING ENGINEERS

LOGGED: MP

CHECKED: DGM

PROJECT: 017209 - Geotechnical Investigation

RECORD OF TEST PIT 4

SHEET: 1 of 1

LOCATION: See Site Sketch Plan, Figure 3

DATUM: Local

TEST PITTING DATE: June 15, 2017

DEPTH SCALE (m)	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (o; 15 30 45 60)	SHEAR STRENGTH nat. Cu (kPa) (▼; 20 40 60 80 100) / SHEAR STRENGTH rem. Cu (kPa) (▽; 20 40 60 80 100)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		99.87							
	Rubber tire mounted excavator	TOPSOIL		0.00 / 99.72							
		Yellow brown SILTY SAND		0.15 / 99.57							
		Grey brown SILTY CLAY		0.30 / 99.07							
		End of test pit. Refusal to advance excavator on possible large boulder or bedrock. Test pit dry upon completion of excavating.		0.80							
1											
2											

DEPTH SCALE: 1:15

MOREY ASSOCIATES LTD. CONSULTING ENGINEERS

LOGGED: MP

CHECKED: DGM

PROJECT: 017209 - Geotechnical Investigation

LOCATION: See Site Sketch Plan, Figure 3

RECORD OF TEST PIT 5

SHEET: 1 of 1

DATUM: Local

TEST PITTING DATE: June 15, 2017

DEPTH SCALE (m)	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (o, 15 30 45 60)	SHEAR STRENGTH nat. Cu (kPa) (▼, 20 40 60 80 100) / SHEAR STRENGTH rem. Cu (kPa) (▽, 20 40 60 80 100)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		99.82							
	Rubber tire mounted excavator	TOPSOIL		0.00 / 99.62							
		Grey brown SILTY SAND		0.20 / 99.22							
		End of test pit. Refusal to advance excavator on possible large boulder or bedrock. Test pit dry upon completion of excavating.		0.60							
1											
2											

DEPTH SCALE: 1:15

MOREY ASSOCIATES LTD. CONSULTING ENGINEERS

LOGGED: MP

CHECKED: DGM

APPENDIX A

LABORATORY SOIL CORROSSIVTY TESTING RESULTS

Certificate of Analysis

eurofins | Environment Testing

Client:	Morey Associates 2672 Highway 43 Kemptville, ON K0G 1J0	Report Number:	1709850
Attention:	Mr. Dan Morey	Date Submitted:	2017-06-19
PO#:		Date Reported:	2017-06-27
Invoice to:	Morey Associates	Project:	017209
		COC #:	183731

Dear Dan Morey:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: Addrine Thomas 2017.06.27 12:40:17 -04'00'

Addrine Thomas
Team Leader, Inorganics

All analysis is completed in Ottawa, Ontario (unless otherwise indicated).

Eurofins Ottawa is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on our CALA scope of accreditation. It can be found at http://www.cala.ca/scopes/2602.pdf.

Eurofins(Ottawa) is certified and accredited for specific parameters by OMAFRA, Ontario Ministry of Agriculture, Food and Rural Affairs (for farm soils). Licensed by Ontario MOE for specific tests in drinking water.

Eurofins(Mississauga) is accredited for specific parameters by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required.

Certificate of Analysis

eurofins | Environment Testing

Client: Morey Associates
2672 Highway 43
Kemptville, ON
K0G 1J0
Attention: Mr. Dan Morey
PO#:
Invoice to: Morey Associates

Report Number: 1709850
Date Submitted: 2017-06-19
Date Reported: 2017-06-27
Project: 017209
COC #: 183731

				Lab I.D.	1298979
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2017-06-15
				Sample I.D.	TP3 1.3M
Group	**Analyte**	**MRL**	**Units**	**Guideline**	
Agri. - Soil	pH	2.0			8.0
	SO4	0.01	%		<0.01
General Chemistry	Cl	0.002	%		<0.002
	Electrical Conductivity	0.05	mS/cm		0.08
	Resistivity	1	ohm-cm		12500

Guideline = *** = Guideline Exceedence**

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).
Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

eurofins | Environment Testing

Client: Morey Associates
2672 Highway 43
Kemptville, ON
K0G 1J0
Attention: Mr. Dan Morey
PO#:
Invoice to: Morey Associates

Report Number: 1709850
Date Submitted: 2017-06-19
Date Reported: 2017-06-27
Project: 017209
COC #: 183731

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 328742 **Analysis/Extraction Date** 2017-06-21 **Analyst** C F			
Method AG SOIL			
SO4	<0.01 %	110	70-130
Run No 328767 **Analysis/Extraction Date** 2017-06-22 **Analyst** C F			
Method C CSA A23.2-4B			
Chloride		98	90-110
Run No 328873 **Analysis/Extraction Date** 2017-06-26 **Analyst** MAG			
Method Ag Soil			
pH	6.0	101	90-110
Method Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	99	85-115
Method Resistivity - soil			
Resistivity	<1 ohm-cm		

Guideline = *** = Guideline Exceedence**
All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).
Results relate only to the parameters tested on the samples submitted.
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MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range