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Structural • Environmental •
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**REPORT ON**

**GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL
DEVELOPMENT 1994 ST. JOSEPH
BOULEVARD, ORLEANS CITY OF
OTTAWA, ONTARIO**

Project # 190361

Submitted to:

M.J. Pulickal Holdings Inc.
1475 York Mills Drive
Ottawa, Ontario
K4A 2N0

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June 21, 2019 190361

M.J. Pulickal Holdings Inc.
1475 York Mills Drive
Ottawa, Ontario
K4A 2N0

RE: GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
1994 ST. JOSEPH BOULEVARD
CITY OF OTTAWA, ONTARIO

Dear Sirs:

This report presents the results of a geotechnical investigation carried out for the above noted proposed commercial development. The purpose of the investigation was to identify the subsurface conditions at the site based on a limited number of boreholes. Based on the factual information obtained, Kollaard Associates Inc. was to provide recommendations and guidelines on the geotechnical engineering aspects of the project design; including construction considerations, which could influence design decisions.

**BACKGROUND INFORMATION AND SITE GEOLOGY**

The subject site for this assessment consists of a property located at civic address 1994 St. Joseph Boulevard, in the City of Ottawa, Ontario (see Key Plan, Figure 1). The site consists of about 0.14 hectares (0.36 acres) of land located on the south side of St. Joseph Boulevard, about 93 metres east of the intersection of Jeanne-d'Arc Boulevard South and St. Joseph Boulevard, in Orleans, City of Ottawa, Ontario.

It is understood that plans are being prepared to construct a commercial development at the site. It is understood that the proposed building will be two storey and will be of steel framed construction with a conventional cast in place concrete foundation and a floating slab. The proposed building will be serviced by municipal sewer and water supply. The proposed development will be accessed by local residential roadways. Surface drainage for the proposed development will be by means of swales, catch basins and storm sewers.

Surrounding land use is currently mixed residential and commercial development. The site is bordered on the north by St. Joseph Boulevard, on the east by a commercial development (Dairy Queen and Cash Money Mart), on the west by a Petro Canada Service Station and on the south by a multi-unit residential apartment building with an asphaltic surfaced parking lot.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by fine textured glaciomarine deposits. Bedrock geology maps indicate that the bedrock underlying the site consists of limestone of the Ottawa Formation of dolomite and limestone of the Oxford Formation.

The local topography is mostly flat lying with a gentle slope from south to north across the property. The regional topography slopes north towards the Ottawa River located approximately 2.2 kilometres from the subject site.

**Site Geology**

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by fine textured glaciomarine deposits. Bedrock geology maps indicate that the bedrock underlying the site consists of limestone of the Ottawa Formation of dolomite and limestone of the Oxford Formation.

Based on a review of overburden thickness mapping for the site area, the overburden is estimated to be between about 55 to 61 metres in thickness above bedrock.

**PROCEDURE**

The field work for this investigation was carried out between June 6 and 7, 2019 in conjunction with an environmental site assessment at the site. The field work for the geotechnical report exclusive from the environmental assessment consisted of the placement of four boreholes, numbered BH1 to BH4 which were put down at the site using a rubber tire mounted drill rig equipped with a hollow stem auger owned and operated by CCC Group of Ottawa, Ontario.

Sampling of the overburden materials encountered at the borehole location was carried out at regular 0.75 metre depth intervals using a 50 millimetre diameter drive open conventional split spoon sampler in conjunction with standard penetration testing (ASTM D-1586 – Penetration Test and Split Barrel Sampling of Soils) and in situ vane shear testing (ASTM D-2573 Standard Test Method for Field Shear Test in Cohesive Soil). Each of the boreholes was advanced to depths of about 4.3 to 8.2 metres below the existing ground surface using 200 mm hollow stem augers. Borehole BH2 was continued to a depth of 33.5 metres below the existing ground surface as a probe hole using dynamic cone penetration testing. The soils were classified using the Unified Soil Classification System.

The subsurface soil conditions at the boreholes were identified based on visual examination of the samples recovered (ASTM D2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), and standard penetration tests (ASTM D-1586) as well as laboratory test results on select samples. Groundwater conditions at the borehole was noted at the time of drilling. Standpipes were installed at BH1 and BH3 for subsequent ground water level monitoring. The boreholes were loosely backfilled with the auger cuttings upon completion of drilling.

One soil sample was delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack on concrete and corrosivity to buried steel.

One soil sample (BH3 – SS6 - 3.05 - 3.66) was submitted for Atterberg Limits (D4318) and Moisture Content (ASTM D2216). The soils were classified using the Unified Soil Classification System.

The field work was supervised throughout by a member of our engineering staff who located the boreholes in the field, logged the boreholes and cared for the samples obtained. A description of the subsurface conditions encountered at the boreholes are given in the attached Record of Borehole Sheets. The results of the laboratory testing of the soil samples are presented in the Laboratory Test Results section and Attachment A following the text in this report. The approximate location of the boreholes are shown on the attached Site Plan, Figure 2.

## SUBSURFACE CONDITIONS

### General

As previously indicated, a description of the subsurface conditions encountered at the boreholes is provided in the attached Record of Borehole Sheets following the text of this report. The borehole logs indicate the subsurface conditions at the specific drill locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at locations other than borehole locations may vary from the conditions encountered at the boreholes.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification was in general completed by visual-manual procedures in accordance with ASTM 2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) with select samples being classified by laboratory testing in accordance with ASTM 2487. Classification and identification of soil involves judgement and Kollaard Associates Inc. 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 on the date the observations were noted in the report and on the borehole 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 boreholes.

### Fill

Beneath the asphaltic concrete at BH4 and from the surface at BH1, BH2 and BH3, a layer of grey crushed stone ranging in thickness from about 200 to 320 millimetres was encountered at the boreholes. Following the asphaltic concrete and grey crushed stone layers, fill materials consisting of grey silty sand, grey silty clay, yellow brown silty sand and yellow brown sand and gravel with a trace to some asphaltic concrete, organics, wood, concrete debris and glass was encountered. The fill materials ranged in thickness from about 0.48 to 3.15 metres and were encountered to depths of about 0.8 to 3.35 metres below the existing ground surface. The fill materials were fully penetrated at all borehole locations.

**Topsoil**

From the surface at borehole BH4, a layer of topsoil with a thickness of about 0.1 metres was encountered. The material was classified as topsoil based on the colour and the presence of organic materials. The identification of the topsoil layer is for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustainable plant growth.

**Silty Clay**

Beneath the fill materials and topsoil, a deposit of red brown and/or grey silty clay was encountered at all of the boreholes. In situ vane shear tests carried out in the silty clay deposit gave undrained shear strength values ranging from about 49 kilopascals to 63 kilopascals. The results of the in situ vane shear testing and tactile examination carried out for the silty clay material indicate that the silty clay is firm to stiff in consistency. Borehole BH2 was advanced through the silty clay by dynamic cone penetration testing to refusal at about 33.47 metres below the existing ground surface. Based on the increase in the standard cone penetration values in blow counts per 300 mm obtained at BH2 at a depth of about 30 metres below the existing ground surface, it is considered that the silty clay deposit layer is about 27.4 metres in thickness. Borehole BH2 was terminated on practical refusal to cone penetration on a boulder or cobbles at a depth of about 33.47 metres below the existing ground surface.

The results of Atterberg Limits tests and moisture content (ASTM D422) conducted on one soil sample (BH3 – SS6 - 3.05 - 3.65 metres) of the silty clay are presented in the following table and in Attachment A at the end of the report. The tested silty clay sample classifies as high plasticity in accordance with the Unified Soil Classification System. The results of the laboratory testing are located in Attachment A.

**Table I – Atterberg Limit and Water Content Results**

| Sample | Depth(metres) | LL (%) | PL (%) | PI (%) | W (%) |
|---|---|---|---|---|---|
| BH3-SS6 | 3.05 - 3.65 | 72.2 | 28.3 | 43.9 | 83.1 |

**LL: Liquid Limit PL: Plastic Limit PI: Plasticity Index w: water content**
CH: Inorganic High Plastic Soils

**Glacial Till**

Borehole BH2 was advanced through the silty clay by dynamic cone penetration testing to refusal at about 33.47 metres below the existing ground surface. The dynamic cone penetration test at BH2 gave values ranging from WH to 93 blows per 0.3 metres. Based on the increase in the standard cone penetration values in blow counts per 300 mm obtained at BH2 at a depth of about 30 metres below the existing ground surface, it is considered that the silty clay deposit layer is about 27.4 metres in thickness. Borehole BH2 was terminated with practical refusal to cone penetration on either a boulder or cobbles at a depth of about 33.47 metres below the existing ground surface.

It is considered likely that the increase in blow count at about 8.2 metres depth indicates the possible presence of glacial till materials.

**Groundwater**

Groundwater seepage was encountered within each of the boreholes at the time of drilling at depths ranging between 1.2 and 2.1 metres below the existing ground surface. On June 10, 2019, groundwater was measured within standpipes installed within boreholes BH1 and BH3 at depths ranging between 1.2 to 3.2 metres below the existing ground surface. It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

**Corrosivity on Reinforcement and Sulphate Attack on Portland Cement**

The results of the laboratory testing of a soil sample for submitted for chemistry testing related to corrosivity is summarized in the following table.

| Item | Threshold of Concern | Test Result | Comment |
|---|---|---|---|
| Chlorides (Cl) | Cl > 0.04 % | 0.004 | Negligible |
| pH | 5.0 < pH | 7.15 | Basic<br>Negligible concern |
| Resistivity | R < 20,000 ohm-cm | 3180 | Corrosive |
| Sulphates ($SO_4$) | $SO_4$ > 0.1% | 0.0128 | Negligible concern |

The results were compared with Canadian Standards Association (CSA) Standards A23.1 for sulphate attack potential on concrete structures and posses a "negligible" risk for sulphate attack on concrete materials and accordingly, conventional GU or MS Portland cement may be used in the construction of the proposed concrete elements.

The pH value for the soil sample was reported to be at 7.15, indicating a durable condition against corrosion. This value was evaluated using Table 2 of Building Research Establishment (BRE) Digest 362 (July 1991).The pH is greater than 5.5 indicating the concrete will not be exposed to attack from acids.

The chloride content of the sample was also compared with the threshold level and present negligible concrete corrosion potential.

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 0.75 loss-oz./$ft^2$/yr (3180 ohm-cm). 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 to highly aggressive corrosion rate to reinforcement steel within below grade concrete walls. Based on the chemical test results, Type GU General use Hydraulic Cement may be used for this proposed development. Special protection is required for reinforcement steel within the concrete walls.

## GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

### 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.

### Foundation Excavation

Any excavation for the proposed structures will likely be carried out through fill material to bear on the native silty clay subgrade. The sides of the excavation should be sloped in accordance with the requirements of Ontario Regulation 213/91, s. 226 under the Occupational Health and Safety Act. According to the Act, the native soils at the site can be classified as Type 2 soil, however this classification should be confirmed by qualified individuals as the site is excavated and if necessary, adjusted.

It is expected that the side slopes of the excavation will be stable in the short term provided the walls are sloped at 1H:1V through the fill materials to 1.2 metres or less from the bottom of the excavation and provided no excavated materials are stockpiled within 3 metres of the top of the excavation.

**Effect of Foundation Excavation on Adjacent Structures and City of Ottawa Services**

As previously indicated, the proposed foundation excavation will be carried out through fill, topsoil and native silty clay. There will be no bedrock excavation or removal. As such, there will be no excavation processes which could contribute to vibration which could potentially damage adjacent City of Ottawa Services.

**Ground Water in Excavation and Construction Dewatering**

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

Ground water was observed at between about 1.0 and 3.2 metres below the ground surface at time of drilling and measured at 1.2 and 3.2 metres below the ground surface in the stand pipes installed within the boreholes. It is considered that the groundwater level at 1.0 to 1.2 metres may be trapped water within fill materials or the native soils at the site and that the groundwater level measured at 3.2 metres below the existing ground surface is reflective of the native conditions. It is considered that the excavation for the new building at the site should not extend below the ground water level. As such a permit to take water is will not be required prior to excavation.

**Effect of Dewatering of Foundation or Site Services Excavations on Adjacent Structures**

Since the existing ground water level at the site is will be below the expected underside of footing elevation, dewatering of the excavation will not remove water from historically saturated soils. The closest building is located about 10 metres east of the subject site. As such dewatering of the foundation or site services excavations, if required, will not have a detrimental impact on the adjacent structures.

**Foundation for Proposed Commercial Building**

**Foundation Design and Bearing Capacity**

As previously indicated, the subsurface conditions at the site encountered at the boreholes advanced during the investigation consisted of asphaltic concrete, crushed stone and deleterious fill materials, silty sand followed by native silty clay. With the exception of the fill materials, the subsurface conditions encountered at the test holes advanced during the investigation are suitable for the support of the proposed building on conventional spread footing foundations placed on a native subgrade or on engineered fill placed on the native subgrade. The excavations for the foundation should be taken through any topsoil or otherwise deleterious material to expose the native, undisturbed silty clay. It is suggested that the building be founded either directly on the underlying silty clay or on engineered fill placed on the silty clay.

The allowable bearing pressure for any footings depends on the depth of the footings below original ground surface, the width of the footings, and the height above the original ground surface of any landscape grade raise adjacent to the building foundation.

For predictable performance of the proposed foundations, all existing fill materials and any deleterious materials should be removed from within the proposed foundation areas to expose the native silty clay.

Strip and pad footings, a minimum 0.5 metres in width bearing on the native undisturbed silty clays at a founding depth of a minimum of 1.8 metres below the original ground surface and above the groundwater level may be designed using a maximum allowable bearing pressure of 100 kilopascals for serviceability limit states and 250 kilopascals for the factored ultimate bearing resistance for the design of conventional strip footings or pad footings, founded on native silty clay or on a suitably constructed engineered pad placed on the native silty clay.

The above allowable bearing pressure is subject to a maximum grade raise of 0.5 metres above the existing ground surface and to maximum strip and pad footing widths of 1.5 metres.

Provided that any loose and disturbed soil is removed from the bearing surfaces prior to pouring concrete, the total and differential settlement of the footings should be less than 25 millimetres and 20 millimetres, respectively.

**Engineered Fill**

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 granular material meeting Ontario Provincial Standards Specifications (OPSS) requirements for Granular A or Granular B Type II and should be compacted in maximum 300 millimetre thick loose lifts to at least 98 percent of the standard Proctor maximum dry density. It is considered that the engineered fill should be compacted using dynamic compaction with a large diameter vibratory steel drum roller or diesel plate compactor. If a diesel plate compactor is used, the lift thickness may need to be restricted to less than 300 mm to achieve proper compaction. Compaction should be verified by a suitable field compaction test method.

To allow the spread of load beneath the footings, the engineered fill should extend out 0.5 metres horizontally from the edges of the footing then down and out at 1 horizontal to 1 vertical, or flatter. The excavations for the proposed residential building should be sized to accommodate this fill placement.

The first lift of engineered fill material should have a thickness of 300 millimetres in order to protect the subgrade during compaction. It is considered that the placement of a geotextile fabric between the engineered fill and the subgrade is not necessary where granular materials meeting the grading requirements for OPSS Granular B Type II or OPSS Granular A are placed on a silty clay subgrade above the normal ground water level. It is recommended that trucks are not used to place the engineered fill on the subgrade. The fill should be dumped at the edge of the excavation and moved into place with a tracked bulldozer or excavator.

The native silty clay soils at this site will be sensitive to disturbance from construction operations and from rainwater or snowmelt, and 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.

**Frost Protection**

In general, all exterior foundation elements and those in any unheated parts of the proposed buildings 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 for frost protection purposes.

**Foundation Wall Backfill and Drainage**

The native soils encountered at this site are considered to be frost susceptible. As such, to prevent possible foundation frost jacking due to frost adhesion, the backfill against any unheated or insulated walls or isolated walls or piers should consist of free draining, non-frost susceptible material. If imported material is required, it should consist of 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 such as "System Platon" 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.

Where the 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 lifts to 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.

Provided the proposed finished floor surfaces are everywhere above the exterior finished grade, the granular materials beneath the proposed floor slab are properly compacted and provided the exterior grade is adequately sloped away from the proposed building, no perimeter foundation drainage system is required.

***Slab on Grade Support***

As stated above, it is expected that the proposed building will be founded on native silty clay or on an engineered pad placed on the native subgrade. For predictable performance of the proposed concrete floor slab all existing fill material, topsoil and any otherwise deleterious material should be removed from below the proposed floor slab 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. Any fill materials consisting of granular material, removed from the proposed concrete floor slab area, could be stockpiled for possible reuse with approval from the geotechnical engineer.

The fill materials beneath the proposed concrete floor slab on grade should consist of a minimum of 150 millimetre thickness of crushed stone meeting OPSS Granular A immediately beneath the concrete floor slab followed by sand, or sand and gravel meeting the OPSS for Granular B Type I, or crushed stone meeting OPSS grading requirements for Granular B Type II, or other material approved by the Geotechnical Engineer. The fill materials should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density.

The slab should be structurally independent from walls and columns, which are supported by the foundations. This is to reduce any structural distress that may occur as a result of differential soil movement. If it is intended to place any internal non-load bearing partitions directly on the slab-on-grade, such walls should also be structurally independent from other elements of the building founded on the conventional foundation system so that some relative vertical movement between the floor slab and foundation can occur freely.

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 placed at a grid spacing not exceeding the lesser of 25 times the slab thickness or 4.5 metres. The slab should be cut as soon as it is possible to work on the slab without damaging the surface of the slab.

Under slab drainage is not considered necessary provided that the floor slab level is everywhere above the finished exterior ground surface level. If any areas of the proposed building are to remain unheated during the winter period or under slab insulation is to be used, thermal protection of the foundation may be required. Further details on the insulation requirements could be provided, if necessary.

**Seismic Design for the Proposed Residential 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 D.

| Borehole 1 & 2 | | | | | |
|---|---|---|---|---|---|
| Layer | Description | Depth (m) | $d_i$ (m) | $S_{ui}$ (kPa) | $d_i/S_{ui}$ (m/kPa) |
| 1 | USF | 1.5 | | | |
| 2 | Silty Clay | 1.5 | 28.5 | 56.3 | 0.5 |
| 3 | Glacial Till | 30 | 1.5 | N/A | |
| | $d_c/(\mathrm{sum}(d_i/S_{ui}))$ | | | | 56.3 |

Since $S_u = 50 < 56.3 < 100$ kPa the seismic site response is Site Class D.

**Potential for Soil Liquefaction**

As indicated above, the results of the boreholes and information from geological maps indicate that the native deposits underlying the site consist of a stiff silty clay crust followed by glacial till then bedrock.

C.F.E.M. section 6.6.3.2 (6) recommends that the Bray et al. (2004) criteria be used to determine liquefaction susceptibility of fine-grained soils:

That is fine-grained soils with $PI \leq 12$ and $W_c > 0.85LL$ are susceptible to liquefaction, soils with $12 \leq PI \leq 20$ and $W_c > 0.8LL$ are moderately susceptible to liquefaction and soils with $PI > 20$ and $W_c < 0.8LL$ are not susceptible to liquefaction.

Seed et al. (2003) proposed liquefaction susceptibility criteria that are similar to those by Bray et al. (2004) except that they include slightly different Wc / LL ratios and include constraints on LL. The criteria by Seed et al. (2003) are described by three zones on the Atterberg limits chart, which are bounded by the following PI and LL values: Zone A soils have PI ≤ 12 and LL ≤ 37 and are considered potentially susceptible to “classic cyclically induced liquefaction” if the water content is greater than 80% of the LL; Zone B soils have PI ≤ 20 and LL ≤ 47 and are considered potentially liquefiable with detailed laboratory testing recommended if the water content is greater than 85% of the LL; and Zone C soils with PI > 20 or LL >47 are considered generally not susceptible to classic cyclic liquefaction, although they should be checked for potential sensitivity.

From the laboratory test results, the silty has a plasticity index PI = of 43.9 and a liquid limit of 72.2 indicating an inorganic highly plastic clay. As such the silty clay is not prone to liquefaction.

### National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.300 with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The results of the test are attached following the text of this report.

## SITE SERVICES

### *Excavation*

The excavations for the site services will be carried out through fill materials and silty clay. For the purposes of Ontario Regulation 213/91 the soils at the site can be considered to be Type 2 soil. 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.

Based on the depths at which groundwater was measured within the standpipe installed in boreholes BH1 and BH3, significant groundwater flow into any excavation is unlikely. Any groundwater inflow into the service trenches should be handled by pumping from sumps from within the excavations.

***Pipe Bedding and Cover Materials***

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 sub-excavation 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 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 or Granular B Type I (with a maximum particle size of 25 millimetres).

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.

***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 existing or future pavement areas, acceptable native materials should be used as backfill between the pavement subgrade level and the depth of seasonal frost penetration (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 roadway.

Where native backfill is used, it should match the native materials exposed on the trench walls. 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 existing or future driveway 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. If the native material is not suitable for backfill, imported granular material may have to be used. If imported granular materials are used, suitable frost tapers should be used OPSD 802.013.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the parking areas, sidewalks, etc., the trench should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. The specified density may be reduced to 90 percent where the trench backfill is not located or in close proximity to existing or future roadways, driveways, sidewalks, or any other type of permanent structure.

## ACCESS ROADWAY PAVEMENTS

Based on the results of the boreholes, the subsurface conditions in the access roadway and parking areas consist of existing asphaltic concrete followed by grey crushed stone overlying silty sand/silty clay fill materials overlying native silty clay. For predictable performance of the pavement structures, it is considered that all of the existing asphaltic concrete will have to be removed in preparation for pavement construction at this site. It is considered that any granular crushed stone fill material that is free of topsoil or organic debris may be stockpiled and upon approval by the engineer used to raise the subgrade of the access roadway and parking areas to the proposed underside of access roadway and subbase elevation of the parking lot.

Once existing asphaltic concrete and granular crushed stone and any deleterious material has been removed, the exposed sub-grade should be inspected and approved by geotechnical personnel and any soft areas evident should be sub-excavated and replaced with suitable earth borrow or granular crushed stone approved by the geotechnical engineer. The sub-grade should be shaped and crowned to promote drainage of the roadway area granular. Following approval of the preparation of the sub-grade, the pavement granulars may be placed.

For any areas of the site that require the sub-grade to be raised to proposed pavement sub-grade level, the material used should consist of OPSS select sub-grade material or OPSS Granular B Type I or Type II. Recycled crushed concrete meeting the grading specifications for Granular B Type II could also be used. Materials used for raising the sub-grade to proposed roadway area sub-grade 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 pavement areas subject to cars and light trucks the pavement should consist of:

50 millimetres of Superpave 12.5 asphaltic concrete over
150 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase over
(50 or 100 millimetre minus crushed stone)
Non-woven geotextile fabric (4 oz/sy) such as Terrafix 270R or Thrace-Ling 130EX or approved alternative.

Performance grade PG 58-34 asphaltic concrete should be specified. 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 above pavement structures will be adequate on an acceptable sub-grade, that is, one where any roadway fill and service trench backfill has been adequately compacted. If the roadway sub-grade 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 roadway subgrade surface and the granular subbase material. The adequacy of the design of the pavement thickness should be assessed by the geotechnical personnel at the time of construction.

## CONSTRUCTION CONSIDERATIONS

It is suggested that the final design drawings for the project, including the proposed site grading plan, be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended and to re-evaluate the guidelines provided in the report with respect to the actual project plans. Items such as actual foundation wall/column loads, etc could have significant impacts on foundation type, frost protection requirements, etc.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed development 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 foundation areas and any engineered fill areas for the proposed commercial building should be inspected by Kollaard Associates Inc. 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 to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for the site services, access roadways and driveway should be inspected and approved by geotechnical personnel. In situ density testing should be carried out on the service pipe bedding and backfill and the pavement granular materials to ensure the materials meet the specifications from a compaction point of view.

The native silty clay deposits at this site will be sensitive to disturbance from construction operations, from rainwater or snow melt, and 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.

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 services to you, please do not hesitate to contact our office.

Regards,
Kollaard Associates Inc.

LICENSED PROFESSIONAL ENGINEER
June 21, 2019
S.E. deWit
100079612
PROVINCE OF ONTARIO

Dean Tataryn, B.E.S., EP.

Steve DeWit, P.Eng.

Attachments: Table I - Record of Boreholes
Key Plan, Figure 1
Site Plan, Figure 2
Laboratory Test Results for Sulphate, Resistivity and pH
Attachment A – Stantec Laboratory Test Results for Soils
Attachment B - National Building Code Seismic Hazard Calculation

## APPENDIX A – SUMMARY OF GEOTECHNICAL RECOMMENDATIONS

This report provides geotechnical recommendations under the Headings: Geotechnical Guidelines and Recommendations; Foundation For Proposed Residential Building; Site Services; Access Roadway Pavements; Construction Considerations:

These geotechnical recommendations include:
Foundation Design
Allowable Bearing Capacity
Settlement
Subgrade preparation
Engineered Fill and Compaction
Frost Protection
Foundation Drainage
Foundation Backfill
Floor Slab
Seismic Design
Excavation for Services and Sewers
Bedding and Cover
Trench Backfill
Subgrade Preparation for Pavements
Pavement Structures
Pavement Placement and compaction
Inspection Requirements.

# RECORD OF BOREHOLE BH1

**PROJECT:** Proposed Commercial Development
**CLIENT:** M.J. Pulickal Holdings Inc.
**LOCATION:** 1994 St. Joseph Boulevard, Ottawa, Ontario
**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 190361
**DATE OF BORING:** June 7, 2019
**SHEET** 1 of 1
**DATUM:** LOCAL

| DEPTH SCALE (meters) | SOIL PROFILE: DESCRIPTION | ELEV. DEPTH (M) | SAMPLES: NUMBER | TYPE | BLOWS/0.3m | UNDIST. SHEAR STRENGTH × Cu, kPa 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa 20 40 60 80 | DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90 | ADDITIONAL LAB TESTING | PIEZOMETER OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|---|
| | Ground Surface | 500.04 | | | | | | | |
| 0 | Grey crushed stone (FILL) | 0.00<br>499.84 | | | | | | | |
| | Yellow brown silty sand (FILL) | 0.20 | 1 | SS | 7 | | | | |
| 1 | | 498.84 | 2 | SS | 7 | | | | |
| | Grey silty sand, trace gravel, glass, organics (FILL) | 1.20 | 3 | SS | 2 | | | | |
| 2 | | 497.56 | 4 | SS | WH | | | | |
| 3 | Grey silty clay, trace organics and wood (FILL) | 2.48<br>496.69 | 5 | SS | 2 | | | | |
| | Firm grey SILTY CLAY | 3.35 | 6 | SS | WH | | | | |
| 4 | | | 7 | SS | WH | | | | |
| | | 495.17 | 8 | SS | WH | | | | |
| 5 | End of Borehole | 4.87 | | | | | | | Water observed in borehole at approximately 1.2 metres below the existing ground surface on June 7, 2019. Water measured in the standpipe at about 1.2 metres below the existing ground surface, June 10, 2019. |
| 6 | | | | | | | | | |
| 7 | | | | | | | | | |
| 8 | | | | | | | | | |

**DEPTH SCALE:** 1 to 50
**BORING METHOD:** Power Auger
**AUGER TYPE:** 200 mm Hollow Stem
**LOGGED:** DT
**CHECKED:** SD

# RECORD OF BOREHOLE BH2

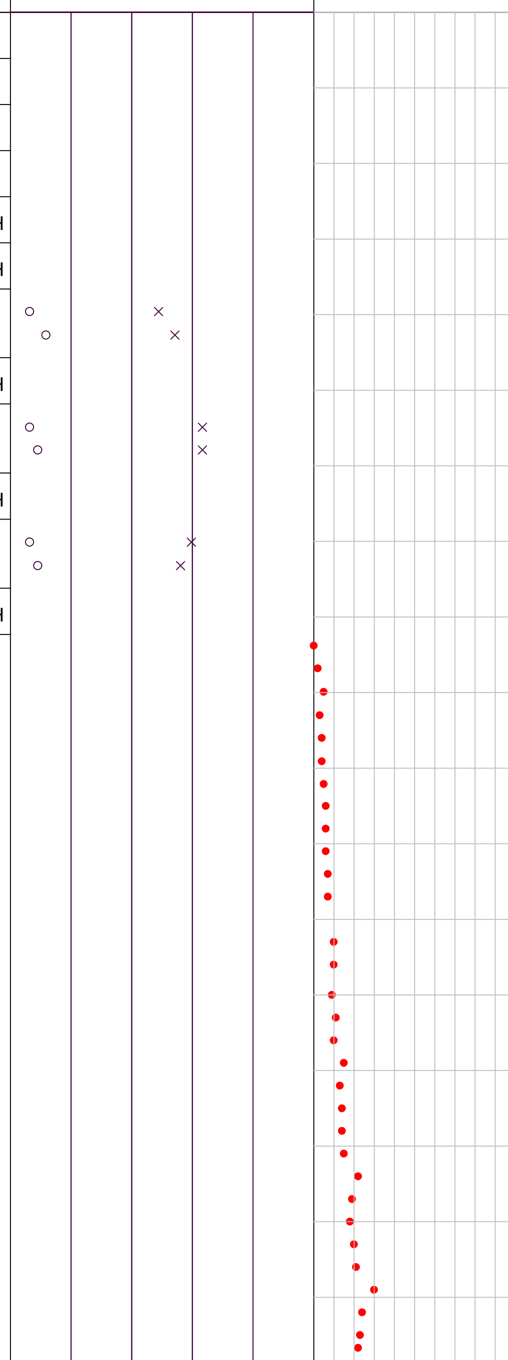
**PROJECT:** Proposed Commercial Development
**CLIENT:** M.J. Pulickal Holdings Inc.
**LOCATION:** 1994 St. Joseph Boulevard, Ottawa, Ontario
**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 190361
**DATE OF BORING:** June 6, 2019

**DATUM:** LOCAL

| DEPTH SCALE (meters) | SOIL PROFILE: DESCRIPTION | ELEV. DEPTH (M) | SAMPLES: NUMBER | TYPE | BLOWS/0.3m |
|---|---|---|---|---|---|
| | Ground Surface | 500.11 | | | |
| 0 | Grey crushed stone (FILL) | 0.00 | 1 | SS | 5 |
| | Yellow brown silty sand, trace organics, clay and wood (FILL) | | 2 | SS | 5 |
| 1–2 | | | 3 | SS | 2 |
| | | | 4 | SS | 3 |
| | Firm grey brown SILTY CLAY | 497.48<br>2.63 | 5 | SS | WH |
| 3 | Grey SILTY CLAY | 497.06<br>3.05 | 6 | SS | WH |
| 4–5 | | | 7 | SS | WH |
| 6 | | | 8 | SS | WH |
| 7–8 | | | 9 | SS | WH |
| 8–17 | Borehole continued as Probe Hole, probably grey SILTY CLAY | 491.88<br>8.23 | | | |

UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80
REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80
DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90
ADDITIONAL LAB TESTING



PIEZOMETER OR STANDPIPE INSTALLATION

Water observed in borehole at approximately 1.4 metres below the existing ground surface on June 6, 2019.

**DEPTH SCALE:** 1 to 100
**BORING METHOD:** Power Auger
**AUGER TYPE:** 200 mm Hollow Stem
**LOGGED:** DT
**CHECKED:** SD

# RECORD OF BOREHOLE BH2

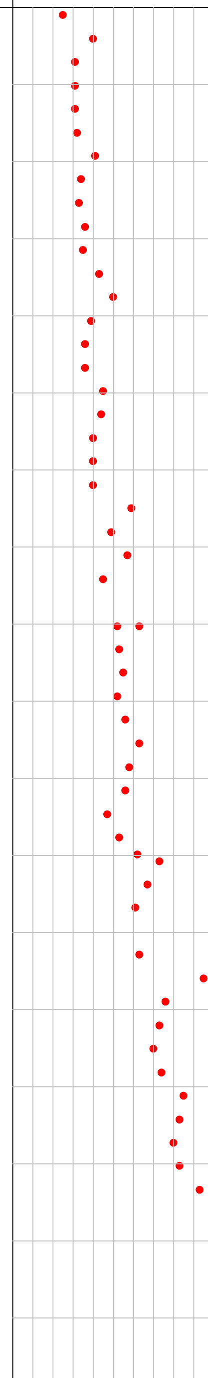
**PROJECT:** Proposed Commercial Development
**CLIENT:** M.J. Pulickal Holdings Inc.
**LOCATION:** 1994 St. Joseph Boulevard, Ottawa, Ontario
**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 190361
**DATE OF BORING:** June 6, 2019

**DATUM:** LOCAL

| DEPTH SCALE (meters) | SOIL PROFILE: DESCRIPTION | STRATA PLOT | ELEV. DEPTH (M) | SAMPLES: NUMBER | TYPE | BLOWS/0.3m | UNDIST. SHEAR STRENGTH × Cu, kPa 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa 20 40 60 80 | DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90 | ADDITIONAL LAB TESTING | PIEZOMETER OR STANDPIPE INSTALLATION |
|---|---|---|---|---|---|---|---|---|---|---|
| 18–30 | | | 470.11 | | | | | | | |
| 30 | Probably grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL) | | 30.00 | | | | | | | |
| 33.47 | | | 466.64 | | | | | | | |
| | End of Borehole, refusal on large boulder or bedrock | | 33.47 | | | | | | | |

Depth scale: 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35



**DEPTH SCALE:** 1 to 100
**BORING METHOD:** Power Auger
**AUGER TYPE:** 200 mm Hollow Stem
**LOGGED:** DT
**CHECKED:** SD

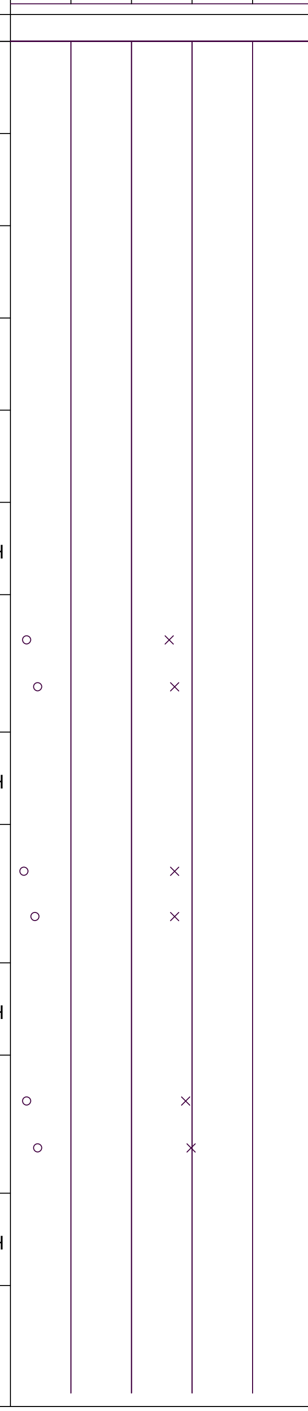
# RECORD OF BOREHOLE BH3

**PROJECT:** Proposed Commercial Development
**CLIENT:** M.J. Pulickal Holdings Inc.
**LOCATION:** 1994 St. Joseph Boulevard, Ottawa, Ontario
**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 190361
**DATE OF BORING:** June 6, 2019
**SHEET** 1 of 1
**DATUM:** LOCAL

| DEPTH SCALE (meters) | SOIL PROFILE: DESCRIPTION | ELEV. DEPTH (M) | SAMPLES: NUMBER | TYPE | BLOWS/0.3m |
|---|---|---|---|---|---|
| | Ground Surface | 500.19 | | | |
| 0 | Grey crushed stone, trace organics (FILL) | 0.00 | | | |
| | Grey silty clay, trace gravel and wood (FILL) | 499.89<br>0.30 | 1 | SS | 10 |
| | | | 2 | SS | 3 |
| 1 | | | 3 | SS | 19 |
| | Yellow brown sand and gravel, trace organics (FILL) | 498.39<br>1.80 | 4 | SS | 15 |
| 2 | Grey SILTY CLAY | 497.69<br>2.50 | 5 | SS | 2 |
| 3 | | | 6 | SS | WH |
| 4 | | | | | |
| 5 | | | 7 | SS | WH |
| 6 | | | 8 | SS | WH |
| 7 | | | | | |
| 8 | | | 9 | SS | WH |
| | End of Borehole | 491.97<br>8.22 | | | |

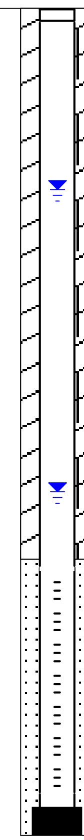
UNDIST. SHEAR STRENGTH × Cu, kPa ×
REM. SHEAR STRENGTH ○ Cu, kPa ○




DYNAMIC CONE PENETRATION TEST
blows/300 mm
10 30 50 70 90

ADDITIONAL LAB TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



Water observed in borehole at approximately 1.2 metres below the existing ground surface on June 6, 2019. Water measured in the standpipe at about 3.2 metres below the existing ground surface, June 10, 2019.

**DEPTH SCALE:** 1 to 50
**BORING METHOD:** Power Auger
**AUGER TYPE:** 200 mm Hollow Stem
**LOGGED:** DT
**CHECKED:** SD

# RECORD OF BOREHOLE BH4

**PROJECT:** Proposed Commercial Development
**CLIENT:** M.J. Pulickal Holdings Inc.
**LOCATION:** 1994 St. Joseph Boulevard, Ottawa, Ontario
**PENETRATION TEST HAMMER:** 63.5kg, Drop, 0.76mm

**PROJECT NUMBER:** 190361
**DATE OF BORING:** June 7, 2019
**SHEET** 1 of 1
**DATUM:** LOCAL

| DEPTH SCALE (meters) | SOIL PROFILE: DESCRIPTION | ELEV. DEPTH (M) |
|---|---|---|
| | Ground Surface | 500.23 |
| 0 | TOPSOIL | 0.00 |
| | Red brown SILTY CLAY | 500.13<br>0.10 |
| 1 | Firm grey SILTY CLAY | 499.21<br>1.02 |
| | End of Borehole | 495.96<br>4.27 |

| SAMPLES: NUMBER | TYPE | BLOWS/0.3m |
|---|---|---|
| 1 | SS | 2 |
| 2 | SS | 8 |
| 3 | SS | WH |
| 4 | SS | WH |
| 5 | SS | WH |
| 6 | SS | 2 |
| 7 | SS | WH |

UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80

REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80

DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90

ADDITIONAL LAB TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Water observed in borehole at approximately 1.0 metres below the existing ground surface on June 7, 2019.

**DEPTH SCALE:** 1 to 25
**BORING METHOD:** Power Auger
**AUGER TYPE:** 200 mm Hollow Stem
**LOGGED:** DT
**CHECKED:** SD

# LIST OF ABBREVIATIONS AND TERMINOLOGY

## SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
MS manual sample
RC rock core
ST slotted tube .
TO thin-walled open Shelby tube
TP thin-walled piston Shelby tube
WS wash sample

## PENETRATION RESISTANCE

Standard Penetration Resistance, N ,

The number of blows by a 63.5 kg hammer dropped 760 millimeter required to drive a 50 mm drive open . sampler for a distance of 300 mm. For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

Dynamic Penetration Resistance

The number .of blows by a 63.5 kg hammer dropped 760 mm to drive a 50 mm diameter, 60° cone attached to 'A' size drill rods for a distance of 300 mm.

WH

_Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drih rig.

PM

Sampler advanced by manual pressure.

## SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH sieve and hydrometer analysis
U unconfined compression test
Q undrained triaxial test
V field vane, undisturbed and remolded shear strength

## SOIL DESCRIPTIONS

| Relative Density | 'N' Value |
|---|---|
| Very Loose | 0 to 4 |
| Loose | 4 to10 |
| Compact | 10 to 30 |
| Dense | 30 to 50 |
| Very Dense | over 50 |

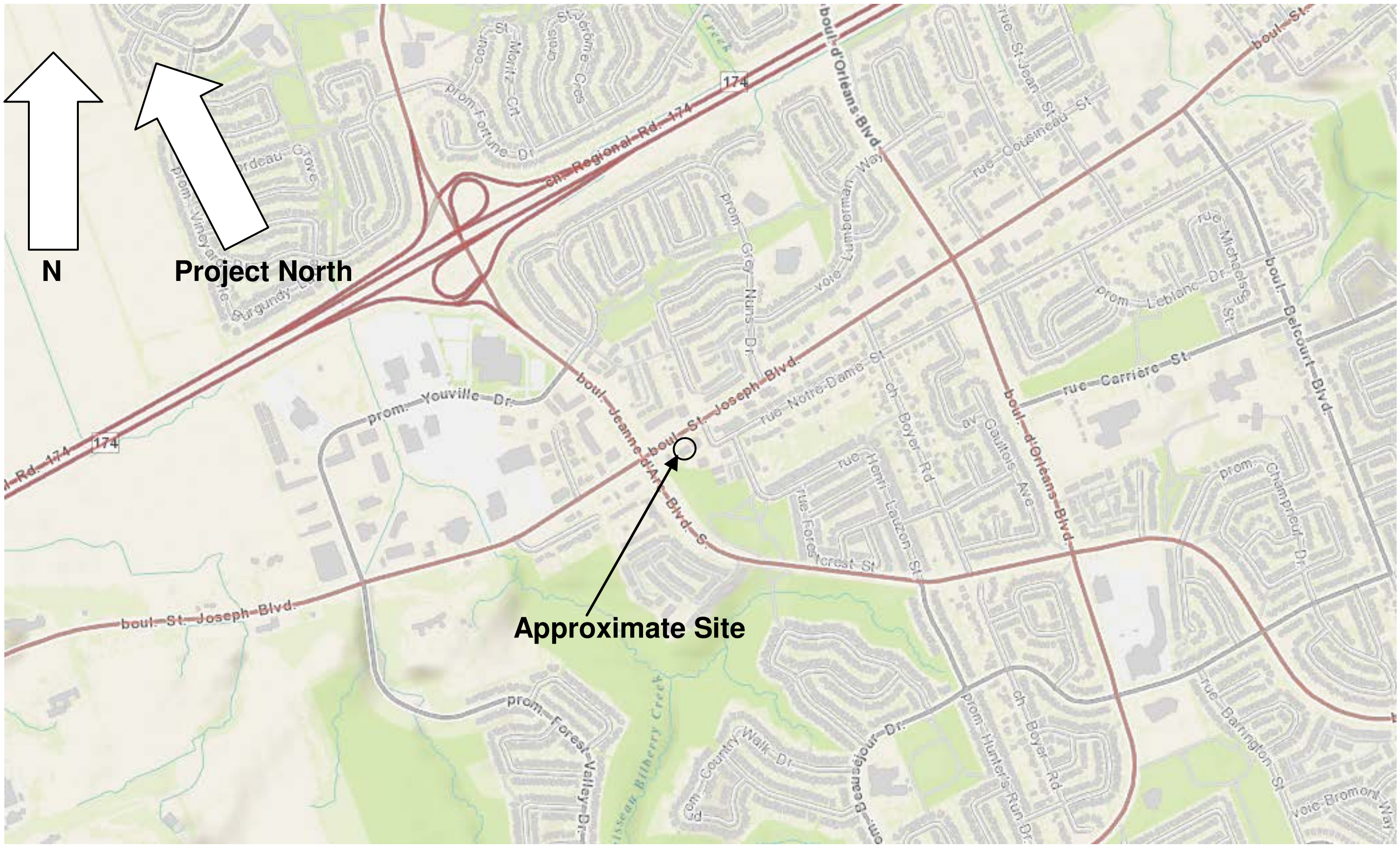
| Consistency | Undrained Shear Strength (kPa) |
|---|---|
| Very soft | 0 to 12 |
| Soft | 12 to 25 |
| Firm | 25 to 50 , |
| Stiff | 50 to100 |
| Very Stiff | over100 |

## LIST OF COMMON SYMBOLS

Cu undrained shear strength
e void ratio
Cc compression index
Cv coefficient of consolidation
k coefficient of permeability
Ip plasticity index
n porosity
u pore pressure
w moisture content
wL liquid limit
Wp plastic limit
$1 effective angle of friction
r unit weight of soil
y1 unit weight of submerged soil
cr normal stress

# KEY PLAN

**FIGURE 1**




**NOT TO SCALE**

Kollaard Associates
Engineers

Project No. 190361

Date June 2019




*DRAWING NUMBER:*
SITE PLAN, FIGURE 2

LEGEND:

BH1 APPROXIMATE BOREHOLE LOCATION

REFERENCE: PLAN SUPPLIED BY
CITY OF OTTAWA EMAPS.

SPECIAL NOTE: THIS DRAWING TO
BE READ IN CONJUNCTION WITH
THE ACCOMPANYING REPORT.

| REV. | NAME | DATE | DESCRIPTION |
|---|---|---|---|
| | | | |

Kollaard Associates
Engineers

PO, BOX 189, 210 PRESCOTT ST
KEMPTVILLE ONTARIO
K0G 1J0 FAX (613) 258–0475
http://www.kollaard.ca
(613) 860–0923
info@kollaard.ca

*CLIENT:*
M.J. PULICKAL HOLDINGS INC.

*PROJECT:*
GEOTECHNICAL INVESTIGATION FOR
PROPOSED COMMERCIAL DEVELOPMENT

*LOCATION:*
1994 ST. JOSEPH BOULEVARD
CITY OF OTTAWA, ONTARIO

| *DESIGNED BY:* –– | *DATE:* MAY 10, 2019 |
|---|---|
| *DRAWN BY:* DT | *SCALE:* N.T.S |

*KOLLAARD FILE NUMBER:*
190361

## Laboratory Test Results for Chemical Properties

Kollaard Associates (Kemptville)
ATTN: Dean Tataryn
210 Prescott Street Unit 1
P.O. Box 189
Kemptville ON K0G 1J0

Date Received: 10-JUN-19
Report Date: 17-JUN-19 14:33 (MT)
Version: FINAL

Client Phone: 613-860-0923

# Certificate of Analysis

Lab Work Order #: L2289192

Project P.O. #: NOT SUBMITTED
Job Reference: 190361
C of C Numbers:
Legal Site Desc:

Melanie Moshi
Account Manager

# ALS ENVIRONMENTAL ANALYTICAL REPORT

| Sample Details/Parameters | Result | Qualifier* | D.L. | Units | Extracted | Analyzed | Batch |
|---|---|---|---|---|---|---|---|
| L2289192-1 BH5 SS3 4'-6'<br>Sampled By: CLIENT on 07-JUN-19<br>Matrix: SOIL | | | | | | | |
| **Physical Tests** | | | | | | | |
| Conductivity | 0.314 | | 0.0040 | mS/cm | | 13-JUN-19 | R4667870 |
| % Moisture | 26.0 | | 0.10 | % | 11-JUN-19 | 12-JUN-19 | R4664114 |
| pH | 7.15 | | 0.10 | pH units | | 13-JUN-19 | R4669029 |
| Resistivity | 3180 | | 1.0 | ohm*cm | | 13-JUN-19 | |
| **Leachable Anions & Nutrients** | | | | | | | |
| Chloride | 0.00425 | | 0.00050 | % | 12-JUN-19 | 12-JUN-19 | R4667967 |
| **Anions and Nutrients** | | | | | | | |
| Sulphate | 0.0128 | | 0.0020 | % | 11-JUN-19 | 12-JUN-19 | R4667967 |

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

# Reference Information

## QC Samples with Qualifiers & Comments:

| QC Type Description | Parameter | Qualifier | Applies to Sample Number(s) |
|---|---|---|---|

## Sample Parameter Qualifier key listed:

| Qualifier | Description |
|---|---|

## Test Method References:

| ALS Test Code | Matrix | Test Description | Method Reference** |
|---|---|---|---|
| CL-R511-WT | Soil | Chloride-O.Reg 153/04 (July 2011) | EPA 300.0 |

5 grams of dried soil is mixed with 10 grams of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

| ALS Test Code | Matrix | Test Description | Method Reference** |
|---|---|---|---|
| EC-WT | Soil | Conductivity (EC) | MOEE E3138 |

A representative subsample is tumbled with de-ionized (DI) water. The ratio of water to soil is 2:1 v/w. After tumbling the sample is then analyzed by a conductivity meter.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

| ALS Test Code | Matrix | Test Description | Method Reference** |
|---|---|---|---|
| MOISTURE-WT | Soil | % Moisture | CCME PHC in Soil - Tier 1 (mod) |
| PH-WT | Soil | pH | MOEE E3137A |

A minimum 10g portion of the sample is extracted with 20mL of 0.01M calcium chloride solution by shaking for at least 30 minutes. The aqueous layer is separated from the soil and then analyzed using a pH meter and electrode.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

| ALS Test Code | Matrix | Test Description | Method Reference** |
|---|---|---|---|
| RESISTIVITY-CALC-WT | Soil | Resistivity Calculation | APHA 2510 B |

Resistivity are calculated based on the conductivity using APHA 2510B where Conductivity is the inverse of Resistivity.

| ALS Test Code | Matrix | Test Description | Method Reference** |
|---|---|---|---|
| RESISTIVITY-CALC-WT | Soil | Resistivity Calculation | MOECC E3138 |

Resistivity are calculated based on the conductivity using APHA 2510B where Conductivity is the inverse of Resistivity.

| ALS Test Code | Matrix | Test Description | Method Reference** |
|---|---|---|---|
| SO4-WT | Soil | Sulphate | EPA 300.0 |

5 grams of soil is mixed with 50 mL of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

*The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:*

| Laboratory Definition Code | Laboratory Location |
|---|---|
| WT | ALS ENVIRONMENTAL - WATERLOO, ONTARIO, CANADA |

**Chain of Custody Numbers:**

**GLOSSARY OF REPORT TERMS**

*Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.*
*mg/kg - milligrams per kilogram based on dry weight of sample*
*mg/kg wwt - milligrams per kilogram based on wet weight of sample*
*mg/kg lwt - milligrams per kilogram based on lipid weight of sample*
*mg/L - unit of concentration based on volume, parts per million.*
*< - Less than.*
*D.L. - The reporting limit.*
*N/A - Result not available. Refer to qualifier code and definition for explanation.*

*Test results reported relate only to the samples as received by the laboratory.*
*UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.*
*Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.*

## Laboratory Test Results for Physical Properties

Stantec

**Stantec Consulting Ltd**
2781 Lancaster Rd, Suite 100 A&B
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 722-2799

June 24, 2019
File: 122410003

**Attention:** **Dean Tataryn, Kollaard Associates Engineers**

**Reference:** **Kollaard File #190361, ASTM D4318 Atterberg Limit & ASTM D2216 Moisture Content**

The following table summarizes test results for BH-3 SS-6.

| Source | Depth | Natural Moisture Content | Liquid Limit | Plastic Limit | Plasticity Index |
|---|---|---|---|---|---|
| BH-3 SS-6 | 10'-12' | 83.1% | 72.2 | 28.3 | 43.9 |

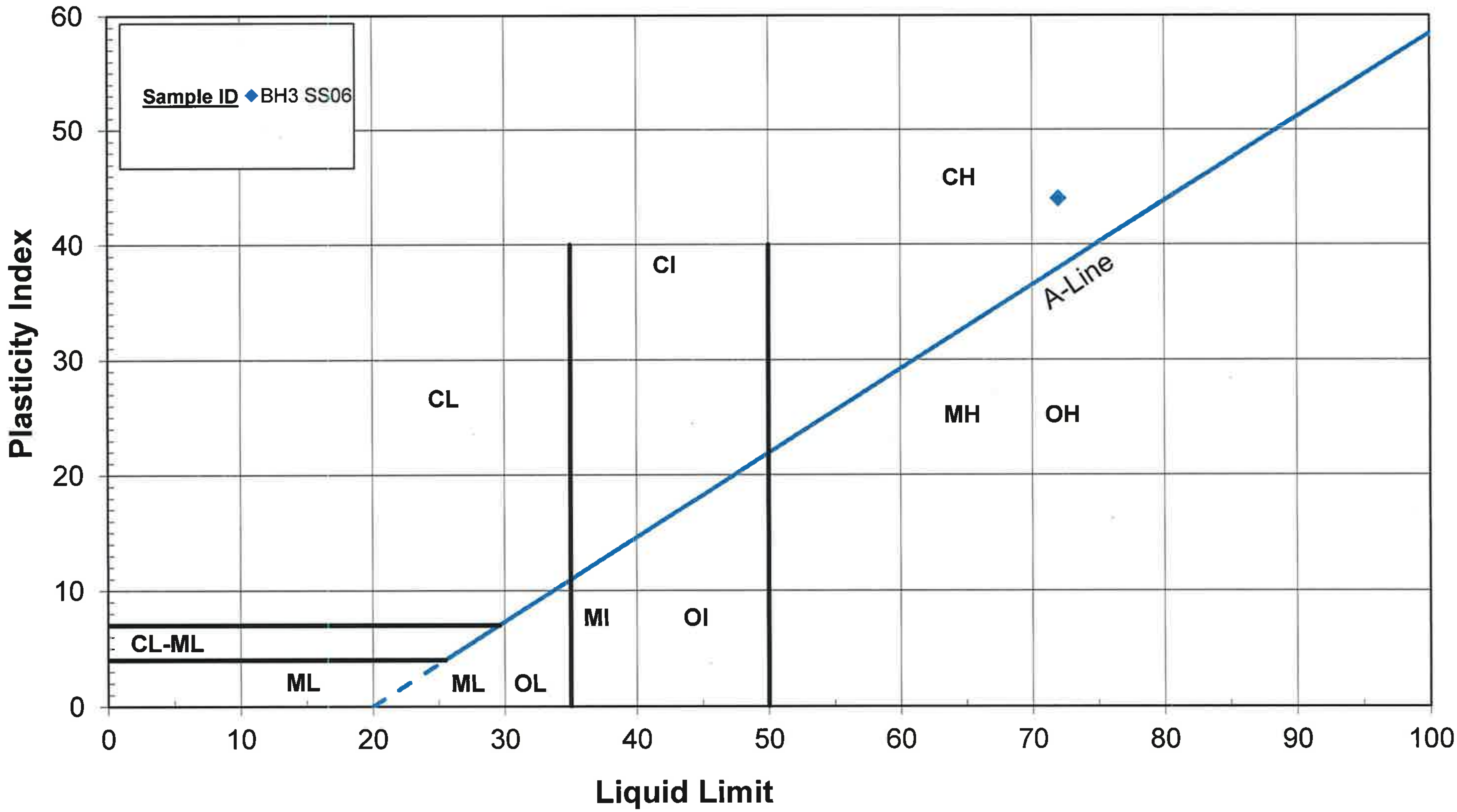
Sincerely,

**Stantec Consulting Ltd**

Brian Prevost

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
Fax: 613-722-2799
brian.prevost@stantec.com

Attachments: Atterberg Limit Plasticity Chart




Stantec

Kollard Associates, File# 190361

1994 St Joseph, Ottawa, ON

# PLASTICITY CHART

Figure No.

Project No. 122410003

## National Building Code Seismic Hazard Calculation

# 2015 National Building Code Seismic Hazard Calculation

**INFORMATION:** Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

**Site:** 45.464N 75.539W **User File Reference:** 1994 St. Joseph Boulevard 2019-06-20 18:45 UT

| Probability of exceedance per annum | 0.000404 | 0.001 | 0.0021 | 0.01 |
|---|---|---|---|---|
| Probability of exceedance in 50 years | 2 % | 5 % | 10 % | 40 % |
| Sa (0.05) | 0.484 | 0.269 | 0.161 | 0.047 |
| Sa (0.1) | 0.563 | 0.323 | 0.201 | 0.065 |
| Sa (0.2) | 0.468 | 0.272 | 0.172 | 0.058 |
| Sa (0.3) | 0.354 | 0.207 | 0.131 | 0.045 |
| Sa (0.5) | 0.249 | 0.145 | 0.092 | 0.032 |
| Sa (1.0) | 0.123 | 0.072 | 0.046 | 0.016 |
| Sa (2.0) | 0.058 | 0.034 | 0.021 | 0.006 |
| Sa (5.0) | 0.015 | 0.008 | 0.005 | 0.001 |
| Sa (10.0) | 0.005 | 0.003 | 0.002 | 0.001 |
| PGA (g) | 0.300 | 0.175 | 0.109 | 0.035 |
| PGV (m/s) | 0.207 | 0.116 | 0.071 | 0.022 |

**Notes:** Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 $m/s^2$). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

---

## References


**National Building Code of Canada 2015 NRCC no. 56190;** Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

**Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)**
**Commentary J**: Design for Seismic Effects

**Geological Survey of Canada Open File 7893** Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada


See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Natural Resources Canada Ressources naturelles Canada

Canada