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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
KING STREET WEST
GANANOQUE, ONTARIO

Project # 201052

Submitted to:

9695443 Canada Inc.
15 Lillico Drive
Ottawa, Ontario
K1V 9L5

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March 17, 2021

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March 17, 2021 201052

9695443 Canada Inc.
15 Lillico Drive
Ottawa, Ontario
K1V 9L5

RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
PART OF LOT 11, CONCESSION 1, PARTS 1-5, PLAN 28R-5002 AND
PARTS 1-3, PLAN 28R-9972
KING STREET WEST
TOWN OF GANANOQUE, ONTARIO

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the above noted proposed residential development located on the south side of King Street West (County Road 2), about a 170 metres west of the intersection of King Street West and Garfield Street, Town of Gananoque, Ontario (See Key Plan, Figure 1).

The purpose of the investigation was to:

- Identify the subsurface conditions at the site by means of a limited number of test pits;
- Based on the factual information obtained, provide recommendations and guidelines on the geotechnical engineering aspects of the project design; including bearing capacity and other construction considerations, which could influence design decisions.

2.0 BACKGROUND INFORMATION AND SITE GEOLOGY

2.1 Existing Conditions and Site Geology

For the purposes of this assessment, project north lies in a direction perpendicular to King Street West (County Road 2) which is located immediately north of the subject site. Currently, the site is a vacant, grassed surfaced lot.

The subject site for this assessment consists of about a 0.49 hectare (1.21 acres) rectangular shaped property located at King Street West, Town of Gananoque, Ontario (see Key Plan, Figure 1).

Surrounding land use is mixed commercial and residential development. The site is bordered on the north by King Street West (County Road 2) followed by commercial development (Colonial Resort), on the west by commercial development (Gateway Motel), on the south by residential development and on the east by a former commercial development (gas station and garage).

The ground surface at the site is currently graded such that surface water drains from west to east across the subject site.

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 overlying shallow bedrock. Bedrock geology maps indicate that the bedrock underlying the site consists of Precambrian bedrock namely granite and syenite.

Based on a review of available borehole information, the overburden at and near the site likely consists of some 1-3 metres of fill materials followed by silty clay followed by bedrock.

2.2 Proposed Development

It is understood that preliminary plans are being prepared for the construction of four micro-unit residential rowhouse buildings, with a total of 22-units. The residential rowhouse buildings will be serviced by adjacent ground surface parking. It is understood that the buildings will be wood framed with some brick veneer and concrete slab-on-grade ground floors. The buildings will be supported using conventional cast-in-place concrete frost walls bearing on spread footing foundations. The proposed buildings will be provided with an asphaltic concrete surfaced access roadway and parking areas. The proposed buildings will be serviced by municipal water and sanitary services.

Surface drainage for the proposed buildings will be by means of swales, catch basins and storm sewers.

3.0 PROCEDURE

The field work for this investigation was carried out on November 13, 2020, at which time seven test pits, numbered TP1 to TP7 were put down at the site using a rubber tire mounted backhoe owned and operated by a local excavation contractor. The test pits put down during the subsurface investigation were for geotechnical purposes only. Identification of the presence or absence of surface or subsurface contamination was outside the scope of work for the investigation. As such, an environmental technician was not on site for environmental sampling or assessment purposes.

The test pits were advanced to depths of about 0.8 to 3.2 metres below the existing ground surface. The subsurface conditions encountered at the test pits were classified based on visual and tactile examination of the materials exposed on the sides and bottom of the test pits (ASTM D2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) and an assessment of the difficulty of digging. In-situ shear vane testing was attempted in the cohesive materials where encountered. The soils were classified using the Unified Soil Classification System. The groundwater conditions were observed in the open test pits at the time of excavating. The test pits were loosely backfilled with the excavated materials upon completion of the fieldwork.

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

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 Pits section following the text of this report. The test pit logs indicate the subsurface conditions at the specific test pit locations only. Boundaries between zones on the test pit logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at locations other than 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 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 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 Materials

Fill materials consisting of topsoil followed by a mixture of grey brown silty clay with a trace to some sand, gravel, cobbles, large boulders and organics were encountered at all of the test pits except TP3. Other materials including a trace of asphalt, brick, concrete, plastic debris and roots were also encountered within the fill materials within those test pits. The fill materials extended to depths of about 0.6 to 1.8 metres below the existing ground surface at the test pit locations. The fill materials were fully penetrated at all of the test pit locations.

4.3 Topsoil

Tosoil was encountered from the surface at TP3 and below the fill materials at TP4, TP5 and TP6. The topsoil thickness at the test pits ranged from about 0.2 to 0.5 metres. The material was classified as topsoil based on 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.

4.4 Silty Clay

A deposit of grey brown to grey silty clay was encountered beneath the fill materials and topsoil at all of the test pits. In situ vane shear tests carried out in the silty clay deposit gave undrained shear strength values of greater than 120 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 stiff to very stiff in consistency. The silty clay deposit was fully penetrated at all test pit locations.

4.5 Bedrock

Bedrock was encountered below the silty clay at all of the test pits at depths ranging between 1.4 to 4.0 metres below the existing ground surface. The surface of the bedrock was scraped using the bucket of the excavator to determine the quality of the upper bedrock. The surface of the bedrock was observed to be smooth and un-fractured within these test pit locations.

4.6 Groundwater

Some seepage was encountered within TP1, TP2 and TP4 at depths of 1.8, 1.7 and 2.7 metres, respectively, 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.

It is considered that the seepage encountered likely consists of water trapped in the fill material above the relatively impermeable silty clay and bedrock and does not constitute a groundwater table.

5.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

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.

5.2 Foundations for Proposed Micro-Unit Rowhouse Residential Buildings

As previously indicated, the subsurface conditions at the site encountered at the test pits advanced for this investigation consisted of fill materials and/or topsoil followed native stiff silty clay overlying bedrock. Based on the subsurface investigation, it is expected that some fill materials and topsoil will be encountered at the proposed building locations. The fill material and topsoil are not considered suitable to support the proposed building foundations.

For predictable performance of the proposed foundations, all existing fill materials, topsoil and any deleterious materials should be removed from within the proposed foundation areas to expose the native silty clay of bedrock. As such it is expected that the proposed buildings will be founded on conventional cast in place concrete foundation supported by spread footings bearing on either native soils or engineered fill below the seasonal frost level.

5.3 Bearing Capacity

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 foundations and the thickness of the soils deposit beneath the footings.

Strip and pad footings, a minimum 0.5 metres in width bearing on the native silty clay about 1.5 to 1.7 metres below the existing ground surface and above the groundwater level or on a suitably constructed engineering pad placed on the native silty clay or bedrock may be designed using a maximum allowable bearing pressure of 150 kilopascals for serviceability limit states and 350 kilopascals for the factored ultimate bearing resistance.

The above allowable bearing pressure is subject to a maximum grade raise of 2.0 metres above the existing ground surface.

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 should be less than 25 millimetres and 20 millimetres, respectively.

5.4 Engineered Fill

Any fill required to raise the footings for the proposed buildings 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 95 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 buildings 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.

5.5 Foundation Excavation

Any excavation for the proposed structures will likely be carried out through fill material to bear within the native silty clay subgrade. The sides of the excavations 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 / Type 3 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 excavations.

The excavations within the fill material above the groundwater level should not present any serious constraints. In contrast, excavations below the groundwater level could present some constraints. There is potential for disturbance to the soil on the sides and bottom of the excavations and relatively flat side slopes may be required to prevent sloughing of material into the excavation unless the groundwater level is lowered in advance of the excavation. In this case, the groundwater inflow should be controlled throughout the excavation by pumping from sumps within the excavations. Notwithstanding, some disturbance and loosening of the subgrade materials could occur, and allowance should be made for subexcavation of any disturbed soil at the subgrade level.

Once the excavation for the foundations are complete, the exposed subgrade should be inspected by a qualified geotechnical person.

5.5.1 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.

Groundwater was observed, in the area of the proposed buildings at about 1.8 to 3.1 metres below the ground surface at time of excavation. Based on the groundwater levels observed, it is considered that the excavation to remove the fill materials from the proposed building foot prints may encounter the groundwater level. However, it is expected that the extent and duration of the excavation for the proposed buildings below the building foot print will be limited. As such a permit to take water prior to excavation for the proposed buildings is not likely required. If construction dewatering requires the removal of more than 50,000 Litres in one normal day, (normal day excludes rainstorms) than an Environmental Activity and Sector Registry will be required.

5.5.2 Effect of Dewatering of Excavations on Adjacent Structures

Since the existing ground water level at the site will be below the expected underside of footing elevations, dewatering of the excavation will not remove water from historically saturated soils. There are no buildings within 50 metres of the proposed buildings at the site. As such dewatering of the foundation or site services excavations, if required, will not have a detrimental impact on the adjacent structures.

5.6 Frost Protection Requirements for Spread Footing Foundations

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.

Where less than the required depth of soil cover can be provided, the foundation elements should be protected from frost by using a combination of earth cover and extruded polystyrene rigid insulation. A typical frost protection insulation detail could be provided upon request, if required.

5.7 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 the foundation walls and 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.

Under slab and perimeter foundation drainage is not considered necessary provided that the floor slab level is above the finished exterior ground surface level on all sides of each rowhouse building.

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.

5.8 Slab on Grade Support

As stated above, it is expected that the proposed buildings 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 areas. 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 sand or granular material, removed during the excavation for the buildings, 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 concrete floor slabs 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.

If any areas of the proposed buildings 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.

5.9 Seismic Design for the Proposed Residential Buildings

5.9.1 Seismic Site Classification

Based on the limited information from the test pits, 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.

5.9.2 National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.111 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.

5.9.3 Potential for Soil Liquefaction

As indicated above, the results of the test pits indicate that the native deposit underlying the site consists of stiff silty clay overlying relatively shallow bedrock As these materials are not prone to liquefaction, it is considered that no damage to the proposed residential buildings should occur due to liquefaction of the native sub-grade under seismic conditions.

6.0 SITE SERVICES

6.1 Excavation

The excavations for the site services will be carried out through fill materials and native silty clay and possibly bedrock. For the purposes of Ontario Regulation 213/91 the soils at the site can be considered to be Type 3 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 beginning 1.2 metres above the base of the excavation. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box. Groundwater seepage into the excavations, if any, should be handled by pumping from sumps in the excavation.

Bedrock will likely be encountered during excavating for site services. Small amounts of bedrock removal, can most likely be carried out by hoe ramming and heavy excavating equipment. Where larger amounts of bedrock removal are required it may be more economically feasible to use drill and blasting techniques which should be carried out under the supervision of a blasting specialist engineer. Monitoring of the blasting should be carried out throughout the blasting period to ensure that the blasting meets the limiting vibration criteria established by the specialist engineer. Pre-blast condition surveys of nearby structures and existing utilities are essential. It is also considered that were large amounts of bedrock are removed by hoe ramming, the hoe ramming could also introduce significant vibrations through the bedrock. A such it is considered that pre-excavation surveys of nearby structures and existing utilities should also be completed before extensive hoe ramming.

Any groundwater inflow into the service trenches should be handled by pumping from sumps from within the excavations.

6.2 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 sub-grade 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.

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 existing or future roadway areas, acceptable native materials should be used as backfill between the roadway sub-grade 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 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 roadway areas. Any boulders larger than 300 millimetres in size should not be used as service trench backfill. 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 sub-grade for the roadways, 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 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.

7.0 ACCESS ROADWAY AND PARKING AREA PAVEMENTS

7.1 Subgrade Preparation

From the test pit results, it is understood that the subsurface conditions in the area of the proposed roadway and parking consist of about 0.2 to 0.3 metres of topsoil overlying fill materials. The fill materials were placed on the native topsoil layer which ranged in thickness from about 0.2 to 0.5 metres. For predictable performance of the pavement structures, it is considered that all of the fill and any topsoil as well as any other soft, wet or deleterious materials will have to be removed in preparation for pavement construction at this site. It is considered that any compactable fill material that is free of topsoil, organic debris and waste materials should be stockpiled for reuse upon approval by the engineer used to raise the subgrade of the roadway and parking areas to the proposed underside of roadway and parking area structure.

It is considered that, with approval by the geotechnical engineer, any existing granular fill material meeting the specifications for OPSS Granular "A" or Granular "B" Type II may be used within the roadway subbase structure.

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 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 granular may be placed.

The areas of the site that require the sub-grade to be raised to proposed underside of pavement structure may be raised as follows:

- Upon approval by the geotechnical engineer, place and compact any stockpiled fill material on the approved sub-grade in maximum 250 mm thick lifts. The stockpiled fill material should be packed with a sheeps foot roller if clayey or with a vibratory compactor if sandy.
- Raise the sub-grade uniformly above the approved subgrade and service trenches until the stockpile fill is used up or the underside of pavement structure is reached.
- If additional material is required to raise the subgrade to the underside of pavement structure, material consisting of OPSS select sub-grade material, OPSS Granular B Type I or Type II maybe used.
- Materials used for raising the sub-grade to proposed underside of pavement structure should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

The pavement structure within the parking area should consist of:

40 millimetres of hot mix asphaltic concrete (HL3) or
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)
6 ounce per square yard non woven geotextile fabric.

The asphaltic concrete thickness within the access roadway and garbage truck turning area should be increased to 90 millimetres as follows:

40 millimetres of hot mix asphaltic concrete (HL3)
Superpave 12.5 asphaltic concrete over
50 millimetres of hot mix asphaltic concrete (HL8)
Superpave 19 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)
6 ounce per square yard non woven geotextile fabric.

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 sub-grade surface and the granular subbase material.

8.0 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 residential buildings should be inspected by Kollaard Associates Inc. to ensure that a suitable sub-grade 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. Any existing granular fill material that is intended to be reused must be inspected and approved by the geotechnical engineer prior to reuse.

The subgrade for the access roadways and parking areas should be inspected and approved by geotechnical personnel. In situ density testing should be carried out on the roadway and parking area 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
17.MAR.2021
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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

c_u undrained shear strength
e void ratio
C_c compression index
C_v coefficient of consolidation
k coefficient of permeability
I_p plasticity index
n porosity
u pore pressure
w moisture content
w_L liquid limit
w_p plastic limit
$^1 effective angle of friction
r unit weight of soil
y^1 unit weight of submerged soil
cr normal stress

RECORD OF TEST PIT 1

PROJECT: Proposed Residential Development
CLIENT: 9695443 Canada Inc.
LOCATION: King Street West, Gananoque, ON

PROJECT NUMBER: 201052
DATE OF EXCAVATING: Nov 13, 2020
SHEET 1 of 1
DATUM: Geodetic

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)	WATER CONTENT % (10 20 30 40)	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		92.77							
0	Topsoil (FILL)		0.00							
	Grey brown silty clay, some large boulders trace gravel, asphalt, concrete, plastic, sand and brick (FILL)									
1										
	Grey SILTY CLAY		90.97 1.80							
2										
3							× >120kPa			Some water observed in test pit at about 1.8 metres below existing ground surface, November 13, 2020.
4	End of test pit, Refusal on BEDROCK		88.77 4.00							
5										

DEPTH SCALE: 1:30
EXCAVATOR TYPE: Rubber Tire Mounted Backhoe

LOGGED: DT
CHECKED: SD

RECORD OF TEST PIT 2

PROJECT: Proposed Residential Development
CLIENT: 9695443 Canada Inc.
LOCATION: King Street West, Gananoque, ON

PROJECT NUMBER: 201052
DATE OF EXCAVATING: Nov 13, 2020
SHEET 1 of 1
DATUM: Geodetic

DEPTH SCALE (meters)	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa (20 40 60 80) / REM. SHEAR STRENGTH ○ Cu, kPa (20 40 60 80)	WATER CONTENT % (10 20 30 40)	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		92.46							
0	Topsoil (FILL)		0.00							
	Grey brown silty clay, some large boulders,trace gravel, asphalt, sand, brick and organics (FILL)									
1										
	Grey SILTY CLAY		90.96 1.50				>120kP			
2										Some water observed in test pit at about 1.7 metres below existing ground surface, November 13, 2020.
							>120kPa			
3							>120kPa			
	End of test pit, Refusal on BEDROCK		88.76 3.70							
4										
5										

DEPTH SCALE: 1:30
EXCAVATOR TYPE: Rubber Tire Mounted Backhoe

LOGGED: DT
CHECKED: SD

RECORD OF TEST PIT 3

PROJECT: Proposed Residential Development
CLIENT: 9695443 Canada Inc.
LOCATION: King Street West, Gananoque, ON

PROJECT NUMBER: 201052
DATE OF EXCAVATING: Nov 13, 2020
SHEET 1 of 1
DATUM: Geodetic

DEPTH SCALE (meters)	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80	WATER CONTENT % 10 20 30 40	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		91.93							
0	TOPSOIL		0.00							
	Grey SILTY CLAY		91.63 0.30							
1										Test pit dry, November 13, 2020.
2	End of test pit, Refusal on BEDROCK		90.03 1.90							
3										
4										
5										

DEPTH SCALE: 1:30
EXCAVATOR TYPE: Rubber Tire Mounted Backhoe

LOGGED: DT
CHECKED: SD

RECORD OF TEST PIT 4

PROJECT: Proposed Residential Development
CLIENT: 9695443 Canada Inc.
LOCATION: King Street West, Gananoque, ON

PROJECT NUMBER: 201052
DATE OF EXCAVATING: Nov 13, 2020
SHEET 1 of 1
DATUM: Geodetic

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa (20 40 60 80) / REM. SHEAR STRENGTH ○ Cu, kPa (20 40 60 80)	WATER CONTENT % (10 20 30 40)	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		92.55							
0	Topsoil (FILL)		0.00							
	Grey brown silty clay, some large boulders,trace gravel, asphalt, concrete, plastic, sand, brick and organics (FILL)		91.45 1.10							
1	TOPSOIL		90.95 1.60							
	Grey SILTY CLAY						× >120kP			
2							× >120kPa			Some water observed in test pit at about 2.7 metres below existing ground surface, November 13, 2020.
3			88.75 3.80				× >120kPa			
4	End of test pit, Refusal on BEDROCK									
5										

DEPTH SCALE: 1:30
EXCAVATOR TYPE: Rubber Tire Mounted Backhoe

LOGGED: DT
CHECKED: SD

RECORD OF TEST PIT 5

PROJECT: Proposed Residential Development
CLIENT: 9695443 Canada Inc.
LOCATION: King Street West, Gananoque, ON

PROJECT NUMBER: 201052
DATE OF EXCAVATING: Nov 13, 2020
SHEET 1 of 1
DATUM: Geodetic

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	WATER CONTENT % 10 20 30 40	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		92.56							
0	Topsoil (FILL)		0.00							
	Grey brown silty clay, trace gravel, asphalt, organics (FILL)		91.86							
	TOPSOIL		0.70							
1	Grey SILTY CLAY		91.16							Test pit dry, November 13, 2020.
	End of test pit, Refusal on BEDROCK		1.40							
2										
3										
4										
5										

DEPTH SCALE: 1:30
EXCAVATOR TYPE: Rubber Tire Mounted Backhoe

LOGGED: DT
CHECKED: SD

RECORD OF TEST PIT 6

PROJECT: Proposed Residential Development
CLIENT: 9695443 Canada Inc.
LOCATION: King Street West, Gananoque, ON

PROJECT NUMBER: 201052
DATE OF EXCAVATING: Nov 13, 2020
SHEET 1 of 1
DATUM: Geodetic

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	WATER CONTENT % 10 20 30 40	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		92.11							
0	Topsoil (FILL)		0.00							
	Grey brown silty clay, trace gravel, asphalt, plastic debris, brick and organics (FILL)		91.11							
1	TOPSOIL		1.00 / 90.81							Test pit dry, November 13, 2020.
	Grey SILTY CLAY		1.30 / 89.51							
2								×>120kPa		
	End of test pit, Refusal on BEDROCK		2.60							
3										
4										
5										

DEPTH SCALE: 1:30
EXCAVATOR TYPE: Rubber Tire Mounted Backhoe

LOGGED: DT
CHECKED: SD

RECORD OF TEST PIT 7

PROJECT: Proposed Residential Development
CLIENT: 9695443 Canada Inc.
LOCATION: King Street West, Gananoque, ON

PROJECT NUMBER: 201052
DATE OF EXCAVATING: Nov 13, 2020
SHEET 1 of 1
DATUM: Geodetic

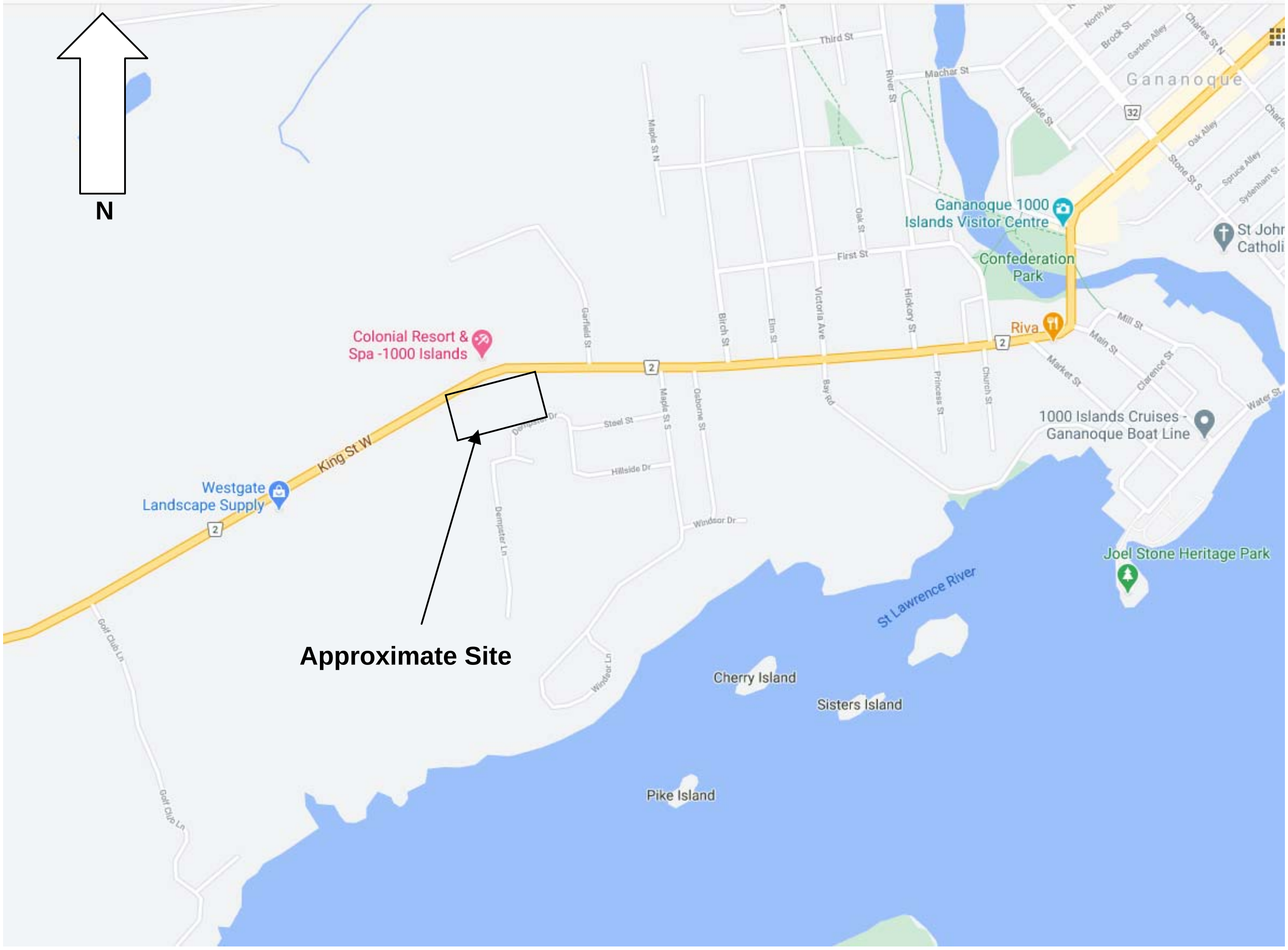
DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa 20 40 60 80	WATER CONTENT % 10 20 30 40	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		92.34							
0	Topsoil (FILL)		0.00							
	Grey brown silty clay, trace gravel, asphalt,concrete, plastic debris, sand, roots and brick (FILL)		91.74 0.60							
1	Grey SILTY CLAY									Test pit dry, November 13, 2020.
2			90.14 2.20				×>120kPa			
	End of test pit, Refusal on BEDROCK									
3										
4										
5										

DEPTH SCALE: 1:30
EXCAVATOR TYPE: Rubber Tire Mounted Backhoe

LOGGED: DT
CHECKED: SD

KEY PLAN

FIGURE 1



NOT TO SCALE

Kollaard Associates
Engineers

Project No. 201052

Date December 2020

DRAWING NUMBER:

SITE PLAN, FIGURE 2

LEGEND:

BH1 APPROXIMATE BOREHOLE LOCATION

TP1 APPROXIMATE TEST PIT LOCATION

REFERENCE: PLAN SUPPLIED BY CIOSINE ONLINE SERVICE

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:

9695443 CANADA INC.

PROJECT:

GEOTECHNICAL INVESTIGATION FOR PROPOSED 22-UNIT RESIDENTIAL DEVELOPMENT

LOCATION:

KING STREET WEST
GANANOQUE, ONTARIO

DESIGNED BY:	*DATE:*
--	NOV 27, 2020
DRAWN BY:	*SCALE:*
DT	N.T.S

KOLLAARD FILE NUMBER:

201052

ATTACHMENT A
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: 44.325N 76.178W

2020-11-26 20:51 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)	0.146	0.092	0.061	0.022
Sa (0.1)	0.192	0.125	0.085	0.033
Sa (0.2)	0.181	0.120	0.083	0.033
Sa (0.3)	0.150	0.101	0.070	0.028
Sa (0.5)	0.120	0.080	0.055	0.022
Sa (1.0)	0.070	0.046	0.032	0.012
Sa (2.0)	0.036	0.023	0.016	0.005
Sa (5.0)	0.010	0.006	0.004	0.001
Sa (10.0)	0.004	0.002	0.002	0.001
PGA (g)	0.111	0.071	0.048	0.018
PGV (m/s)	0.102	0.064	0.042	0.014

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