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REPORT ON

HILAN VILLAGE

**GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
38 CARSS STREET
ALMONTE, ONTARIO**

Project # 210864

Submitted to:

Westview Projects Inc.
18 Louisa Street, Suite 180
Ottawa, Ontario
K1R 6Y6

Rev 0 – Issued for Draft Plan Approval

April 11, 2022

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Westview Projects Inc.
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RE: GEOTECHNICAL INVESTIGATION – HILAN VILLAGE
PROPOSED RESIDENTIAL SUBDIVISION
38 CARSS STREET
ALMONTE, ONTARIO

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the above noted proposed residential subdivision to be located at 38 Carss Street in Almonte, Ontario (see Key Plan, Figure 1). The property is situated southwest of Martin Street and the Ottawa Valley Rail Trail, and northeast of the Mississippi river. The site is approximately 7.4 hectares in area, and is currently vacant and was vegetated with a mix of trees and brush. It is understood that the proposed residential development will consist of a mixture of single family dwellings, semi-detached dwellings and rowhouse development for a total of some 139 units.

The purpose of the investigation was to:

- Identify the subsurface conditions at the site by means of a limited number of test holes;
- 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

For the purposes of this report, Carss Street is considered to be oriented along and east west axis. The proposed residential development is located along the north side of Carss Street at the north side of the existing town of Almonte immediately east of the Mississippi River.

A slope stability evaluation titled *Slope Stability Evaluation, Assessment of Slope Stability and Limit of Hazard Lands Setback* dated November 30, 2021 was completed by Kollaard Associates Inc. for the site.

2.1 Existing Conditions and Site Geology

The subject site for this assessment consists of about a 7.4 hectare (18.3 acres) irregular shaped property located west of the Ottawa Valley Rail Trail (Former CP railway line) and Martin Street and east of the Mississippi River. The property has a civic address known as 38 Carss Street, Almonte, Ontario (see Key Plan, Figure 1). The site has a total average depth between the Ottawa Valley Rail Trail and the normal water level in the Mississippi River of about 203 metres. Of this depth an average of about 74 metres is occupied by the valley slope of the Mississippi River. The site has a width along the former railway line of about 435 metres resulting in a table land above the valley slope of about 5.2 hectares.

Surrounding land use is currently a mixture of residential development, farmland and forest. The site is bordered to the south by Carss Street followed by residential development, to the west by the Mississippi river, to the east by the Ottawa Valley Rail Trail followed by residential development and forest and to the north by farmland and forest.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by silty clay overlying paleozoic bedrock. Bedrock geology maps indicate that the bedrock underlying the site consists of dolostone and sandstone of the Beekmantown group.

Based on a review of overburden thickness mapping for the site area, the overburden is estimated to be between about 0 to 7 metres in thickness above bedrock.

2.2 Proposed Development

It is understood that preliminary plans are being prepared for the construction of a residential subdivision at the site containing a mixture of single, semi and row-house type dwelling units. It is understood that the proposed dwellings will be wood framed and cast-in-place concrete construction with conventional concrete spread footing foundations with basements.

The proposed dwellings will be provided with asphaltic concrete driveways. The proposed development will be serviced by municipal water and by municipal sanitary and storm sewers. It is understood that a pumping station will be required to facilitate the sanitary sewer services.

3.0 PROCEDURE

The field work for this investigation was carried out on September 11, 2019 at which time seven test pits numbered TP1 to TP7 inclusive were put down at the site. The approximate locations of the test holes are shown on the attached Site Plan, Figure 2.

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 0.4 to 3.7 metres below the existing ground surface using a rubber tire mounted backhoe supplied and operated by a local excavation contractor. The soil conditions observed in the test pits were classified based on visual and tactile examination of the materials on the walls and bottom of the test pits and an assessment of the difficulty of digging. The water conditions were observed in the open test pits at the time of the field work.

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. Descriptions of the subsurface conditions encountered at the boreholes are given in the attached Record of Borehole sheets following this report. The results of the laboratory testing of the soil samples are presented in the Laboratory Test Results section and Attachments A and B following the text in this report. The approximate locations of the boreholes are shown on the attached Site Plan, Figure 2.

A series of probe holes were drilled along the proposed development roadways on August 31, 2021 to verify the depth to bedrock using a track mounted drill rig owned and operated by OGS Inc. No geotechnical sampling or soils identification was carried out. The probe holes were advanced to the lesser of the depth to refusal to further auger advancement or 3.1 metres. The probe hole drilling program was not supervised by Kollaard Associates. The results of the depth to bedrock were provided to Kollaard.

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 following the text of this report. The test pit logs indicate the subsurface conditions at the specific 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 test hole 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 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 test hole 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 Topsoil

A thin layer of topsoil was encountered from the ground surface at all of the test hole locations (with the exception of test hole TP5). The topsoil layer ranged in thickness from about 0.1 metres to 0.4 metres. 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. The topsoil was fully penetrated at the test pit locations.

4.3 Fine Sand / Silty Sand

Yellow brown to red brown fine or silty sand was encountered below the topsoil layer in test pits TP2, TP3 and TP4. The sand was fully penetrated at depths of 1.05, 0.4 and 0.9 metres respectively. The sand was underlain by bedrock at TP2 and TP3 and by silty clay at TP4.

4.4 Silty Clay

Silty clay was encountered beneath the topsoil in test pits TP1, TP6 and TP7 and beneath the fine silty sand in TP4. The silt clay was damp to dry to the touch and stiff to very stiff in consistency. The silt clay was fully penetrated in test pits TP1, TP4 and TP6 at depths of 0.4, 3.6 and 1.2 metres respectively. TP7 was terminated in the silty clay at 3.7 metres below grade. The silt clay was underlain by bedrock at TP1 and TP4 and by glacial till at TP6.

It is noted that OMAFRA mapping indicates marine deposited clays along the east side of the Mississippi River. As indicated by the factual information obtained from the test pits, the silty clay at the site is limited in thickness (depth) and extent. Further the silty clay encountered at the site is weathered and consolidated into a stiff to very stiff consistency. The marine deposited silty clay at the site is not considered to be sensitive at the moisture content and degree of consolidation or consistency present at the site. As such, the marine deposited silty clay is not considered to fit into the Hazardous sites category indicated in section 6 of the Provincial Policy Statement. The silty clay at the site does not require any special construction technique or mitigation with respect to the proposed development.

4.5 Glacial Till

Glacial till was encountered below the topsoil in TP5 and below the sand in TP6 at depths of 0.5 and 1.2 metres respectively. The glacial till was compact to very dense and was underlain by bedrock.

4.6 Bedrock

Bedrock was encountered at all of the test pits with the exception of TP7. The depth to bedrock ranged from 0.4 to greater than 3.7 metres. The bedrock where observed consisted of near horizontally bedded dolostone.

The results of the additional probe hole drilling program indicates that the deep to bedrock exceeds 3.0 metres along the east side of the proposed development for about the first 200 metres of the site from Carss Street. The depth to bedrock decreases along the east side of the site from less than 3.0 metres at about 200 metres from Carss Street to about 0.4 metres at the north east end of the development. The depth to bedrock varies from greater than 3 metres about 100 metres east of the the top of slope to less than 1.5 metres about 35 metres east of the top of the slope.

4.7 Groundwater

All test pits were dry at the time of excavation on September 11, 2019. It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

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 Foundation Subgrade

The subsurface conditions at the site encountered below the topsoil at the test holes advanced during the investigation varied from yellow brown to red brown fine silty sand in the centre of the site to silty clay overlying glacial till followed by bedrock at the south and silty clay overlying bedrock to the north end of the site.

5.3 Foundation Design and Bearing Capacity

As previously indicated, the proposed dwellings will be constructed on conventional cast in place concrete basement foundations supported by spread footings. 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 and consistency of the soils deposit beneath the footings.

With the exception of the topsoil, the subsurface conditions encountered at the test holes advanced during the investigation are suitable for the support of the proposed buildings on conventional spread footing foundations placed on a native subgrade or on engineered fill placed on the native subgrade. The excavations for the foundations should be taken through any topsoil or otherwise deleterious material to expose the native sand, silty clay, glacial till or bedrock.

Strip and pad footings, a minimum of 0.5 metres in width bearing on the native undisturbed fine sand above the ground water level may be designed using a maximum allowable bearing pressure of 75 kilopascals for serviceability limit states and 150 kilopascals for the factored ultimate bearing resistance. Where foundation is placed on a subgrade which varies from fine silty sand to silty clay, glacial till or bedrock, all of the footings supporting the foundation should be designed for the fine sand.

Strip and pad footings, a minimum or 0.5 metres in width bearing on the native undisturbed grey brown glacial till or silty clay above the groundwater level or on a suitably constructed engineering pad placed on the native grey brown glacial till or silty clay and above the ground water level may be designed using a maximum allowable bearing pressure of 95 kilopascals for serviceability limit states and 200 kilopascals for the factored ultimate bearing resistance.

Strip and pad footings, a minimum 0.5 metres in width bearing on the native bedrock or on an engineered pad placed on the bedrock above the groundwater level may be designed using a maximum allowable bearing pressure of 250 kilopascals for serviceability limit states and 500 kilopascals for the factored ultimate bearing resistance.

The above allowable bearing pressures are subject to a maximum grade raise of 3.0 metres above the existing ground surface and to maximum strip and pad footing widths of 1.5 metres when bearing on the native silty sand, glacial till or silty clay. There is no maximum grade raise or footing width restriction when bearing on bedrock.

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 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 the subgrade above the 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 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 Excavations

Any excavation for the proposed structures will likely be carried out through topsoil, sand, clay and/or glacial till to bear within the native clay, till or bedrock 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 3 soil above bedrock and Type 1 soil below the bedrock surface, however this classification should be confirmed by qualified individuals as the site is excavated and if necessary, adjusted.

Based on the expected depths of excavation for the foundations, It is expected that the side slopes of the excavation will be stable provided the walls are sloped at 1H:1V provided no excavated materials are stockpiled within 2 metres of the top of the excavations.

5.6 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.

All test pits were observed to be dry at the time of excavation. In addition, the proposed development is located on the table land at a elevation significantly above the adjacent river. As such, it is considered that the excavations will not extend below the ground water level. A permit to take water will not be required prior to excavation.

5.7 Effect of Dewatering of Foundation or Site Services 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 foundation excavations will not remove water from historically saturated soils.

The excavation for the site services is expected to extend through the surficial soils and into the underlying bedrock in some locations. It is expected that ground water will not be encountered within the surficial soils. The underlying bedrock is not sensitive to changing moisture conditions.

As such dewatering of the foundation or site services excavations, if required, will not have a detrimental impact on adjacent structures.

5.8 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. It is anticipated that the underside of the footing for each foundation will be stepped if required to achieve sufficient depth for frost protection.

Where foundation footings are founded on bedrock, an assessment of the bedrock for frost susceptibility should be completed by a qualified geotechnical person. If it is considered that the bedrock is not susceptible to frost, the minimum earth cover for frost protection could be reduced and insulation will not be required.

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.

5.9 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.

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.

The basement foundation walls should be designed to resist the earth pressure, P, acting against the walls at any depth, h, calculated using the following equation.

$$\mathbf{P = k_0 (\gamma h + q)}$$

Where:	P	=	the pressure, at any depth, h, below the finished ground surface
	k_0	=	earth pressure at-rest coefficient, 0.5
	γ	=	unit weight of soil to be retained, estimated at 22 kN/m^3
	q	=	surcharge load (kPa) above backfill material
	h	=	the depth, in metres, below the finished ground surface at which the pressure, P, is being computed

This expression assumes that the water table would be maintained at the founding level by the above mentioned foundation perimeter drainage and backfill requirements.

A conventional, perforated perimeter drain, with a 150 millimetre surround of 20 millimetre minus crushed stone, should be provided at the founding level for the cast-in-place concrete basement floor slab and should lead by gravity flow to a sump. The sump should discharge by gravity via a storm service to the adjacent storm sewer or ground surface. The storm service should be equipped with a backup flow protector.

5.10 Basement Floor Slab Support

As stated above, it is expected that the proposed buildings will be founded on silty sand, native clay, till or bedrock subgrade or on an engineered pad placed on a native subgrade. For predictable performance of the proposed concrete basement floor slab all existing 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.

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.

It is common practice to backfill from the underside of footing level to the basement floor slab using clear crushed stone. Where the subgrade soils consist of sand or silty sand, or glacial till, it is recommended that clear crushed stone not be used as backfill below the concrete floor slab without the use of a Type 1 geotextile fabric between the clearstone and the native subgrade. If clear crushed stone is used, the clear stone should be properly consolidated using several passes with a large diesel plate compactor.

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.

5.11 Seismic Design for the Proposed Residential Buildings

5.11.1 Seismic Site Classification

Based on the limited information from the boreholes, 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 above the bedrock and Site class B below the bedrock surface.

5.11.2 National Building Code Seismic Hazard Calculation

The online 2015 National Building Code Seismic Hazard Calculation was used to verify the seismic conditions at the site. The design Peak Ground Acceleration (PGA) for the site was calculated as 0.222 with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The seismic site classification for the site is indicated to be Seismic Site Class D above bedrock and Site Class B below the bedrock surface. The results of the calculation are attached following the text of this report.

5.11.3 Potential for Soil Liquefaction

As indicated above, the results of the test pits and information from geological maps indicate that the native deposits underlying the site consist of silty sand or silty clay followed by glacial till in some areas then bedrock.

These native soils are not considered to be prone to liquefaction during a seismic event at the degree of compaction and thickness present at the site.

6.0 SITE SERVICES

The proposed development will be provided with municipal services. The municipal water main will be extended from Union Street to service the site and will be looped through the Lansdowne Street road allowance to complete a loop in the municipal system.

The proposed development will be provided with a gravity sanitary sewer system which will outlet to a proposed pump station within the development. The pump station will discharge to the municipal gravity system along Union Street by means of sanitary force mains.

A storm sewer system will be installed within the proposed development. The storm sewer system will discharge to the adjacent Mississippi River.

6.1 Excavation

The excavations for the site services will be carried out through topsoil, silty sand, silty clay, glacial till and bedrock. For the purposes of Ontario Regulation 213/91 the soils above the bedrock at the site can be considered to be Type 3 soil and below the bedrock level can be considered to be Type 1 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 to within 1.2 metres of the bottom of the service trench.

Work within an excavation in the bedrock should follow the requirements of Ontario Regulation 213/91 in particular O.Reg 213/91 S230 – S233. Excavation walls within bedrock may be made near vertical.

Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box.

6.1.1 Bedrock Removal

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 where large amounts of bedrock are removed by hoe ramming, the hoe ramming could also introduce significant vibrations through the bedrock. As such it is considered that pre-excavation surveys of nearby structures and existing utilities should also be completed before extensive hoe ramming.

6.1.2 Excavation Dewatering

All of the test pits were dry at the time of drilling. As such, 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. A permit to take water is not expected to be required.

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 OPSS Granular A.

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

6.3 Trench Backfill

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

In areas where the service trench will be located below or in close proximity to 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 material consists of bedrock, Granular A or Granular B Type 2 may be used for backfill.

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 ROADWAY PAVEMENTS

7.1 Subgrade Preparation

In preparation for pavement construction at this site any fill and topsoil and any soft, wet or deleterious materials should be removed from the proposed roadway areas. The exposed subgrade surface should then be inspected and approved by geotechnical personnel. Based on the results of the test pits, the subsurface conditions in the roadway areas in general consist of fine grained silty sand, silty clay or glacial till. Any soft or unacceptable areas evident should be subexcavated and replaced with suitable earth borrow material. The subgrade should be shaped and crowned to promote drainage of the roadway granulars. Following approval of the preparation of the subgrade, the pavement granulars may be placed.

For any areas of the site that require the subgrade to be raised to proposed roadway subgrade level, the material used should consist of OPSS select subgrade material or OPSS Granular B Type I or Type II. Materials used for raising the subgrade to proposed roadway area subgrade level should be placed in maximum 300 millimetre thick loose lifts and be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

7.2 Pavement Structure

The proposed pavement structure of the roadways should consist of the following:

40 millimetres of Superpave 12.5 asphaltic concrete over
50 millimetres of Superpave 19 asphaltic concrete over
150 millimetres of OPSS Granular A base over
350 millimetres of OPSS Granular B, Type II subbase over
(50 or 100 millimetre minus crushed stone)

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 sub-grade 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.

8.0 TREES

The upper soils at the site consist of silty sand, fine sand, glacial till and stiff silty clay. Due to the moisture content and consistency, the silty clay soils on the majority of the site are not particularly sensitive to changes in moisture content.

In keeping with the City of Ottawa, Tree Planting in Sensitive Marine Clay Soils - 2017 Guidelines small and medium sized trees can be planted as close as 4.5 metres from the proposed dwelling provided sufficient soil volume is available around the proposed tree location. Large trees should be planted no closer than 1 times their height from a proposed dwelling.

9.0 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.

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.

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 pavement granular materials and on the service trench bedding, cover and backfill to ensure the materials meet the specifications from a compaction point of view.

The native soils at this site will be sensitive to disturbance 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
11.APR. 2022
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve DeWit, P.Eng.

RECORD OF TEST PITS
GEOTECHNICAL ASSESSMENT
38 CARSS STREET
ALMONTE, ONTARIO

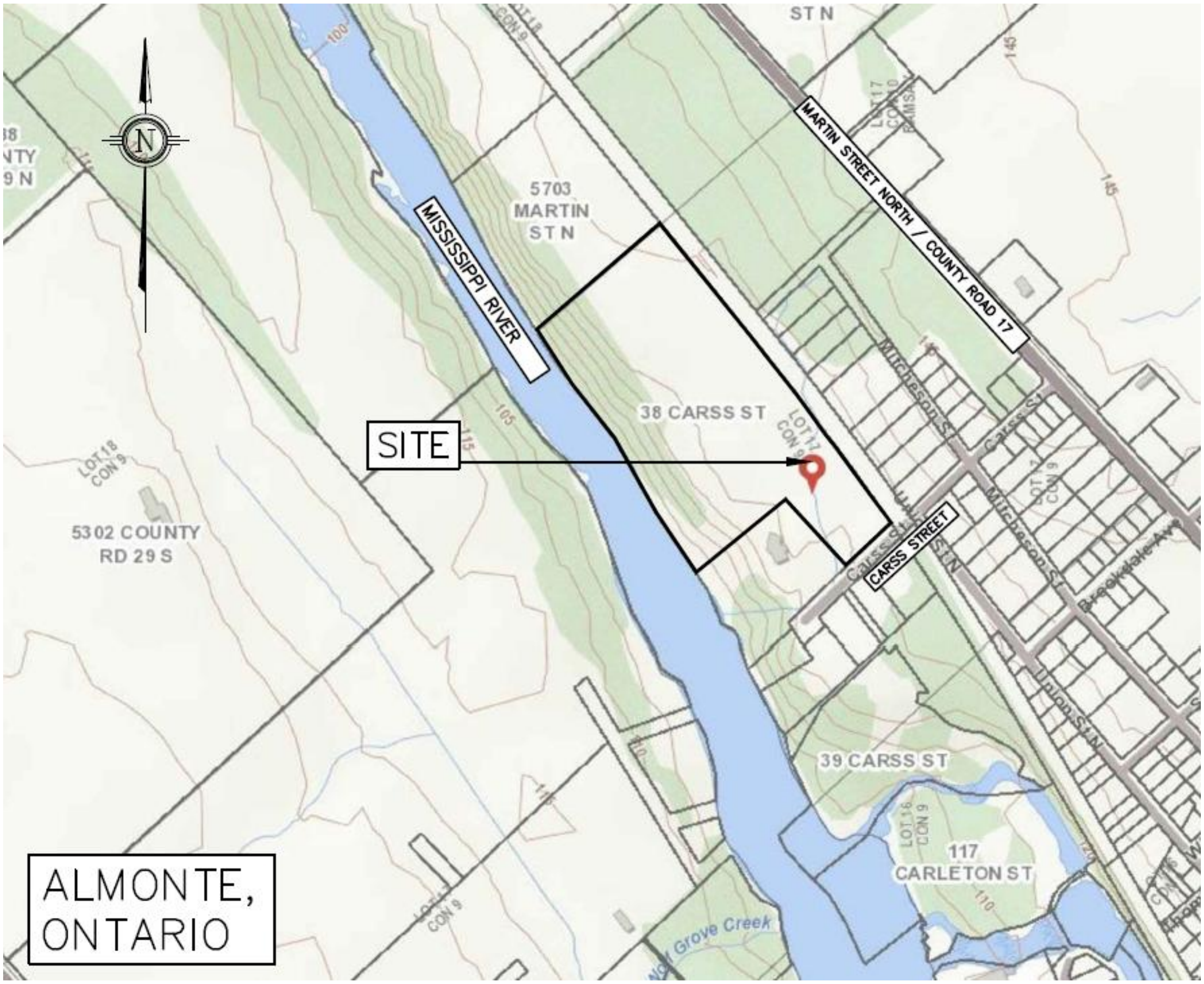
TABLE I
TEST PITS ADVANCED USING TIRE MOUNTED BACKHOE SEPTEMBER 11, 2019

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP1 (Line 1) Sept 11, 2019	0.00 – 0.20	TOPSOIL
	0.20 – 0.40	brown SILTY CLAY, trace fine sands
	0.40	End of test pit on BEDROCK
TP2 (under hydro line) Sept 11, 2019	0.00 – 0.25	TOPSOIL
	0.25 - 1.05	Till, red brown FINE SAND with cobbles and boulders trace clay
	1.05	End of test pit on BEDROCK
TP3 (line 2) Sept 11, 2019	0.00 – 0.30	TOPSOIL
	0.30 – 0.40	Red brown FINE SAND trace silt
	0.40	End of test pit on BEDROCK
TP4 Sept 11, 2019	0.00 – 0.25	TOPSOIL
	0.25 – 0.90	Yellow brown, fine SILTY SAND trace clay and cobbles
	0.90 – 3.60	Grey brown STIFF CLAY
	3.60 -	End of test pit on BEDROCK

TABLE I(continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP5 (near line 3 by garden) Sept 11, 2019	0.00 – 0.40	Fill Clay
	0.40 – 0.50	TOPSOIL
	0.50 – 1.30	Yellow brown to grey brown GLACIAL TILL (fine sand mixed with boulders and cobbles with clay pockets)
	1.30	End of test pit on BEDROCK
TP6 Sept 11, 2019	0.0 - 0.30	TOPSOIL
	0.30 – 1.20	Grey brown stiff SILTY CLAY (Sample taken)
	1.20 – 2.10	GLACIAL TILL – Grey brown clay with sand pockets boulders and cobbles
	2.10	End of test pit on large boulder or BEDROCK
TP7 Sept 11, 2019	0.0 – 0.25	TOPSOIL
	0.25 – 3.70	Grey brown STIFF CLAY
	3.70 –	End of test pit

KEY PLAN	FIGURE 1



NOT TO SCALE

Kollaard Associates
Engineers

Project No. 210864

Date April 2022

DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

BH2 — APPROXIMATE TESTHOLE 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:
WESTVIEW PROJECTS INC

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED SUBDIVISION

LOCATION:
38 CARSS STREET
ALMONTE, ONTARIO

DESIGNED BY:	DATE:
--	MARCH 1, 2022
DRAWN BY: DT	SCALE: N.T.S

KOLLAARD FILE NUMBER:
210864

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: 45.234N 76.204W **User File Reference:** 38 Carss Street 2022-02-23 16:34 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)	0.344	0.180	0.106	0.033
Sa (0.1)	0.410	0.226	0.139	0.047
Sa (0.2)	0.346	0.198	0.125	0.045
Sa (0.3)	0.266	0.155	0.100	0.036
Sa (0.5)	0.192	0.114	0.074	0.027
Sa (1.0)	0.099	0.060	0.039	0.014
Sa (2.0)	0.048	0.029	0.018	0.005
Sa (5.0)	0.013	0.007	0.004	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.222	0.125	0.077	0.026
PGV (m/s)	0.161	0.091	0.057	0.018

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

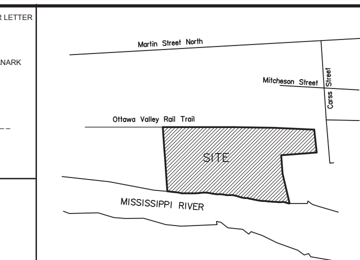
See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

SUBJECT TO THE CONDITIONS, IF ANY, SET FORTH IN OUR LETTER DATED

________________________.

THIS DRAFT PLAN IS APPROVED BY THE COUNTY OF LANARK UNDER SECTION 51 OF THE PLANNING ACT.
THIS ____ DAY OF ____________, 20__.

____
KURT GREAVES
CHIEF ADMINISTRATIVE OFFICER
COUNTY OF LANARK



KEY MAP
NOT TO SCALE

DRAFT PLAN OF SUBDIVISION OF

PART OF LOT 17
CONCESSION 9
Geographic Township of Ramsay
Municipality of Mississippi Mills
COUNTY OF LANARK

Prepared by Annis , O'Sullivan , Vollebekk Ltd.

Scale 1 : 500

20 15 10 5 0 10 20 Metres

Metric

DISTANCES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048

SURVEYOR'S CERTIFICATE

I CERTIFY THAT :
The boundaries of the lands to be subdivided and their relationship to adjoining lands have been accurately and correctly shown.

____________ Date

____________________ E. H. Herweyer
ONTARIO LAND SURVEYOR

OWNER'S CERTIFICATE

This is to certify that I am the owner / agent of the lands to be subdivided and that this plan was prepared in accordance with my instructions.

____________ Date

______________________________ L. Aggarwal
2849358 Ontario Inc.
I have authority to bind the corporation

ADDITIONAL INFORMATION REQUIRED UNDER SECTION 51-17 OF THE PLANNING ACT

(a) see plan
(b) see plan
(c) see plan
(d) (purpose for which lots are to be used)
(e) see plan
(f) see plan
(g) see plan
(h) Municipality of Mississippi Mills
(i) see soils report
(j) see plan
(k) (municipal services available or to be available)
(l) see plan

Ontario Land Surveyors
ANNIS, O'SULLIVAN, VOLLEBEKK LTD.
14 Concourse Gate, Suite 500
Nepean, Ont. K2E 7S6
Phone: (613) 727-0850 / Fax: (613) 727-1079
Email: Nepean@aovltd.com

Job No. 21770-21 Westview PtLt 17 C9 RA DPS D4 | ig