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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-UNIT RESIDENTIAL BUILDING
58 FLORENCE STREET
CITY OF OTTAWA, ONTARIO

Project # 190186

Submitted to:

Falsetto Homes Inc.
52 Sullivan Avenue
Ottawa, Ontario
K2G 1V2

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June 5, 2019

Revision 1 - Response to City of Ottawa Review Comments April 2, 2020

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April 2, 2020 190186

Falsetto Homes Inc.
52 Sullivan Avenue
Ottawa, Ontario
K2G 1V2

RE: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-UNIT RESIDENTIAL BUILDING
58 FLORENCE STREET
CITY OF OTTAWA, ONTARIO

Dear Sirs:

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

BACKGROUND INFORMATION AND SITE GEOLOGY

The site consists of about 0.04 hectares (0.09 acres) of land located on the south side of Florence Street, about 42 metres east of the intersection of Kent Street and Florence Street, City of Ottawa, Ontario. The site is currently occupied by a two and a half storey single family dwelling (see Key Plan, Figure 1). In general, the buildings in the area are residential buildings with varying degrees of historical value.

It is understood that it is planned to redevelop the site into a multi-unit three storey residential building. It is understood that the proposed building will be of wood framed construction with a conventional cast in place concrete basement foundation. The proposed building will be serviced by municipal sewer and water supply. The proposed development will be accessed by local residential roadways. Surface drainage for the proposed development will be by means of swales, catch basins and storm sewers.

The site is bordered on the east, south and west by residential development, on the north by Florence Street followed by mixed residential and institutional development.

Historic Information

Historically, the property immediately west of the subject site, now known as 429 Kent Street was occupied by several small buildings and parking areas. The building immediately west of the site was removed between 1991 and 1999. 429 Kent was redeveloped beginning in 2011 to contain a 4 and 5 storey residential building complete with 2 levels of underground parking. It is understood that the parking levels of the building at 429 Kent extend to more than 7.5 metres below the existing ground surface at the site. The existing building at 429 is set back from the property line between the 58 Florence Street and 429 Kent Street by about 1.2 metres. A ventilation shaft for the below grade parking has essentially 0 setback from this property line.

It is understood that during the construction of the building at 429 Kent Street, the shoring between the subject site and 429 Kent Street Excavation failed. This failure resulted in significant movement and settlement of the soils below the foundation at the west side of the existing dwelling on the subject site. This movement has resulted in significant differential settlement and rotation of portions of the existing dwelling. Due to this failure of the shoring and movement of the soils, it is considered that the soils, which will be under a part of the proposed dwelling, will be disturbed.

Site Geology

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by fine textured glaciomarine deposits. Bedrock geology maps indicate that the bedrock underlying the site consists of dark grey almost black limestone of the Eastview Formation.

Based on a review of overburden thickness mapping for the site area, the overburden is estimated to be between about 5 to 10 metres in thickness above bedrock.

PROCEDURE

The field work for this investigation was carried out on May 22, 2019 at which time two boreholes, numbered BH1 and BH2 were put down at the site using a track mounted drill rig equipped with a hollow stem auger owned and operated by CCC Group of Ottawa, Ontario. Due to the limited space available on the site which allowed access by a drill rig, both test holes were advanced through the driveway along the east side of the site. BH1 was advanced near the front of the site and BH2 was advanced near the rear of the existing building.

Sampling of the overburden materials encountered at the borehole location was carried out at regular 0.75 metre depth intervals using a 50 millimetre diameter drive open conventional split spoon sampler in conjunction with standard penetration testing (ASTM D-1586 – Penetration Test and Split Barrel Sampling of Soils) and in situ vane shear testing (ASTM D-2573 Standard Test Method for Field Shear Test in Cohesive Soil). Each of the boreholes was advanced to 8.2 metres below the existing ground surface using 200 mm hollow stem augers. Borehole BH1 was continued to a depth of 12.8 metres below the existing ground surface as a probe hole using dynamic cone penetration testing. The soils were classified using the Unified Soil Classification System.

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

One soil sample was delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack on concrete and corrosivity to buried steel. One soil sample (BH1 – SS5) was submitted for Atterberg Limits (D4318), Moisture Content (ASTM D2216) and one sample (BH2 – SS6) was submitted for Hydrometer testing (ASTM D422). The soils were classified using the Unified Soil Classification System.

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

SUBSURFACE CONDITIONS

General

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

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification was in general completed by visual-manual procedures in accordance with ASTM 2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) with select samples being classified by laboratory testing in accordance with ASTM 2487. Classification and identification of soil involves judgement and Kollaard Associates Inc. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

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

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

Asphaltic Concrete

Asphaltic concrete was encountered from the surface at boreholes BH1 and BH2. The asphaltic concrete measured about 40 millimetres in thickness.

Fill

Beneath the asphaltic concrete, a layer of grey crushed stone ranging in thickness from about 100 to 110 millimetres was encountered at both boreholes. Following the asphaltic concrete and grey crushed stone layers, fill materials consisting of red brown silty sand with a trace of gravel was encountered. The sand fill materials ranged in thickness from about 1.95 to 2.0 metres and were encountered at depths of about 0.4 to 1.8 metres below the existing ground surface. The fill materials were fully penetrated at both borehole locations.

Sand

A thin deposit of grey brown medium to coarse sand was encountered beneath the fill materials at both boreholes. The sand was encountered at depths of about 2.1 metres in below the existing ground surface at both boreholes. The thickness of the sand layer was about 30 millimetres and was fully penetrated at both borehole locations.

Silty Clay

Beneath the sand, a deposit of grey silty clay was encountered at both boreholes. In situ vane shear tests carried out in the silty clay deposit gave undrained shear strength values ranging from about 85 kilopascals to 135 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 was fully penetrated in Borehole BH1 at a depth of 7.95 metres below the existing ground surface. Borehole BH2 was terminated in the silty clay at a depths of about 8.2 metres below the existing ground surface.

The results of Atterberg Limits tests and moisture content (ASTM D422) conducted on one soil sample (BH1 - SS5 -3.05 to 3.65 metres) of the silty clay are presented in the following table and in Attachment A at the end of the report. The tested silty clay sample classifies as high plasticity in accordance with the Unified Soil Classification System. The results of the laboratory testing are located in Attachment A.

Table I – Atterberg Limit and Water Content Results

Sample	Depth(metres)	LL (%)	PL (%)	PI (%)	W (%)
BH1-SS5	3.05 - 3.65	82.2	27.8	54.4	73.6

LL: Liquid Limit PL: Plastic Limit PI: Plasticity Index w: water content

CH: Inorganic High Plastic Soils

One sample of silty clay (BH2-SS6–6.10 to 6.70 metres) was also submitted to Stantec for hydrometer testing (ASTM D422). The results of the hydrometer testing indicated that the sample consisted of about 51.0 percent clay, 40.6 percent silt, 7.4 percent sand and 1.0 percent gravel. The results are located in Attachment A.

Glacial Till

A deposit of grey silty sand glacial till was encountered beneath the silty clay layer at BH1. The glacial till consists of gravel and possible cobbles and boulders in a matrix of silty sand with some clay and shale fragments. The glacial till was encountered at a depth of about 7.95 metres below the existing ground surface.

BH1 was continued by dynamic cone penetration testing beginning at a depth of 8.2 metres below the existing ground surface. The dynamic cone penetration test carried out at BH1 gave values ranging from WH to 65 blows per 0.3 metres. The dynamic cone penetration test values increased with depth below about 9.4 metres and ranged from 9 to 65 blows per 0.3 metres. At a depth of some 12.8 metres below the existing ground surface, refusal to cone penetration was encountered and indicates either large boulders or bedrock.

Groundwater

No groundwater seepage was observed within boreholes at the time of drilling on May 22, 2019. It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

Corrosivity on Reinforcement and Sulphate Attack on Portland Cement

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

Item	Threshold of Concern	Test Result	Comment
Chlorides (Cl)	Cl > 0.04 %	0.008	Negligible
pH	5.0 < pH	8.30	Basic Negligible concern
Resistivity	R < 20,000 ohm-cm	3130	Corrosive
Sulphates (SO_4)	SO_4 > 0.1%	0.007	Negligible concern

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

The pH value for the soil sample was reported to be at 8.30, indicating a durable condition against corrosion. This value was evaluated using Table 2 of Building Research Establishment (BRE) Digest 362 (July 1991).The pH is greater than 5.5 indicating the concrete will not be exposed to attack from acids.

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

Corrosivity Rating for soils range as follows: extremely corrosive with a resistivity rating <1000 ohm-cm; highly corrosive with a resistivity of 1,000 to 3,000 ohm-cm; corrosive with a resistivity of 3,000 to 5,000 ohm-cm; moderately corrosive with a resistivity of 5,000 to 10,000 ohm-cm; mildly corrosive with a resistivity of 10,000 to 20,000 ohm-cm; non-corrosive with a resistivity of >20,000 ohm-cm. The Soil resistivity was found to be 3,130 ohm-cm for the sample analyzed making the soil corrosive for buried steel.

Alternatively: The results of the laboratory testing of a soil sample for resistivity and pH indicates the soil sample tested has an underground corrosion rate of about 0.75 loss-oz./ft^2/yr. Based on the findings of Fischer and Bue (1981) underground corrosion rates (loss-oz./ft^2/yr) of 0.30 and less are considered nonaggresive, from 0.30 to 0.75 the rate is considered slightly aggressive, from 0.75 to 2.0 the rate is considered aggressive and 2.0 and greater the rate is considered very aggressive. Accordingly, the above mentioned soil sample is considered to have a aggressive corrosion rate to reinforcement steel within below grade concrete walls. Special protection is required for reinforcement steel within the concrete walls.

GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

General

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

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from offsite sources are outside the terms of reference for this report.

Foundation Excavation

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

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

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

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

The building is proposed to have a full basement. The depth of the excavation will be taken through the fill materials, native sand and into the native silty clay soils. There should be no excavated material stockpiled with a distance from the excavation equal to the depth of excavation. At this time the building setbacks are not known. However, it is anticipated that the proposed building setbacks will be limited to within 1.2 to 1.5 metres. Typically excavations are advanced with 1 metre offsets from the proposed foundation wall so to allow forming. As such the base of the excavation is expected to extent to about 0.2 metres from the property lines.

It is considered however that the side slopes of the excavation will not be achievable without encroaching on to the adjacent properties. Permission for encroachment should be obtained. Where permission is not given or encroachment is not possible, the side slope of the excavation will have to be shored.

If there is insufficient space to provide adequate side slopes, the excavation will require shoring on all sides.

The shoring should be designed by a shoring specialist to support the lateral earth pressure ‘p’ plus the additional surcharge load of the adjacent row house building. The lateral earth pressure 'p' can be calculated using the following equation:

$$p = k(\gamma h + q) + \gamma_w H$$

Where p	=	the lateral earth pressure, at any depth, h, below the ground surface
k	=	earth pressure coefficient of 0.35
γ	=	unit weight of soil to be retained, estimated at 20 kN/m^3
h	=	the depth, in metres, at which pressure, p, is being computed
γ_w	=	unit weight of water (9.81 kN/m^3)
H	=	height of water level, in metres, from bottom of the excavation
q	=	the equivalent surcharge acting on the ground surface adjacent to the shoring including expected vehicular loads

Alternatively, in keeping with O.Reg 213/91, s. 234 a written opinion could be provided by a professional engineer at the time of excavation that the walls of the walls of the excavation are sufficiently stable that no worker will be endangered if no support system is used.

Ground Water in Excavation and Construction Dewatering

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

Since groundwater was not encountered in the boreholes during drilling, it is considered that the excavation will not extend below the normal ground water level. Since the normal ground water level is below the base of the excavation the soils adjacent the excavation have previously been dewatered and will not be sensitive to any water removal from the excavation.

A permit to take water is not expected to be required. It is noted that construction dewatering due to surface water draining into the excavation is considered to be taking water and as such an environmental sector registry may be required if surface drainage is not properly controlled.

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 elevation, dewatering of the excavation will not remove water from historically saturated soils. As such dewatering of the foundation or site services excavations, if required, will not have a detrimental impact on the adjacent structures.

Foundation for Proposed Multi-Unit Residential Building

Foundation Design and Bearing Capacity

As previously indicated, the subsurface conditions at the site encountered at the boreholes advanced during the investigation consisted of asphaltic concrete, crushed stone and deleterious fill materials, fine to coarse sand, silty clay followed by glacial till.

Also as previously indicated, the soils below the west half of the proposed residential building have been disturbed by the failure of the shoring on the adjacent property known as 429 Kent Avenue. It is considered that these soils are not sufficient to support the proposed building. Based on the currently available information, it is expected that the disturbed soils will extent to at least 7.5 metres below the existing ground surface.

Due to the expected depth at which undisturbed native soils will be encountered it is recommended that a foundation system consisting of helical piles be used. It is recommended that the proposed residential building be founded on a helical piles foundation bearing in the undisturbed silty clay or glacial till in combination with cast in place concrete pile caps and/or footings and foundation walls.

The helical piles are a proprietary foundation system and should be designed by a specialist and the specific pile selected shall be based on the soil conditions identified in the report.

Frost Protection

All exterior foundation elements (pile caps and/or footing and bottom of foundation wall) and those in any unheated parts of the proposed building should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated foundation elements adjacent to surfaces, which are cleared of snow cover during winter months should be provided with a minimum 1.8 metres of earth cover. Where less than the required depth of soil cover can be provided, the foundation elements 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, if required.

Foundation Wall Backfill and Drainage

The fill and native silty clay materials 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 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.

A conventional, perforated perimeter drain, with a 150 millimetre surround of 20 millimetre minus crushed stone, should be provided at founding level and should lead by gravity flow to a sump. The sump should be equipped with a backup pump and generator. The proposed basement should also be provided with under floor drains consisting of perforated pipe with a surround of 20 millimetre minus crushed stone to reduce the potential for buildup of hydrostatic pressure below the basement floor. The under floor drains should be placed beginning at the inside edge of the foundation wall and should be spaced a maximum of 5 metres apart. The under floor drain should also be directed to the sump. The sump discharge should be equipped with a backup flow protector.

Concrete Basement Floor Slab

It is expected that the excavation for the proposed building will extend through the exiting fill to the native silty clay. For predictable performance of the proposed concrete floor slab all existing topsoil, fill and any, soft, loose 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. Should complete removal of all deleterious material result in a subgrade below the basement floor structure, the subgrade can be built up using engineered fill.

Engineered fill materials provided to support the concrete floor slab should consist of sand, or sand and gravel meeting the Ontario Provincial Standards Specifications (OPSS) for Granular B Type I or crushed stone meeting OPSS grading requirements for Granular B Type II. A minimum 150 millimetre thickness of crushed stone meeting OPSS Granular A should be provided immediately beneath the concrete floor slab. The engineered fill materials should be compacted in maximum 300 millimetre thick lifts to at least 98 percent of the standard Proctor maximum dry density. Alternatively clear crushed 20 mm minus stone could be used immediately below the concrete floor slab provided the clear stone is well compacted prior to concrete placement.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage and expansion 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..

Seismic Design for the Proposed Residential Building

For seismic design purposes, in accordance with the 2012 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class D.

Borehole 1							
Layer	Description	Depth (m)	d_i (m)	$N(60)_i$ (blows / 0.3m)	S_{ui} (kPa)	d_i/N_i	d_i/S_{ui} (m/kPa)
1	Silty Clay	2.5	5.5		105		0.052
2	Glacial Till	8.0	4.9	18		0.327	
3	Bedrock	12.9	19.6				
	$d_c/(\text{sum}(d_i/S_{ui}))$						105
	$d_c/(\text{sum}(d_i/N(60)_i)$					18	

Since the N(60) = 15 < 18 < 50, the seismic site response is Site Class D.

Potential for Soil Liquefaction

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

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

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

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

From the laboratory test results, the silty has a plasticity index PI = of 54.4 and a liquid limit of 82.2. The clay content from the laboratory sample tested was about 51% for when clay is defined as grains finer than 0.002 mm. As such the silty clay is not prone to liquefaction.

National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.280 with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The results of the test are attached following the text of this report.

SITE SERVICES

Excavation

The excavations for the site services will be carried out through fill materials and silty clay. For the purposes of Ontario Regulation 213/91 the soils at the site can be considered to be Type 2 soil. The sides of the excavations in overburden materials should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Ontario Occupational Health and Safety Act. That is, open cut excavations with overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box.

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

Pipe Bedding and Cover Materials

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

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A or Granular B Type I (with a maximum particle size of 25 millimetres).

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

Trench Backfill

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

In areas where the service trench will be located below or in close proximity to existing or future pavement areas, acceptable native materials should be used as backfill between the pavement subgrade level and the depth of seasonal frost penetration (i.e. 1.8 metres below finished grade) in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway.

Where native backfill is used, it should match the native materials exposed on the trench walls. Some of the native materials from the lower part of the trench excavations may be wet of optimum for compaction. Depending on the weather conditions encountered during construction, some drying of materials and/or recompaction may be required. Any wet materials that cannot be compacted to the required density should either be wasted from the site or should be used outside of existing or future driveway areas. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I. If the native material is not suitable for backfill, imported granular material may have to be used. If imported granular materials are used, suitable frost tapers should be used OPSD 802.013.

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

ACCESS ROADWAY PAVEMENTS

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

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

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

For pavement areas subject to cars and light trucks the pavement should consist of:

50 millimetres of Superpave 12.5 asphaltic concrete over
150 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase over

(50 or 100 millimetre minus crushed stone)

Non-woven geotextile fabric (4 oz/sy) such as Terrafix 270R or Thrace-Ling 130EX or approved alternative.

Performance grade PG 58-34 asphaltic concrete should be specified. Compaction of the granular pavement materials should be carried out in maximum 300 millimetre thick loose lifts to 100 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment.

The above pavement structures will be adequate on an acceptable sub-grade, that is, one where any roadway fill and service trench backfill has been adequately compacted. If the roadway sub-grade is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase. The adequacy of the design of the pavement thickness should be assessed by the geotechnical personnel at the time of construction.

EFFECTS OF TREES

This site is underlain by deposits of sensitive marine clay (SMC), a material which is known to be susceptible to shrinkage with a change/reduction in moisture content. Research by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada has shown that trees can cause a reduction of moisture content in the sensitive silty clays in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations bearing on or above the silty clay. These recommendations have been modified as part of the City of Ottawa's March 2015 Building Better and Smarter Suburbs: Strategic Directions and Action Plan report. The modified guidelines are titled Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines

The modified Guidelines indicate that where SMC soils have been identified, the tree to foundation setbacks may be reduced to 4.5m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5m-14m) as all of six conditions are met or can be met as follows:

1) The soils tests indicate that the plasticity index is below 40%.

Since the SMC soils at this site have a plasticity index of greater than 40%, the modified guidelines do no apply and the minimum required setback for trees is equal to 1 times the mature height of the tree.

It is considered that small could be used at the site provided sufficient setback is maintained. The use of medium and large trees is likely not feasible.

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 building should be inspected by Kollaard Associates Inc. to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundations should be inspected to ensure that the materials used conform to the grading and compaction specifications.

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

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

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

Regards,
Kollaard Associates Inc.

LICENSED PROFESSIONAL ENGINEER
April 2, 2020
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve DeWit, P.Eng.

Attachments: Record of Boreholes
Figures 1 and 2
Laboratory Test Results for Chemical Properties
Laboratory Test Results for Physical Properties – Stantec Laboratory Test Results for Soils

APPENDIX A – SUMMARY OF GEOTECHNICAL RECOMMENDATIONS

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

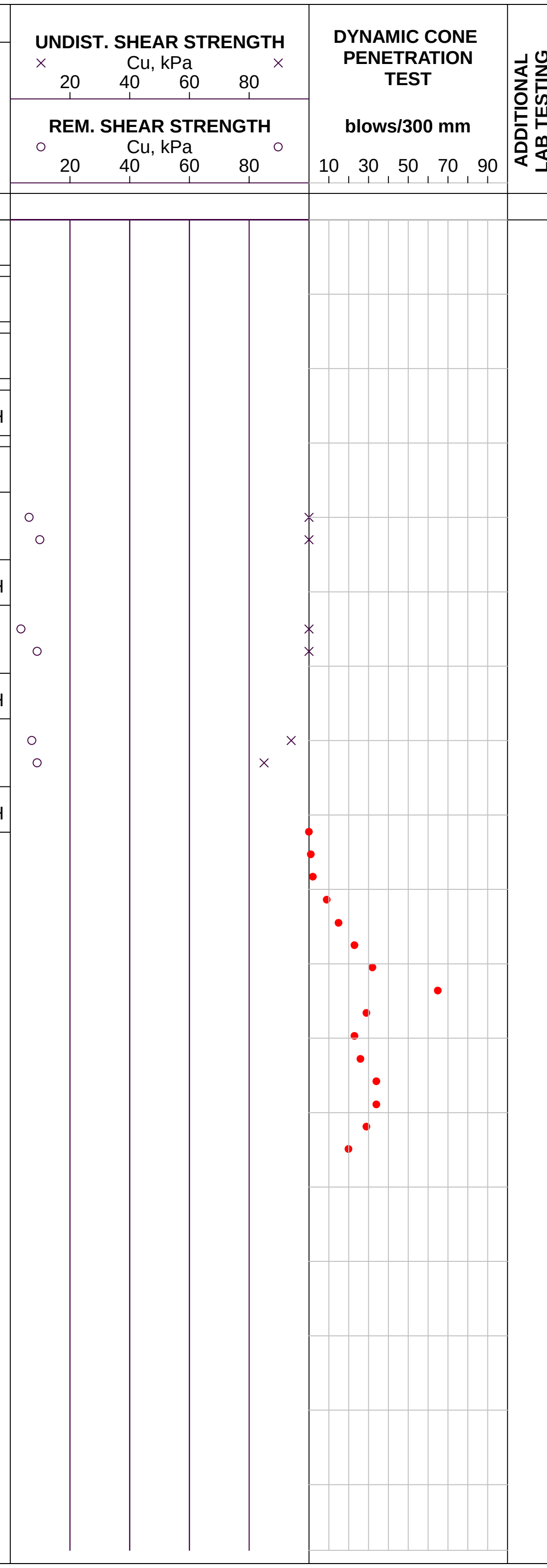
These geotechnical recommendations include:
Foundation Design
Allowable Bearing Capacity
Settlement
Subgrade preparation
Engineered Fill and Compaction
Frost Protection
Foundation Drainage
Foundation Backfill
Floor Slab
Seismic Design
Excavation for Services and Sewers
Bedding and Cover
Trench Backfill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

RECORD OF BOREHOLE BH1

PROJECT: Proposed Residential Development
CLIENT: Falsetto Homes Inc.
LOCATION: 58 Florence Street, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 190186
DATE OF BORING: May 22, 2019
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface					
0	ASPHALTIC CONCRETE	0.00	1	SS	4	
	Grey crushed stone (FILL)					
1	Red brown silty sand, trace gravel (FILL)		2	SS	2	
2			3	SS	6	
	Grey medium to coarse SAND	2.10				
	Grey SILTY CLAY		4	SS	WH	
3			5	SS	1	
4						Borehole dry, May 22, 2019.
5			6	SS	WH	
6			7	SS	WH	
7						
8	Grey silty sand, trace gravel, cobbles, boulders and clay (GLACIAL TILL)	7.95	8	SS	WH	
9	Borehole continued as Probe Hole, probably grey silty clay, then grey silty sand, with some gravel, cobbles and boulders (GLACIAL TILL)					
13	End of Borehole, refusal on large boulder or bedrock	12.83				



DEPTH SCALE: 1 to 75
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH2

PROJECT: Proposed Residential Development
CLIENT: Falsetto Homes Inc.
LOCATION: 58 Florence Street, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 190186
DATE OF BORING: May 22, 2019
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m
	Ground Surface				
0	ASPHALTIC CONCRETE	0.00			
	Grey crushed stone (FILL)		1	SS	7
1	Red brown silty sand, trace gravel (FILL)		2	SS	2
2			3	SS	4
	Grey medium to coarse SAND	2.10			
3	Grey SILTY CLAY		4	SS	WH
5			5	SS	WH
6			6	SS	WH
8			7	SS	2
	End of Borehole	8.20			

UNDIST. SHEAR STRENGTH × Cu, kPa 20 40 60 80
REM. SHEAR STRENGTH ○ Cu, kPa 20 40 60 80
DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90
ADDITIONAL LAB TESTING
PIEZOMETER OR STANDPIPE INSTALLATION: Borehole dry, May 22, 2019.

DEPTH SCALE: 1 to 75
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

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

PENETRATION RESISTANCE

Standard Penetration Resistance, N

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

Dynamic Penetration Resistance

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

WH

Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drih rig.

PM

Sampler advanced by manual pressure.

SOIL TESTS

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

SOIL DESCRIPTIONS

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

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

LIST OF COMMON SYMBOLS

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

KEY PLAN	FIGURE 1

NOT TO SCALE

Kollaard Associates
Engineers

Project No. 190186

Date June 2019

DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

BH1 APPROXIMATE BOREHOLE LOCATION

REFERENCE: PLAN SUPPLIED BY CITY OF OTTAWA EMAPS.

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

REV.	NAME	DATE	DESCRIPTION

Kollaard Associates
Engineers

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

CLIENT:
FALSETTO HOMES INC.

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION:
58 FLORENCE STREET
CITY OF OTTAWA, ONTARIO

DESIGNED BY:	DATE:
--	MAY 9, 2019
DRAWN BY: DT	SCALE: N.T.S

KOLLAARD FILE NUMBER:
190186

Laboratory Test Results for Chemical Properties

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

Date Received: 23-MAY-19
Report Date: 30-MAY-19 14:50 (MT)
Version: FINAL

Client Phone: 613-860-0923

Certificate of Analysis

Lab Work Order #: L2278542

Project P.O. #: NOT SUBMITTED
Job Reference: P10186
C of C Numbers:
Legal Site Desc:

Melanie Moshi
Account Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2278542-1 BH15S 7'5 - 9'5							
Sampled By: CLIENT on 22-MAY-19							
Matrix: SOIL							
Physical Tests							
Conductivity	0.320		0.0040	mS/cm		29-MAY-19	R4648810
% Moisture	43.3		0.10	%	27-MAY-19	28-MAY-19	R4645159
pH	8.30		0.10	pH units		28-MAY-19	R4645902
Resistivity	3130		1.0	ohm*cm		29-MAY-19	
Leachable Anions & Nutrients							
Chloride	0.00785		0.00050	%	29-MAY-19	30-MAY-19	R4651253
Anions and Nutrients							
Sulphate	0.0073		0.0020	%	28-MAY-19	28-MAY-19	R4648271

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

Reference Information

Test Method References:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Chain of Custody Numbers:

GLOSSARY OF REPORT TERMS

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

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

Laboratory Test Results for Physical Properties

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

Stantec

June 11, 2019
File: 122410003

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

Reference: **Kollaard**

File #190186

ASTM D4318 Atterberg Limit & ASTM D2216 Moisture Content

The table below summarizes Atterberg Limit & Moisture Content results.

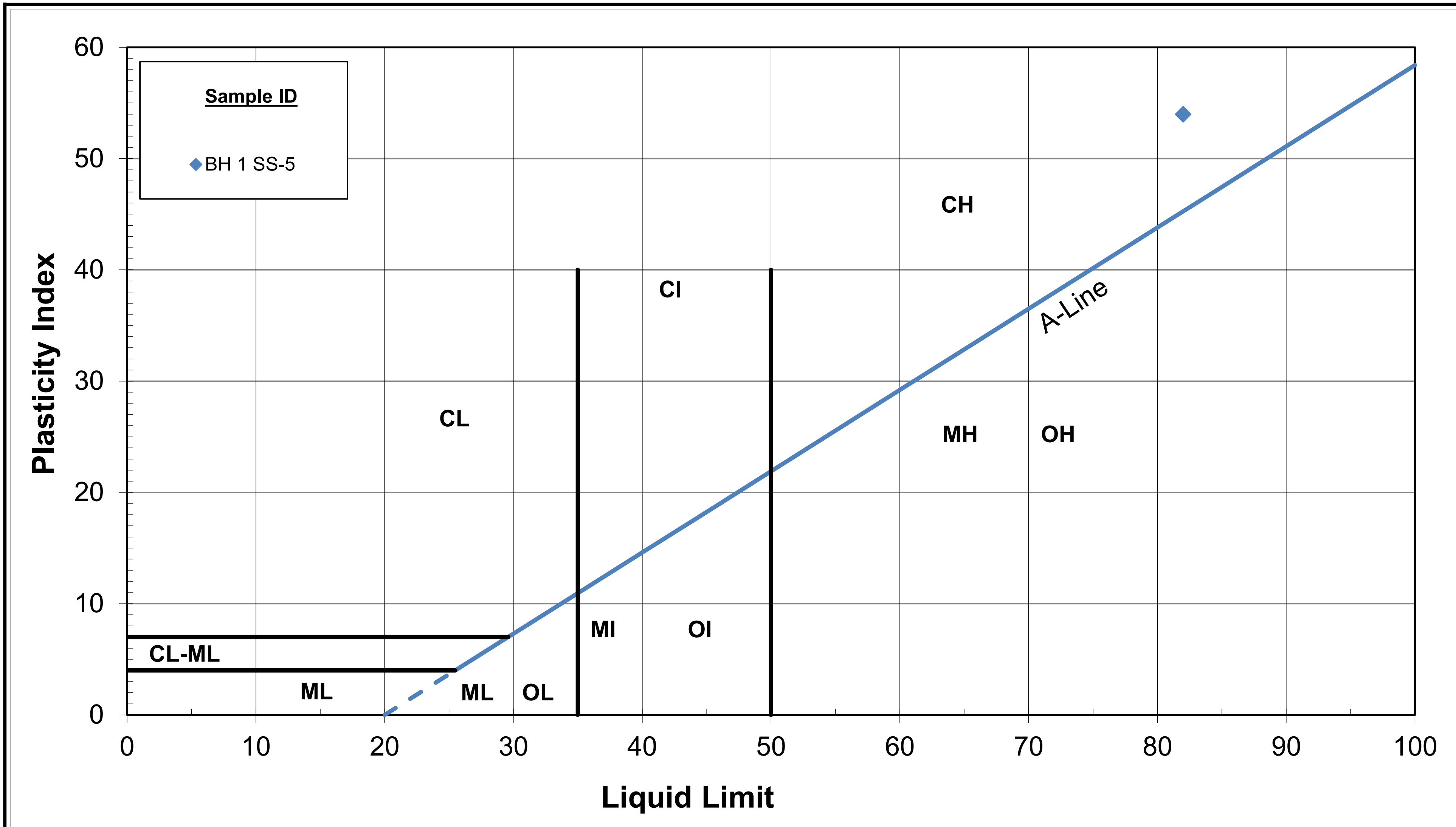
Source	Depth	Natural Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index
BH1 SS5	10'-12'	73.6%	82.2	27.8	54.4

Sincerely,

Stantec Consulting Ltd

Denis Rodriguez
Laboratory Technician
Tel: 613-738-6075
Fax: 613-722-2799
denis.rodriguez@stantec.com

Attachments: Atterberg Limit Plasticity Chart



Stantec

Kollaard Associates, File #190186

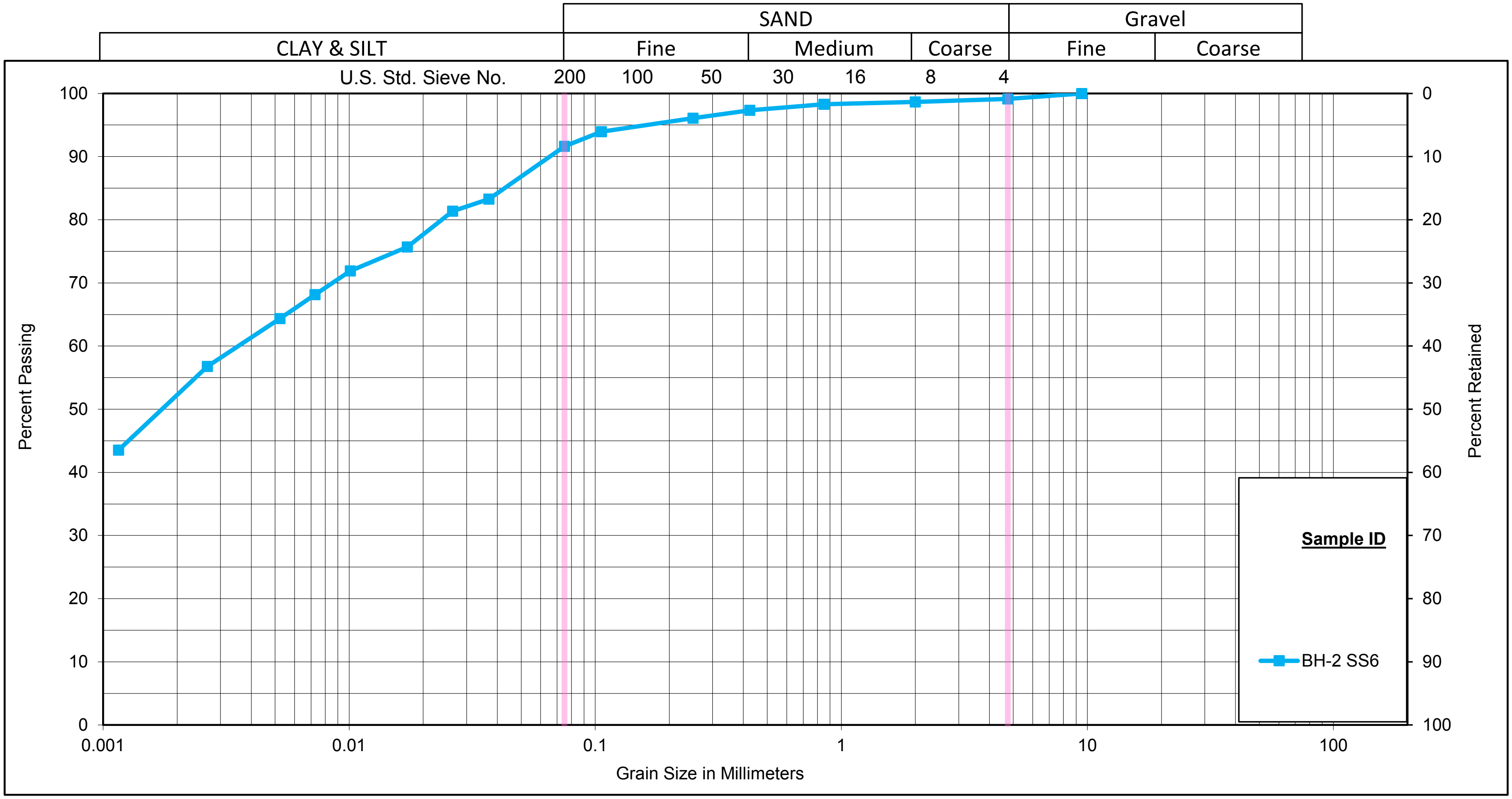
58 Florence Street

PLASTICITY CHART

Figure No.

Project No. 122410003

Unified Soil Classification System



Sample ID	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH-2 SS6	20'-22'	1.0	7.4	40.6	51.0

Stantec

GRAIN SIZE DISTRIBUTION

Kollaard Associates, File #190186

58 Florence Street, Ottawa

Figure No.

Project No. 122410003

Stantec

Particle-Size Analysis of Soils

LS702

ASTM D422

PROJECT DETAILS

Client:	**Kollaard Associates, File #190186**	Project No.:	**122410003**
Project:	**58 Florence Street, Ottawa**	Test Method:	**LS702**
Material Type:	**Soil**	Sampled By:	**Kollaard Associates**
Source:	**BH-2**	Date Sampled:	**May 22, 2019**
Sample No.:	**SS6**	Tested By:	**Daniel Boateng**
Sample Depth	**20'-22'**	Date Tested:	**June 10, 2019**

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	51.00
Sample Weight after Hydrometer and Wash (g)	3.70
Percent Passing No. 200 Sieve (%)	**92.7**
Percent Passing Corrected (%)	**91.50**

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	**141.40**
Sample Weight After Sieve (g)	**140.80**
Percent Loss in Sieve (%)	**0.42**

SOIL INFORMATION

Liquid Limit (LL)		
Plasticity Index (PI)		
Soil Classification		
Specific Gravity (G_s)	2.750	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	40	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	86.22
Air Dried Mass (W_a), (g)	87.23
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9884
Air Dried Mass in Analysis (M_a), (g)	51.60
Oven Dried Mass in Analysis (M_o), (g)	51.00
Percent Passing 2.0 mm Sieve (P_{10}), (%)	98.66
Sample Represented (W), (g)	51.70

SIEVE ANALYSIS

Sieve Size mm	Cum. Wt. Retained	Percent Passing
75.0		100.0
63.0		100.0
53.0		100.0
37.5		100.0
26.5		100.0
19.0		100.0
13.2		100.0
9.5	0.0	100.0
4.75	1.2	99.2
2.00	1.9	98.7
Total (C + F)[1]	140.80	
0.850	0.18	98.31
0.425	0.68	97.34
0.250	1.33	96.08
0.106	2.43	93.96
0.075	3.63	91.63
PAN	3.70	

Note 1: (C + F) = Coarse + Fine

HYDROMETER DETAILS

Volume of Bulb (V_B), (cm^3)	63.0
Length of Bulb (L_2), (cm)	14.47
Length from '0' Reading to Top of Bulb (L_1), (cm)	10.29
Scale Dimension (h_s), (cm/Div)	0.155
Cross-Sectional Area of Cylinder (A), (cm^2)	27.25
Meniscus Correction (H_m), (g/L)	1.0

START TIME 8:56 AM

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
10-Jun-19	8:57 AM	1	51.0	7.0	23.0	44.0	83.27	8.30904	9.39251	0.012818	0.03695
10-Jun-19	8:58 AM	2	50.0	7.0	23.0	43.0	81.38	8.46404	9.39251	0.012818	0.02637
10-Jun-19	9:01 AM	5	47.0	7.0	22.5	40.0	75.70	8.92904	9.50295	0.012894	0.01723
10-Jun-19	9:11 AM	15	45.0	7.0	22.5	38.0	71.92	9.23904	9.50295	0.012894	0.01012
10-Jun-19	9:26 AM	30	43.0	7.0	22.5	36.0	68.13	9.54904	9.50295	0.012894	0.00727
10-Jun-19	9:56 AM	60	41.0	7.0	22.5	34.0	64.35	9.85904	9.50295	0.012894	0.00523
10-Jun-19	1:06 PM	250	37.0	7.0	22.0	30.0	56.7751	10.47904	9.61570	0.012970	0.00266
11-Jun-19	8:56 AM	1440	30.0	7.0	22.5	23.0	43.5276	11.56404	9.50295	0.012894	0.00116

Remarks:

Reviewed By: Denis Rodriguez

Date: June 11, 2019

V:\01216\active\laboratory_standing_offers\2019 Laboratory Standing Offers\122410003 Kollaard Associates Engineers\May 22, Limit, Hyd & MC, Kollaard #190186\Hydrometer Analysis MTO Projects May2014.xlsx

National Building Code Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

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

Site: 45.412N 75.696W **User File Reference:** 58 Florence Street 2019-06-25 15:01 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)	0.446	0.247	0.148	0.044
Sa (0.1)	0.523	0.300	0.186	0.061
Sa (0.2)	0.439	0.255	0.161	0.055
Sa (0.3)	0.334	0.195	0.124	0.043
Sa (0.5)	0.237	0.138	0.088	0.031
Sa (1.0)	0.118	0.069	0.044	0.015
Sa (2.0)	0.056	0.033	0.020	0.006
Sa (5.0)	0.015	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.280	0.163	0.102	0.033
PGV (m/s)	0.197	0.111	0.068	0.021

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.**

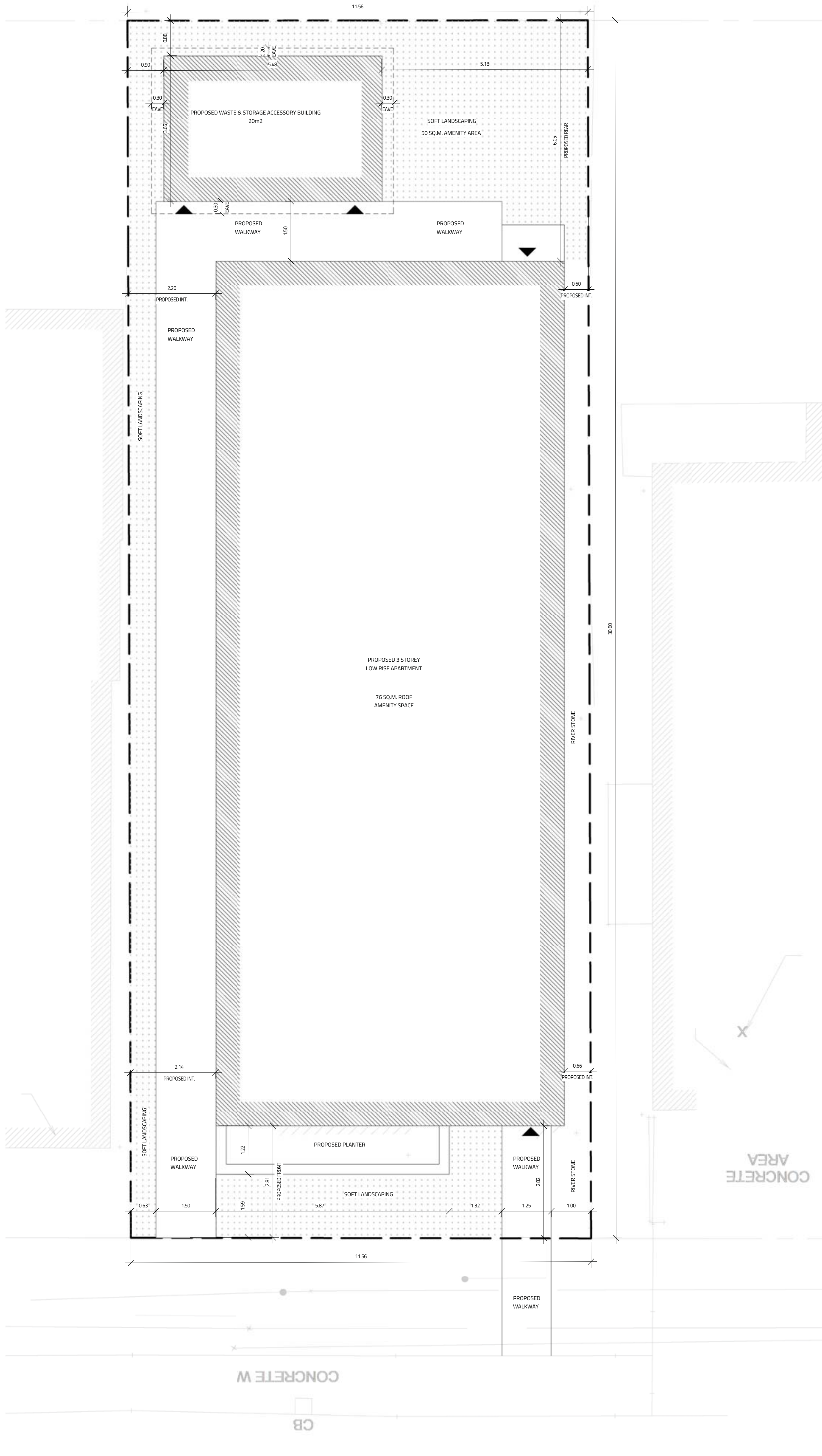
References

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



ALL TOPOGRAPHIC INFORMATION IS DERIVED FROM J.D BARNES LIMITED LAND INFORMATION SPECIALISTS DRAWING NUMBER 19-100-09-00 DATED MARCH 25, 2019

NOT FOR CONSTRUCTION

NO.	REVISION	DATE
3	ACCESSORY BUILDING REVISION	DEC 16, 2019
2	RE-ISSUED FOR SITE PLAN CONTROL	OCT 18, 2019
1	ISSUED FOR SITE PLAN CONTROL	JULY 29, 2019

MODEL:
58 FLORENCE

GROUND FLOOR: 1890 SQ. FT.
SECOND FLOOR: 1960 SQ. FT.
THIRD FLOOR: 1960 SQ. FT.
ROOF TOP: 220 SQ. FT.

OTTAWA, ON

Evolution DESIGN & DRAFTING
613-808-7185 // 613-884-7068

DRAWING TITLE: **SITE PLAN**

DATE DRAWN: JULY 5, 2019	SCALE: 1 : 50
DRAWN BY: SG	FILE NAME: 58 FLORENCE
CHECKED BY: MV	DWG. NO. D0.0