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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
800 EAGLESON ROAD
CITY OF OTTAWA, ONTARIO

Project # 180084

Submitted to:

Ironclad Developments Inc.
101-57158 Symington Road 20E
Springfield, MB R2J 4L6

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February 27, 2018

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Ironclad Developments Inc.
101-57158 Symington Road 20E
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RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
800 EAGLESON ROAD
CITY OF OTTAWA, ONTARIO

Dear Sirs:

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

BACKGROUND INFORMATION AND SITE GEOLOGY

The subject property for this assessment is located at 800 Eagleson Road in the City of Ottawa, Ontario (see Key Plan, Figure 1). The site consists of a triangular shaped parcel of land some 2.82 hectares (6.98 acres) in plan area located to the southwest of the intersection of Fernbank Road and Eagleson Road in the City of Ottawa, Ontario.

Based on information provided by Ironclad Developments Inc., it is understood plans are being prepared to construct a 6 storey, 148 unit apartment development with one storey of underground parking. It is understood that the total building area will be about 13,681 square metres. At the time of the investigation, the property was tree covered. The site has about 109 metres of frontage to the east onto Eagleson Road. The site is bound to the northwest by Fernbank Road and to the south by the Monahan Drain / City of Ottawa Stormwater Management Pond.

The site is located in an area of existing commercial, institutional and residential development. At the time of this investigation, the existing ground surface of the site is relatively low lying with an average elevation of about 2 to 2.5 metres below the centerline elevation of Fernbank Road and about 1.5 to 2 meters below the centerline elevation of Eagleson Road. The site slopes towards the existing City of Ottawa Stormwater Management Pond which has a normal water level of about 1.8 metres below the average existing ground surface elevation of the site and a 100 year flood level of about 0.4 metres above the average existing ground surface elevation of the site.

Preliminary plans indicate that the proposed building will be of concrete or steel framed construction with a cast-in-place concrete foundation and slab on grade concrete parking garage floor. The proposed building will be serviced by municipal water and sanitary services and will be provided with access roadways and additional exterior parking areas. Surface drainage for the proposed development will be by means of swales, catch basins and storm sewers.

A review of the existing surficial geology map for the site indicates the site is underlain by organic deposits mostly muck and peat and/or clay, silty clay and silt. The factual information obtained from a previous geotechnical investigation carried for the site in 2013 indicates that there will be a relatively thin layer of topsoil underlain by a stiff silty clay crust followed by firm to soft silty clay to depths of some 26 metres. Bedrock geology maps indicate that the bedrock underlying the site consists of limestone or dolostone of the Ottawa formation.

A Ministry of Environment of water well record was obtained for a drilled cased well installed approximately 30 meters east of the site. The water well record is attached following the text of this report. The water well record indicates that limestone bedrock was encountered at a depth of 110 feet (33.5 metres) below the existing ground surface.

From overburden thickness maps and geotechnical investigations completed in the vicinity of the site, it is expected that bedrock will be encountered between 25 and 30 metres below the existing ground surface.

PROCEDURE

The field work for this investigation was carried out between February 13 and February 16, 2018 at which time eight boreholes, numbered BH1 to BH8 inclusive, were put down at the site using a track mounted drill rig equipped with a hollow stem auger owned and operated by Marathon Drilling Co. Ltd. of Greely, Ontario.

Sampling of the overburden materials encountered at the borehole location was carried out at using a 50 millimetre diameter drive open conventional split spoon sampler (ASTM D-1586 – Penetration Test and Split Barrel Sampling of Soils) in conjunction with standard penetration testing. In boreholes BH1 and BH2, the sampling was completed at intervals of 0.75 metres to depth of about 2.3 metres, then in intervals of 1.5 metres to depths of about 8.2 metres followed by intervals of about 3.0 metres to the depths ranging from 25.6 to 28 metres. In BH3 to BH8, the sampling was completed at regular 0.75 metres intervals to depths ranging from about 2.9 to 4.4 metres. In situ vane shear testing (ASTM D-2573 Standard Test Method for Field Shear Test in Cohesive Soil) was completed at intervals in the cohesive materials encountered within boreholes BH1 and BH2 between the depths of about 2.5 and 25 metres.

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), standard penetration test results (ASTM D-1586), and in situ vane shear testing (ASTM D-2573) as well as laboratory test results on select samples. Groundwater conditions at the borehole was noted at the time of drilling. A standpipe was installed at BH3 for subsequent groundwater level monitoring. The boreholes were loosely backfilled with the auger cuttings upon completion of drilling.

Five soil samples selected from BH1, BH2, BH5 and BH7 were submitted for Atterberg Limits, Particle Size Analysis and Moisture Content Analysis (ASTM D4318, ASTM D422 and ASTM D2216). The samples were selected based on depth and tactile examination to be representative of the various soil conditions encountered at the site. A sample of soil obtained from BH2 was also delivered to a chemical laboratory for testing for any indication of potential soil sulphate attack and soil corrosion on buried concrete and steel. A total of thirty-nine soil samples recovered from the boreholes were also tested for moisture content (ASTM D2216).

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. The soils were classified in the field based on visual and tactile inspection (ASTM D2488) and by the results of the in situ shear vane tests (ASTM 2573) and standard penetration tests (ASTM D1586). Classifications were confirmed by laboratory testing by test methods conforming to ASTM D4318, ASTM D2216 and ASTM D422.

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 existing ground surface elevations at the borehole locations are geodetic based on topographic information obtained from D. B. Gray Engineering Inc., Drawing C-2, Grading Plan and Erosion & Sediment Control Plan, Revision 1, dated March 10, 2014.

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

Topsoil

Peaty topsoil was encountered from the surface at all of the boreholes. The topsoil thickness at the boreholes ranged from about 0.45 to 1.05 metres. The material was classified as topsoil based on colour and the presence of organic materials. The identification of the topsoil layer is for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustainable plant growth. The topsoil was fully penetrated at the borehole locations.

Silt/Silty Sand

A deposit of grey brown silty sand/silt with a trace to some clay was encountered beneath the topsoil at BH2, BH3, and BH5. The deposits of silty sand/silt with a trace to some clay ranged in thickness from about 0.15 to 0.5 metres and extended from the underside of the topsoil layer to between about 0.45 to 1.06 metres below the existing ground surface where encountered. The results of standard penetration testing carried out in the silty sand material, indicates a loose state of packing.

Silty Clay

Two deep boreholes (BH1 and BH2) and six shallow boreholes (BH3, BH4, BH5, BH6, BH7 and BH8) were put down at the site. A deposit of grey brown to grey silty clay was encountered below the topsoil and/or silty sand at all of the boreholes. In situ vane shear tests carried out in the silty clay deposits at BH1 and BH2 gave undrained shear strength values ranging from about 17 kilopascals to 66 kilopascals. One vane shear test conducted within BH1 at a depth of 9.9 metres gave an undrained shear strength of 12 kPa; however this low value is not considered to be representative of the shear strength of the silty clay deposit and is like a result of disturbance during the drilling operation. 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 soft to stiff in consistency. The thickness of the silty clay deposit was determined to be about 25.14 and 24.08 metres, respectively, for BH1 and BH2. The silty clay deposit was fully penetrated at both deep borehole locations.

As mentioned above, six shallow boreholes were put down at the site and were advanced to depths of between about 2.9 and 4.2 metres below the existing ground surface. The upper portion of the silly clay encountered in general consisted of a weathered silty clay to depths of about 2.5 to 3 metres. The upper portion of the silty clay deposit contains trace sand to depths of about 1 to 2 metres. Standard penetrate test N values ranging from 1 to 9 blows per 0.3 metres of penetration and in situ shear vane test results of 60 kPa were obtained within the silty clay crust indicating a stiff to very consistency. The silty clay crust was underlain by deposits firm to soft silt clay which extended to between 25.1 and 24.1 metres below the existing ground surface.

As previously indicated, the in situ shear vane test results carried out in the silty clay deposits gave undrained shear strength values ranging from about 17 to 42 kilopascals below the upper crust. The remolded vane shear strength values ranged from 1.5 to 7.5 kilopascals providing sensitivity ratios in the silty clay ranging from 4.5 to 13 above a depth of 15.5 meters and from 12 to 21 between 15.5 metres and the depth of the deposit. Based on the Canadian Foundation Engineering Manual, silty clays with a sensitivity of $4 < St < 8$ are considered to be sensitive, silty clays with a sensitivity of $8 < St < 16$ are extra-sensitive, and silty clays with a sensitivity of $St > 16$ are considered to be quick clay.

The measured water contents of samples of the weathered silty clay ranged from about 12 to 58 percent. The results are present in Attachment A at the end of the report.

Five samples of silty clay, representative of the various soil conditions encountered at the site (BH1-SS3-1.52-2.13, BH1-SS10-16.0-16.6, BH2-SS8-9.91-10.5, BH5-SS6-3.81-4.42 and BH7-SS2-0.76-1.37 metres) were submitted to the Stantec soils laboratory for Atterberg Limits testing (ASTM D4318), Moisture Contents, (ASTM D2216) and Particle Size Analysis (hydrometer testing ASTM D422)

The results of Atterberg Limits and moisture content tests conducted on five soil samples of silty clay are presented in Table I and in Attachment A at the end of the report. Based on the Atterberg limit test results, four of the samples tested (BH1-SS3, BH2-SS8, BH5-SS6 and BH7-SS2) are classified as silt clays of low plasticity (CL), and one sample (BH1-SS10) classified as silty clay of with intermediate plasticity (CI) in accordance with the Unified Soil Classification System.

Table I – Atterberg Limit and Water Content Results

Sample	Depth(metres)	LL (%)	PL (%)	PI (%)	W (%)
BH1-SS3	1.52 - 2.13	32.0	16.5	15.5	28.0
BH1-SS10	16.0 - 16.6	35.8	23.0	12.7	49.5
BH2-SS8	9.91 - 10.5	24.6	15.9	8.7	36.3
BH5-SS6	3.81 - 4.42	31.7	19.8	11.9	36.6
BH7-SS2	0.76 - 1.37	31.7	16.8	14.3	26.4

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

Atterberg Limit tests completed during the 2013 provide similar results to those indicated above and have been included for reference purposes in the following table.

Atterberg Limit and Water Content Results for 2013 Investigation

Sample	Depth(metres)	LL (%)	PL (%)	PI (%)	W (%)
BH2-SA1	1.5 - 1.8	46.4	18.9	27.4	35.5
BH2-SA3	2.3 – 2.7	37.2	17.7	19.6	36.7

The above Atterberg limit test results obtained from the samples submitted during the current investigation were compared to the results from the 2013 investigation. In addition, the Atterberg limit test results were compared to the clay content as indicated in the particle size analysis presented in the following section. Based on the comparisons, it is considered that the sample represented by BH2-SS8 is not indicative of the subsurface silty clay conditions at the site. It is considered likely that the sample selected for Atterberg Limit testing became contaminated during the sampling and handling process.

The results of the Particle Size Analysis (hydrometer) conducted on five soil samples of silty clay are presented in Table II and in Attachment A at the end of the report. The results of the hydrometer testing indicated that the samples consisted of about 40 to 66 percent silt and 27 to 60 percent clay size particles.

Table II – Particle Size Analysis Test Results

Sample	Depth(metres)	Particle Size in Sample (%)			
		Gravel 4.75 to 71 (mm)	Sand 0.075 to 4.75 (mm)	Silt 0.002 to 0.075 (mm)	Clay < 0.002 (mm)
BH1-SS3	1.52 - 2.13	0	3.3	62.7	34
BH1-SS10	16.0 - 16.6	0	1.0	39.0	60
BH2-SS8	9.91 - 10.5	0	3.9	63.1	33
BH5-SS6	3.81 - 4.42	0	3.7	54.3	42
BH7-SS2	0.76 - 1.37	0	6.7	66.3	27

Glacial Till

Glacial till was encountered beneath the silty clay at BH1 and BH2. The glacial till consisted of gravel and cobbles, in a matrix of grey brown sand, with a trace of clay. It is noted that boulders were also encountered within the glacial till. The results of standard penetration testing carried out in the glacial till material, which range from 7 to 29 blows per 0.3 metres with an average value of 18 blows per 0.3 metres, indicate a compact state of packing. It is considered that the low blow counts encountered at varying depths within the glacial till materials are the results of silt/sand layers and wetter soil conditions encountered within the glacial till materials with depth.

Sampling and advancement of Boreholes BH1 and BH2 by drilling was terminated due to refusal to further advancement on the surface of large boulders at depths of approximately 26.7 and 32.1 metres, respectively, below the existing ground surface level.

Borehole BH1 was advanced by coring through cobbles and boulders within the glacial till to a depth of 28.85 metres. The borehole was abandoned due to collapse of the core hole and jamming of core barrel with fractured rock.

Borehole BH2 was advanced by coring through cobbles and a boulder from 32.1 metres below grade to 32.6 metres below grade. The glacial till was fully penetrated in BH2 at a depth of 32.1 metres below grade.

One soil sample of glacial till (BH2 - SS13 - 24.4 to 25.0 metres) was submitted to Stantec for particle size analysis (ASTM D422). The results of the particle size analysis testing indicated that the sample consists of about to 10 percent clay, 68 percent fine to coarse sand and about 22 percent gravel. The sample is indicated to have between about 42 percent silt and clay size particles. The results are located in Attachment A.

Bedrock

The boreholes were advanced by coring to verify the existence of bedrock and assess depth to sound bedrock. At borehole BH2, the corehole was advanced from 32.6 to 34.29 metres below the existing ground surface or 1.69 metres into bedrock. The core from BH2 was recovered and measured to determine the following: Total Core Recovery = 100%, Solid Core Recovery = 100%, Rock Quality Designation = 92%.

Groundwater

Groundwater was encountered in boreholes BH1, BH2, BH4, BH6 and BH8 at the time of drilling between February 13 and 16, 2018 at depths ranging from about 1.4 to 2.1 metres below the existing ground surface. Groundwater was measured in a standpipe installed within BH5 at depth of about 0.5 metres below the existing ground surface on February 20, 2018. Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

Corrosivity on Reinforcement and Sulphate Attack on Portland Cement

The results of the laboratory testing of a soil sample for sulphate gave a percent sulphate of <0.02. The National Research Council of Canada (NRC) recognizes four categories of potential sulphate attack of buried concrete based on percent sulphate in soil. From 0 to 0.10 percent the potential is negligible, from 0.10 to 0.20 percent the potential is mild but positive, from 0.20 to 0.50 percent the potential is considerable and 0.50 percent and greater the potential is severe. Based on the above, the soils are considered to have a negligible potential for sulphate attack on buried concrete.

The results of the laboratory testing of a soil sample for resistivity and pH indicates the soil sample tested has an underground corrosion rate of about 0.70 loss-oz./ft^2/yr. Based on the findings of Fischer and Bue (1981) underground corrosion rates (loss-oz./ft^2/yr) of 0.30 and less are considered nonaggresive, from 0.30 to 0.75 the rate is considered slightly aggressive, from 0.75 to 2.0 the rate is considered aggressive and 2.0 and greater the rate is considered very aggressive. Accordingly, the above mentioned soil sample is considered to have a slightly aggressive rate corrosion rate on buried steel.

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 %	0.023	Negligible concern
pH	5.0 < pH	7.9	Neutral / Slightly Basic Negligible concern
Resistivity	R < 1500 ohm-cm	2780	Moderate concern
Sulphates (SO_4)	SO_4 > 0.1%	<0.01	Negligible concern

Based on the chemical test results, Type GU General use Hydraulic Cement may be used for this proposed development. No special protection is required for reinforcement steel within the concrete foundation other than the minimum cover required as per CAN/CSA A23.

GEOTECHNICAL DESIGN 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.

Seismic Design Considerations for the Proposed Residential Building

Based on the limited information from the test pits, for seismic design purposes, in accordance with the 2012 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class E. The assumed basement floor level is about 1.5 metres below the existing ground surface.

Seismic Site Response Site Class Calculation

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	1.5	2.5		66		0.037
2	Silty Clay	4.0	11.6		20.3		0.572
3	Silty Clay	15.5	9.6		39.0		0.246
4	Glacial Till	25.1	N/A				
	$sum(d_i/S_{ui})$						0.855
	$d_c/(sum(d_i/S_{ui}))$						27.6

Since S_u = 27.6 < 50 kPa the seismic site response is Site Class E.

National Building Code Seismic Hazard Calculation

The design Peak Ground Acceleration (PGA) for the site was calculated as 0.260 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.

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 firm to soft clays to depths ranging from about 24.1 to 25.1 metres.

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.

C.F.E.M. section 6.6.3.2 (7) discusses residual strength for silts and clays, that is it recommends that the residual strength for silt and clay zones be determined as per the following guidelines given below:
a) $W_c/LL \geq 0.85$ and $PI \leq 12$: Sr = remolded shear strength,
b) $W_c/LL \geq 0.8$ and $12 < PI < 20$ Sr = 0.85 Su where Su = static undrained shear strength
c) $W_c/LL < 0.80$ and $PI \geq 20$: Sr = Su

From the laboratory test results, the silty clay samples tested had plasticity indexes PI = of 8.7 to 15.5 and liquid limits of 24.6 to 35.8. Including the test results for the site from 2013 and excluding the test results from BH2-SS8l the plasticity indexes range from 11.9 to 27.4 and the liquid limits range from 31.7 to 46.2. The clay content from the laboratory sample tested was about 28 to 62% for when clay is defined as grains finer than 0.002 mm. As such the silty clay is not prone to liquefaction.

With reference to the liquefaction criteria provided in the C.F.E.M manual, and the criteria provided by Seed et al. the silty clay at the site can be considered to be moderately susceptible to liquefaction, that is the soils are not subject to classic cyclic liquefaction but will experience loss of strength with remoulding or monotonic accumulation of shear deformation.

From the C.F.E.M it is considered that the undrained shear strength should be considered to have been reduced to the remolded shear strength values following a significant seismic event of sufficient magnitude to induce shear deformation within the silty clay structure.

FOUNDATIONS

Site Limitations

The site is underlain by a deposit of sensitive silty clay. Based on the undrained shear strength measurements within the silty clay deposit, the silty clay below the weather crust has a soft to firm consistency and has a limited capacity to support loads from footings and grade raise fill. 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 foundation and the thickness of the soils deposit beneath the footings.

The site plan, and site servicing and grading plans have not been completed at the time of this report and have not been provided by the client to the geotechnical engineering for review. It is further understood that building designs are preliminary. Based on the relative difference between the centerline of road elevation of the adjacent road relative to the site, it is assumed that there will likely be a minimum grade raise at the site of 1.5 to 2 metres in order to facilitate the proposed building grades, exterior parking and access roadways.

As previously indicated, it is understood that the proposed building will consist of a 6 storey concrete or steel frame building with one storey of underground parking. It is considered that the existing subsurface soils have insufficient capacity to support the proposed building on convention shallow foundations.

It is considered that the proposed residential building may be founded on a pile foundations such as driven piles deriving support in end bearing on bedrock in combination with cast in place concrete pile caps, grade beams, and/or foundation walls.

End Bearing Piles

Due to the insufficiency of the subsurface soils to support the proposed structure, piles founded on bedrock are considered to be the most suitable foundation system from a geotechnical perspective. There are two pile types considered in this report: Steel H-Piles and Closed End Concrete Filled Steel Pipe Piles.

Pipe Piles

Closed end, concrete filled, steel pipe piles are commonly used in the Ottawa Area to support large loads in end bearing on bedrock. It is anticipated that the pipe piles will be driven to refusal on the underlying limestone or dolostone bedrock at an expected depth of about 33.0 to 34.0 meters below grade.

Recommended values for the design capacity of the pipe piles for typical sections driven to practical refusal in the limestone or dolostone bedrock at the anticipated depths indicated above are as indicated in the following table:

Pipe Diameter (mm)	Wall Thickness (mm)	Geotechnical Reaction at SLS (kN)	Factored Geotechnical Resistance at ULS (kN)
244	12	1,100	1,350
273	13	1,260	1,550
324	9.5	1,580	1,950

Note: The SLS and ULS loads assume that the yield strength of the steel is at least 350 MPa and that the piles are filled with concrete having a minimum compressive strength of 30 MPa.

Steel H-Piles

Steel H-Piles are also considered feasible to support the proposed foundation loads. The steel H-Piles should be driven to refusal on the underlying limestone or dolostone bedrock at an expected depth of about 33.0 to 34.0 metres below grade.

Recommended values for the design capacity of the H-Piles for typical sections driven to practical refusal in the limestone or dolostone bedrock at the anticipated depths indicated above are as indicated in the following table:

Pile Section	Geotechnical Reaction at Serviceability Limit States (kN)	Factored Geotechnical Resistance at ULS (kN)
HP 310 x 79	1,066	1,255
HP 310 x 110	1,500	1,775

Note: The SLS and ULS loads assume that the yield strength of the steel is at least 350 MPa

General Pile Comments

The resistance at serviceability limit states typically allows for 25 mm of compression of the pile and the founding medium. The design is not expected to be governed by settlement since the loading required to produce 25 mm of compression will be much larger than the factored resistance at ULS.

Boreholes BH1 and BH2 encountered glacial till containing significant cobbles and boulders at about 25.1 and 24.1 metres, respectively, below the existing ground surface. It is possible that some of the piles may encounter refusal to driving on or within the bouldery glacial till. The use of a pile with a thick wall may allow penetration of the glacial till with less damage. Notwithstanding, some problems with misalignment, plumbness, bending and/or sweeping of the piles, and hard driving conditions could occur due to the presence of cobbles and boulders above the bedrock surface. As such, allowance should be made to drive additional piles and to enlarge some of the pile caps, etc., as required. The requirement for this, if any, would have to be evaluated at the time of construction.

It is considered that the impact force and transferred energy of the equipment provided by some contractors to drive piles is relatively high and causes the top of the pile to mushroom when driving. It is recommended therefore, that the specifications call for the contractors to mobilize equipment that is compatible with the pile to be driven to minimize the potential for damage.

The following is intended to provide guidance in this regard and is provided for preliminary design and planning purposes only. The hammer type and corresponding set will be subject to the pile driving contractor’s equipment and procedures.

The piles should be driven with a hammer transferring at least 30,000 ft./lb. (41 kilojoules) of energy to the pile to a set consisting of a minimum 20 blows per 25 mm for 3 successive sets (minimum 20 blows per 25 mm for last 75 mm of driving). The required set will be dictated by the pile section selected, the design capacity/axial resistance as well as the transferred energy and impact force on the piles of the hammer selected to install the piles. The actual set should be reviewed when design details are established and confirmed adequate by dynamic analysis during pile installation. Further recommendations regarding the set can be provided when the pile capacity and type of driving equipment are established.

An allowance should be made in the specifications for re-striking all of the piles at least once, after adjacent piles within 4.0 metres distance have been installed to confirm the permanence of the pile set and to check for upward displacement caused by driving adjacent piles. Piles that do not meet the design set criteria on the first re-strike should receive additional re-striking at 2 day intervals. Furthermore, the specifications should make provision for dynamic testing of selected piles during the early stages of the pile driving operations to verify the transferred energies and pile capacities.

Driving shoes should be provided (OPSD 3302.000 Type I, for pipe piles and OPSD 3000.100 Type I for H-piles) to minimize the potential for damage when driving into the till and bedrock.

Pile installation operations should be inspected on a full time basis by qualified geotechnical personnel to ensure the uniformity of set, founding elevation, alignment, plumbness and properly spliced welds.

Seismic Considerations

In accordance with Ontario Building Code requirements (refer to Clause 4.1.9.4), each pile should be interconnected in a minimum of two directions to resist seismic loading. Pile foundation / pile caps supporting column loads must be connected to the adjacent foundation wall system in two directions perpendicular to each other to prevent independent lateral movement during a seismic event. The piles may be tied either by extending the pile into a suitably constructed and reinforced pile cap in the case of steel H-Piles or by extending reinforcement dowels into the concrete filled steel pipe piles during the concrete placement. The design of the pile caps and/or reinforcement is the responsibility of the structural engineer.

Below Grade Basement and Parking Structure Foundation

The topsoil and/or peat is considered to be highly compressible and are not considered suitable for the support of the proposed structures. All topsoil should be removed from the proposed building areas.

As previously indicated, grading plans have not been generated for the proposed building. However, based on the elevation difference between the site and the adjacent center line of road and on a proposed 1 storey basement parking garage, it is expected that the finished floor elevation will be about 1.5 metres below the existing ground surface. It is expected that the foundation walls / pile caps may extend as much as 1 meter below this level and that an elevator pit may extend as much as 2 meters below this elevation. As such it is considered that the foundation excavation may extend to between 3 and 4 meters below the existing ground surface.

The excavations for the foundation will be carried out through topsoil and silty clay. For the purposes of Ontario Regulation 213/91 the upper silty clays soils at the site above a level of about 3.5 meters below grade can be considered to be Type 3 soil. Should the required excavation extend below this depth further evaluation of the excavation side slopes should be made by a qualified geotechnical engineer during excavation. 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.

Where, due to space constraints, adequate side slopes cannot be provided, the excavation walls should be adequately shored. The shoring should be designed to support the lateral earth pressure ‘p’ 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 22 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

The hydrostatic pressure, γ_w H, may be neglected where soldier piles and timber lagging are used as drainage is expected to occur between the lagging and thus no build-up of hydrostatic pressure is likely. Groundwater inflow from the native soils into the excavations during construction, if any should be handled by pumping from sumps within the excavations.

Since ground water was measured at 0.5 metres below the ground surface in the stand pipe within the building footprint at borehole 5 on the site, it is considered that the excavation will extend below the ground water level. It is recommended that registry within the Environmental Activity and Sector Registry water-taking sector for the purposes of construction site dewatering be completed prior to construction.

Building Basement and Below Grade Parking Structure Foundation Walls

As previously indicated, the subsurface sensitive silty clay soils at the site have limited capacity to support additional loading. It is considered that, in order to facilitate the proposed exterior access roadways and parking areas indicated on the site plan in conjunction with the proposed basement parking, there will be a grade raise of up to 2 metres adjacent the exterior foundation walls.

Since the proposed building will be founded on piles, the grade raise will not have a significant effect on the building. It is considered however, that the additional loading resulting from the grade raise will result in a pressure bulb which will extend beneath the building potential causing settlement of the parking area / basement concrete floor. As such it is considered that the grade raise above the existing ground surface be limited to a maximum of 0.5 meters without the use of lightweight fill. The lightweight fill should consist of expanded polystyrene insulation with a compressive resistance at 1% deformation of 25 kPa and a maximum water absorption by total immersion of 4 percent. The lightweight fill should extend a minimum of 2.4 metres horizontally out from the foundation walls.

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

The parking structure and 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:

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.

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.

Seismic Design for the Proposed Residential Building Foundation Walls

Sentence 4.1.8.16.(4) of the 2012 OBC requires that the foundation walls be designed to resist earthquake pressure from backfill or natural ground. The seismic pressure can be modelled by a static load distribution with a maximum at the ground surface level and a minimum at the base of the foundation. Using the Mononobe-Okabe method, the load distribution can be considered linear. The lateral seismic soil pressure, P_s, acting against the walls at any depth, h, calculated using the following equation.

$$\mathbf{P_s = 0.5\ \gamma\ h\ (1\text{-}k_v)\ K_{ae}}$$

Where: k_v = vertical acceleration coefficient and can be set at 0

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

h = the height of the foundation wall above the founding level at the depth p_{ae} is being calculated.

Kae = 0.5 for any ground surface level or sloping away from the foundation

The total lateral seismic pressure P_{ae} acting against the foundation wall is equal to

$$\mathbf{P_{ae} = 0.5\ \gamma\ H^2\ (1\text{-}k_v)\ K_{ae}}$$

Where: H = Height of wall.

The lateral seismic soil pressure at the ground surface level for the foundation walls can be obtained from the following formula. $\mathbf{P_s = 0.5\ \gamma\ H\ (1\text{-}k_v)\ K_{ae}}$ The minimum lateral seismic soil pressure at the base is 0 kPa.

Frost Protection Requirements for Perimeter Foundation Walls and Pile Caps

The underside of all exterior foundations, all exterior pile caps and those in any unheated parts of the proposed building should be provided with at least 1.8 metres of earth cover for frost protection purposes.

The depth of frost cover could be reduced in areas where polystyrene light weight fill, such as EPS or Isofill, are used along the exterior of the foundation walls. Where there is more than 0.3 metres thickness of polystyrene lightweight fill, beneath and/or extending outwards from the foundation, there is no requirement for additional frost protection.

Below Grade and Parking Floor Slab

For predictable performance of the proposed concrete floor slab all soft or loose and any deleterious material should be removed within the proposed building area. The exposed native sub-grade surface should then be inspected and approved by geotechnical personnel.

As previously indicated, the subsurface sensitive silty clay soils at the site have limited capacity to support additional loading. It is considered that with an expected finished floor level of at least 1 meter below the existing ground surface, sufficient soil weight will have been removed during excavation to offset the loading resulting from the parking area / basement floor structure.

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 crack control cuts should be placed at a grid spacing not exceeding the lesser of 25 times the slab thickness or 4.5 metres.

SITE SERVICES

It is recommended that the site services not be placed beneath the proposed access roadways without the use of lightweight fill. It is further considered that should services be located under areas where the landscape grade raise exceeds 1 metre, lightweight fill should be used to prevent long term settlement of the subgrade below the services.

Excavation

The excavations for the site services will be carried out through topsoil and silty clay. For the purposes of Ontario Regulation 213/91 the soils at the site can be considered to be Type 3 soil. The sides of the excavations in overburden materials should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Ontario Occupational Health and Safety Act. That is, open cut excavations with overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box.

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

Since ground water was measured at 0.5 metres below the ground surface in the stand pipe within the building footprint at borehole 5 on the site, it is considered that the excavation will extend below the ground water level. It is recommended that registry within the Environmental Activity and Sector Registry water-taking sector for the purposes of construction site dewatering be completed prior to construction.

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). Allowance should be made in the contract for placing a 150 to 300 millimetre thick subbedding layer of OPSS Granular B Type II below the bedding material if soft to firm, grey silty clay material is encountered at the level of the service pipes or if unavoidable disturbance to the subgrade occurs. To minimize disturbance to the silty clay subgrade, excavating to final grade should be carried out with a bucket equipped with a flat blade.

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. The use of clear crushed stone as bedding or sub-bedding material should not be permitted.

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 roadway 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.

The backfill height, above the native ground surface, without the use of lightweight fill should be limited to a maximum of 1.0 metres when beyond 2.4 meters from the building and 0.5 meters when closer to the building. The lightweight fill should consist of EPS as outlined detailed above. A minimum of 0.5 metres of soil cover over the EPS is recommended.

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.

The soils that exist at this site are highly frost susceptible and are prone to significant ice lensing. In order to carry out the work during freezing temperatures and maintain adequate performance of the trench backfill as a roadway subgrade, the service trenches should be opened for as short a time as practicable and the excavations should be carried out only in lengths which allow all of the construction operations, including backfilling, to be fully completed in one working day. The sides of the trenches should not be allowed to freeze. In addition, the backfill should be excavated, stored and replaced without being disturbed by frost or contaminated by snow or ice.

Seepage Barriers

The permanent lowering of the groundwater level at the site can be caused by drainage through the granular bedding and cover materials within the sewer or water service trenches. Groundwater lowering can cause stress within the silty clay materials which underlie the site and in turn result in settlement of the concrete slab on grade floor. To minimize the possibility of groundwater lowering at this site due to the presence of the proposed sewers or water service, it is considered that clay dykes should be provided within sewer and water service trenches at about 50 metre spacing. Details for construction of the proposed clay dykes are shown in the attached Figure 3.

ACCESS ROADWAY AND PARKING AREA PAVEMENTS

Based on the results of the boreholes, the subsurface conditions in the access roadway and parking areas consist of topsoil overlying grey brown silty clay. For predictable performance of the pavement structures, it is considered that all of the topsoil will have to be removed in preparation for pavement construction at this site.

Once existing topsoil and all 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. Any materials proposed for this use should be approved by the geotechnical engineer before placement within the roadway. 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.

The total road and parking area structure should be limited to a maximum grade raise of 0.5 metres within 2.4 metre of the proposed building and within 1.0 metres of the services without the use of lightweight fill. Should lightweight fill be require below the road structure, the lightweight fill should consist of expanded polystyrene insulation with a minimum compressive resistance at 1% deformation of 40 kPa for light duty pavement and with a minimum compressive resistance at 1% deformation of 50 kPa for heavy duty pavement. The lightweight fill should have a maximum water absorption by total immersion of 3%. The lightweight fill should be covered with a minimum thickness of 1.0 meters.

Pavement Structure

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

50 millimetres of hot mix asphalt concrete (HL3) or Superpave 12.5 asphaltic concrete over
150 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase over
(50 or 100 millimetre minus crushed stone)
Non-woven geotextile fabric (4 oz/sy) such as Terrafix 270R or Thrace-Ling 130EX or approved alternative.

For pavement areas subject to heavy truck loading the pavement should consist of:

40 mm of hot mix asphaltic concrete (HL3) or Superpave 12.5 asphaltic concrete over
40 mm of hot mix asphaltic concrete (HL8) or Superpave 19.0 asphaltic concrete over
150 mm of OPSS Granular A base over
400 mm of OPSS Granular B, Type II subbase
(50 or 100 mm 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 materials should be carried out in maximum 300 millimetre thick loose lifts to a minimum of 100 percent SPMDD (standard proctor maximum dry density) for the Granular A base and 98 percent SPMDD for the Granular B Type II subbase 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. A thick granular pad will be required for pile driving rigs.

If the pavement structure is to be used by construction traffic or as a temporary haul road and staging area for the construction, we suggest that the following minimum pavement structure:

90 mm of Superpave 12.5 (Traffic Level A or B) asphaltic concrete, over
150 mm of OPSS Granular A base, over
450 mm of OPSS Granular B Type II subbase, over

Non-woven geotextile separator, such as Linq 150EX (6 oz/yd^2 non-woven geotextile) or an approved equivalent.

The placement of the wear course asphaltic concrete could be delayed until after the construction.

The above pavement structure assumes that the subgrade surface is prepared as described in this report. The Granular B Type II thickness given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase if the roadway subgrade surface becomes disturbed or wetted due to construction operations or precipitation. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction.

In areas where the new pavement will abut the existing pavement, the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement.

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. Where storm sewers are used to convey surface water runoff, catch basins should be provided with minimum 3 metre long perforated stub drains which extend in at least two directions from each catch basin at pavement subgrade level.

Effects of Trees

This site is underlain by deposits of sensitive silty clay, 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. Therefore, no deciduous trees should be permitted closer to the building (or any ground supported structures which may be affected by settlement) than the ultimate height of the trees. For groups of trees or trees in rows, the separation distance should be increased to 1.5 times the ultimate height of the trees.

The effects of trees should be considered in landscaping the property.

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 and building dimensions 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 sub-grade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundation should be inspected to ensure that the materials used conform to the grading and compaction specifications.

The sub-grade for the site services and access roadways 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 roadway granular materials to ensure the materials meet the specifications from a compaction point of view.

Full time field review will be required for pile foundations to check that the piling installation meets specifications. The pile type, installation procedures and refusal criteria proposed by the piling contractor should be reviewed and accepted by the geotechnical engineer prior to the start of construction. Copies of mill certificates for the piling material should also be submitted and accepted before delivery of the material to the site.

The native silty clay 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 sub-grade should be kept to an absolute minimum and the sub-grade 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
Feb 27.2018
S.E. deWit
100079612
PROVINCE OF ONTARIO

Dean Tataryn, B.E.S., EP. Steve DeWit, P.Eng.

Attachments: List of Abbreviations and Terminology
Record of Boreholes
Figures 1, 2 and 3
Table I - Order of Water Demand for Common Trees
Laboratory Test Results for Chemical Properties
Laboratory Test Results for Physical Properties – Stantec Laboratory Test Results for Soils and Moisture Contents

®

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 millimetre required to drive a 50 mm drive open . sampler for a distance of 300 mm. For split spoon samples wbere 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 remoulded shear strength

SOIL DESCRIPTIONS

Relative Density	'N' Value
Very Loose	O to4
Loose	4to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	over50

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

RECORD OF BOREHOLE BH1

PROJECT: Proposed Residential Development
CLIENT: Ironclad Developments Inc.
LOCATION: 800 Eagleson Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180084
DATE OF BORING: February 13, 2018
SHEET 1 of 2
DATUM:

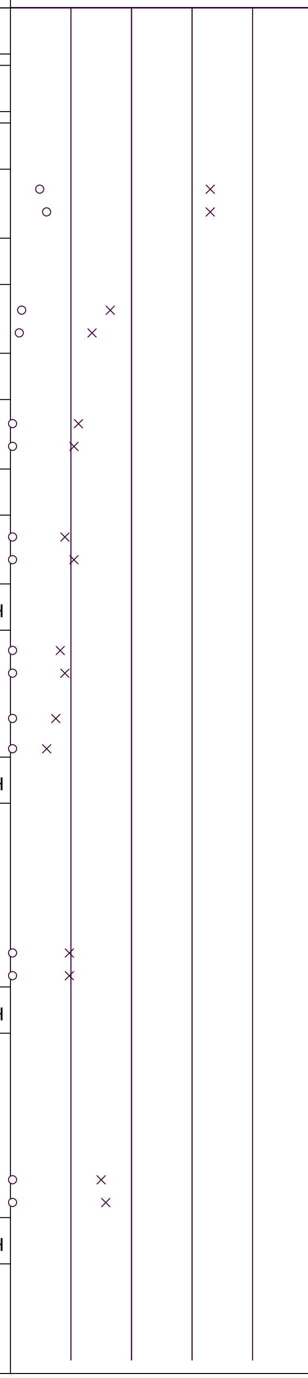
SOIL PROFILE

DESCRIPTION	ELEV. / DEPTH (M)
Ground Surface	94.30
Black, Peaty TOPSOIL	0.00
Stiff grey brown SILTY CLAY, trace sand	93.70 / 0.60
Firm to soft grey SILTY CLAY	93.24 / 1.06

SAMPLES

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING (MC%)
1	SS	4	27
2	SS	3	
3	SS	10	
4	SS	9	31
5	SS	4	40
6	SS	2	44
7	SS	WH	39
8	SS	WH	41
9	SS	WH	37
10	SS	WH	

UNDIST. SHEAR STRENGTH × Cu, kPa (20, 40, 60, 80)
REM. SHEAR STRENGTH ○ Cu, kPa (20, 40, 60, 80)



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

DEPTH SCALE (meters): 0 – 17

PIEZOMETER OR STANDPIPE INSTALLATION

Water observed in borehole at approximately 2.1 metres below the existing ground surface on February 13, 2018.

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

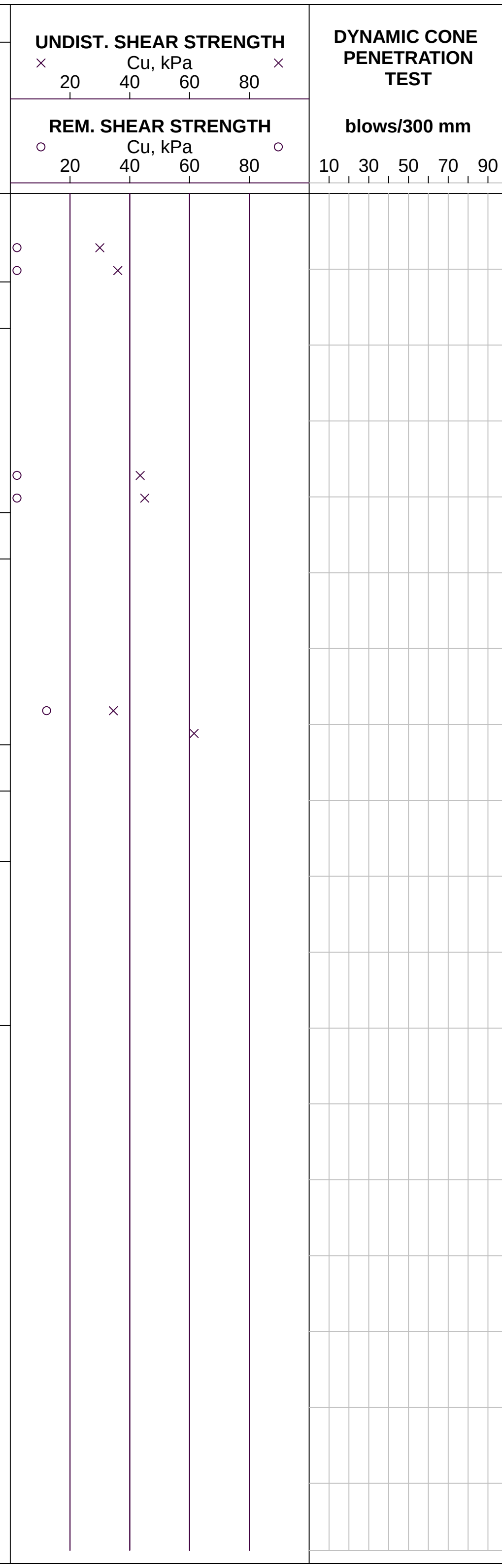
RECORD OF BOREHOLE BH1

PROJECT: Proposed Residential Development
CLIENT: Ironclad Developments Inc.
LOCATION: 800 Eagleson Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180084
DATE OF BORING: February 13, 2018

DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
19–20				11	SS	1	56	
22				12	SS	4	54	
25	Grey silty clay, trace sand and gravel, becoming bouldery with depth (GLACIAL TILL)		69.16 25.14	13	SS	7	15	
26.69	Practical refusal to augering in Glaciall Till. Advanced borehole by coring through cobbles and boulders		67.61 26.69	14	RC			
28.85	End of Borehole		65.45 28.85					



DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH2

PROJECT: Proposed Residential Development
CLIENT: Ironclad Developments Inc.
LOCATION: 800 Eagleson Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

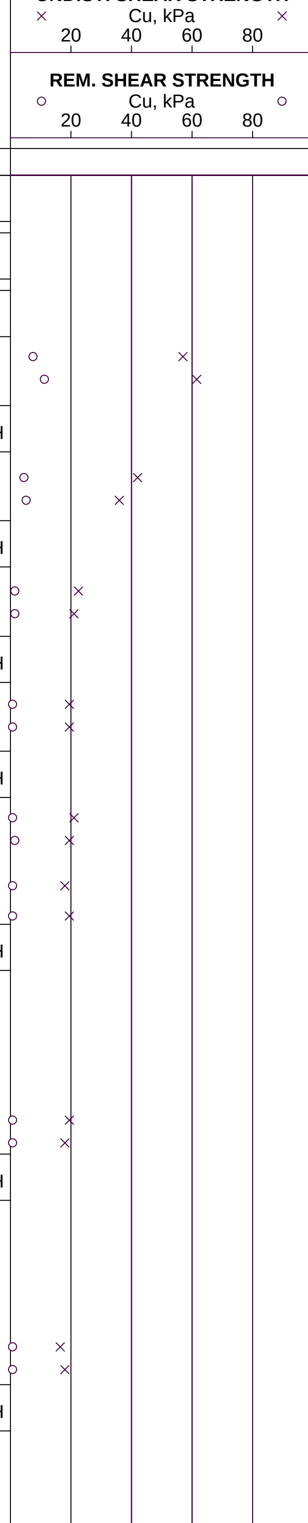
PROJECT NUMBER: 180084
DATE OF BORING: February 13, 2018
SHEET 1 of 2
DATUM:

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (M)
Ground Surface	93.90
Black, Peaty TOPSOIL	0.00
Grey brown SILTY SAND, some clay	93.30 / 0.60
Stiff grey brown SILTY CLAY, trace sand	92.79 / 1.11
Firm to soft grey SILTY CLAY	92.35 / 1.55

SAMPLES – NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING (MC%)
1	SS	6	32
2	SS	5	
3	SS	10	
4	SS	WH	47
5	SS	WH	44
6	SS	WH	44
7	SS	WH	45
8	SS	WH	
9	SS	WH	43
10	SS	WH	52

UNDIST. SHEAR STRENGTH × Cu, kPa ×

REM. SHEAR STRENGTH ○ Cu, kPa ○



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

PIEZOMETER OR STANDPIPE INSTALLATION

Water observed in borehole at approximately 1.5 metres below the existing ground surface on February 13, 2018.

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH2

PROJECT: Proposed Residential Development
CLIENT: Ironclad Developments Inc.
LOCATION: 800 Eagleson Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180084
DATE OF BORING: February 13, 2018
SHEET 2 of 2
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	ADDITIONAL LAB TESTING
18–24						
			11	SS	2	58
			12	SS	1	35
24	Grey silty clay, trace sand and gravel, becoming bouldery with depth (GLACIAL TILL)	69.82 24.08				
			13	SS	13	15
			14	SS	29	12
32	Advanced corehole through BEDROCK	61.75 32.15				
			15	RC		
34	End of Borehole	59.61 34.29				

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

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH3

PROJECT: Proposed Residential Development
CLIENT: Ironclad Developments Inc.
LOCATION: 800 Eagleson Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180084
DATE OF BORING: February 16, 2018
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa (20 40 60 80) / REM. SHEAR STRENGTH ○ Cu, kPa (20 40 60 80)	DYNAMIC CONE PENETRATION TEST blows/300 mm (10 30 50 70 90)	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		93.80						MC%	
0	Black, Peaty TOPSOIL		0.00	1	SS	2				
			92.74	2	SS	2				
1	Grey brown SILTY SAND, trace clay		1.06							
	Grey SILTY CLAY			3	SS	3			35	
2				4	SS	9			24	Borehole dry, February 16, 2018.
			90.75	5	SS	6			28	
3	End of Borehole		3.05							
4										
5										
6										
7										
8										
9										

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH4

PROJECT: Proposed Residential Development
CLIENT: Ironclad Developments Inc.
LOCATION: 800 Eagleson Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180084
DATE OF BORING: February 16, 2018
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING MC%	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		93.90							
0	Black, Peaty TOPSOIL		0.00	1	SS	2				
1	Grey brown SILTY CLAY		92.89 1.01	2	SS	4			34	
	Grey SILTY CLAY			3	SS	6			28	
2				4	SS	3			33	Water observed in borehole at about 2.1 metres below existing ground surface, February 16, 2018.
3	End of Borehole		91.01 2.89							
4										
5										
6										
7										
8										
9										

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH5

PROJECT: Proposed Residential Development
CLIENT: Ironclad Developments Inc.
LOCATION: 800 Eagleson Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180084
DATE OF BORING: February 16, 2018
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (M)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB TESTING (MC%)
	Ground Surface	94.10				
0	Black, Peaty TOPSOIL	0.00	1	SS	8	
	Grey brown SILT, trace sand	93.55 / 0.55				
1	Grey brown SILTY CLAY, trace sand		2	SS	4	28
	Grey SILTY CLAY	92.55 / 1.55	3	SS	3	28
2–3			4	SS	2	29
3			5	SS	2	39
4			6	SS	1	
	End of Borehole	89.68 / 4.42				

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

PIEZOMETER OR STANDPIPE INSTALLATION

Water measured in standpipe at approximately 0.5 metres below the existing ground surface on February 21, 2018.

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH6

PROJECT: Proposed Residential Development
CLIENT: Ironclad Developments Inc.
LOCATION: 800 Eagleson Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180084
DATE OF BORING: February 16, 2018
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING (MC%)	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		94.60							
0	Black, Peaty TOPSOIL		0.00	1	SS	15				
	Grey brown SILTY CLAY, trace sand		94.15 0.45	2	SS	6			31	
				3	SS	2			28	
2	Grey SILTY CLAY		92.59 2.01	4	SS	2			35	Water observed in borehole at about 2.1 metres below existing ground surface, February 16,
3	End of Borehole		91.71 2.89							

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH7

PROJECT: Proposed Residential Development
CLIENT: Ironclad Developments Inc.
LOCATION: 800 Eagleson Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 180084
DATE OF BORING: February 16, 2018
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		93.60						MC%	
0	Black, Peaty TOPSOIL		0.00	1	SS	5				
	Grey brown SILTY CLAY, trace sand		93.02 0.58	2	SS	5				
1										
	Grey SILTY CLAY		91.77 1.83	3	SS	5			41	Borehole dry, February 16, 2018.
2				4	SS	2			30	
3	End of Borehole		90.71 2.89							
4										
5										
6										
7										
8										
9										

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

RECORD OF BOREHOLE BH8

PROJECT: Proposed Residential Development
CLIENT: Ironclad Developments Inc.
LOCATION: 800 Eagleson Road
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

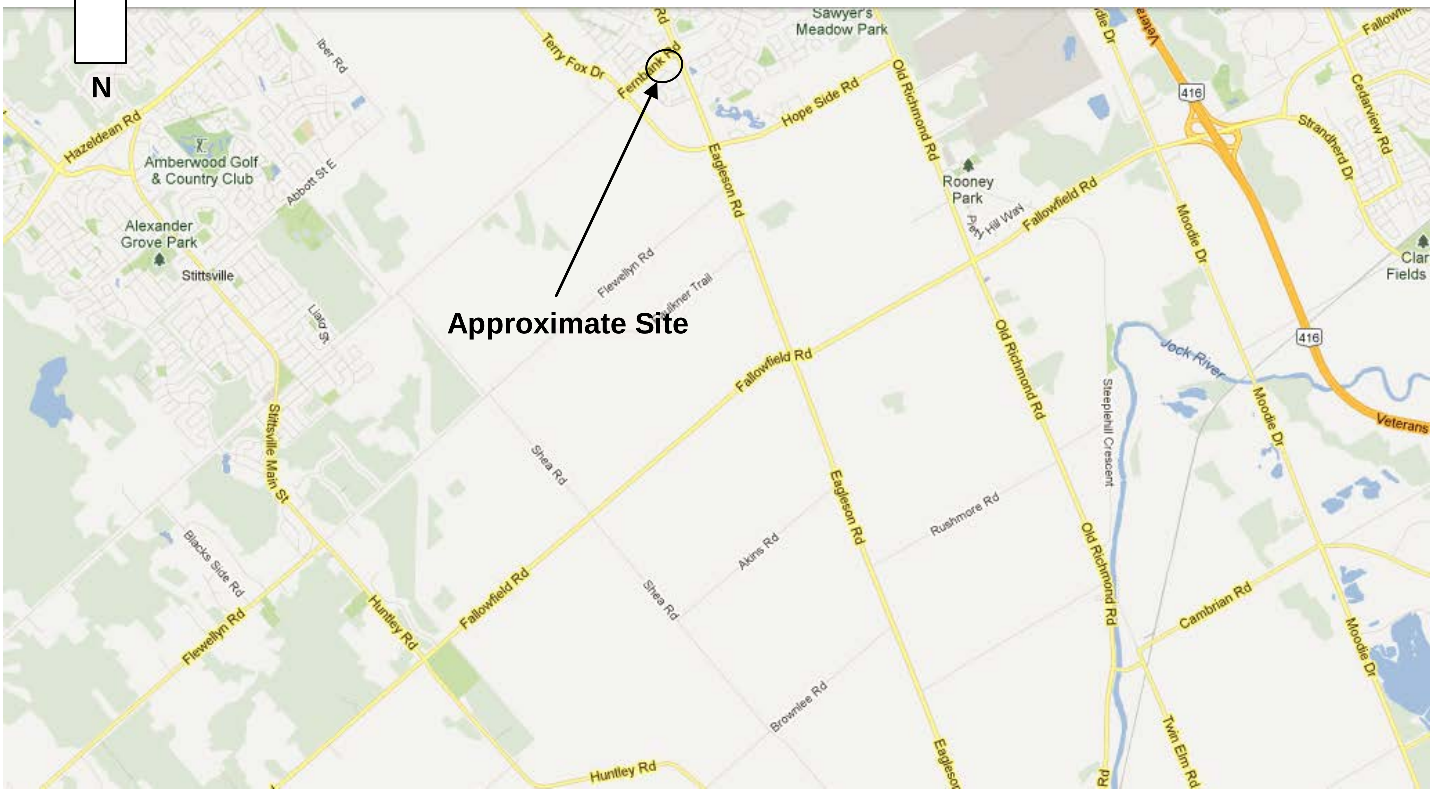
PROJECT NUMBER: 180084
DATE OF BORING: February 16, 2018
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa (20 40 60 80) / REM. SHEAR STRENGTH ○ Cu, kPa (20 40 60 80)	DYNAMIC CONE PENETRATION TEST blows/300 mm (10 30 50 70 90)	ADDITIONAL LAB TESTING (MC%)	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface		94.10							
0	Black, Peaty TOPSOIL		0.00	1	SS	4				
			93.52							
	Grey brown SILTY CLAY, trace sand		0.58							
1				2	SS	6			29	
			92.73							
	Grey SILTY CLAY		1.37							
2				3	SS	4			30	Water observed in borehole at about 1.4 metres below existing ground surface, February 16, 2018.
				4	SS	2			33	
			91.21							
3	End of Borehole		2.89							
4										
5										
6										
7										
8										
9										

DEPTH SCALE: 1 to 55
BORING METHOD: Power Auger
AUGER TYPE: 200 mm Hollow Stem
LOGGED: DT
CHECKED: SD

KEY PLAN	FIGURE 1

N



NOT TO SCALE

Kollaard Associates
Engineers

Project No. 180084

Date February 2018

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

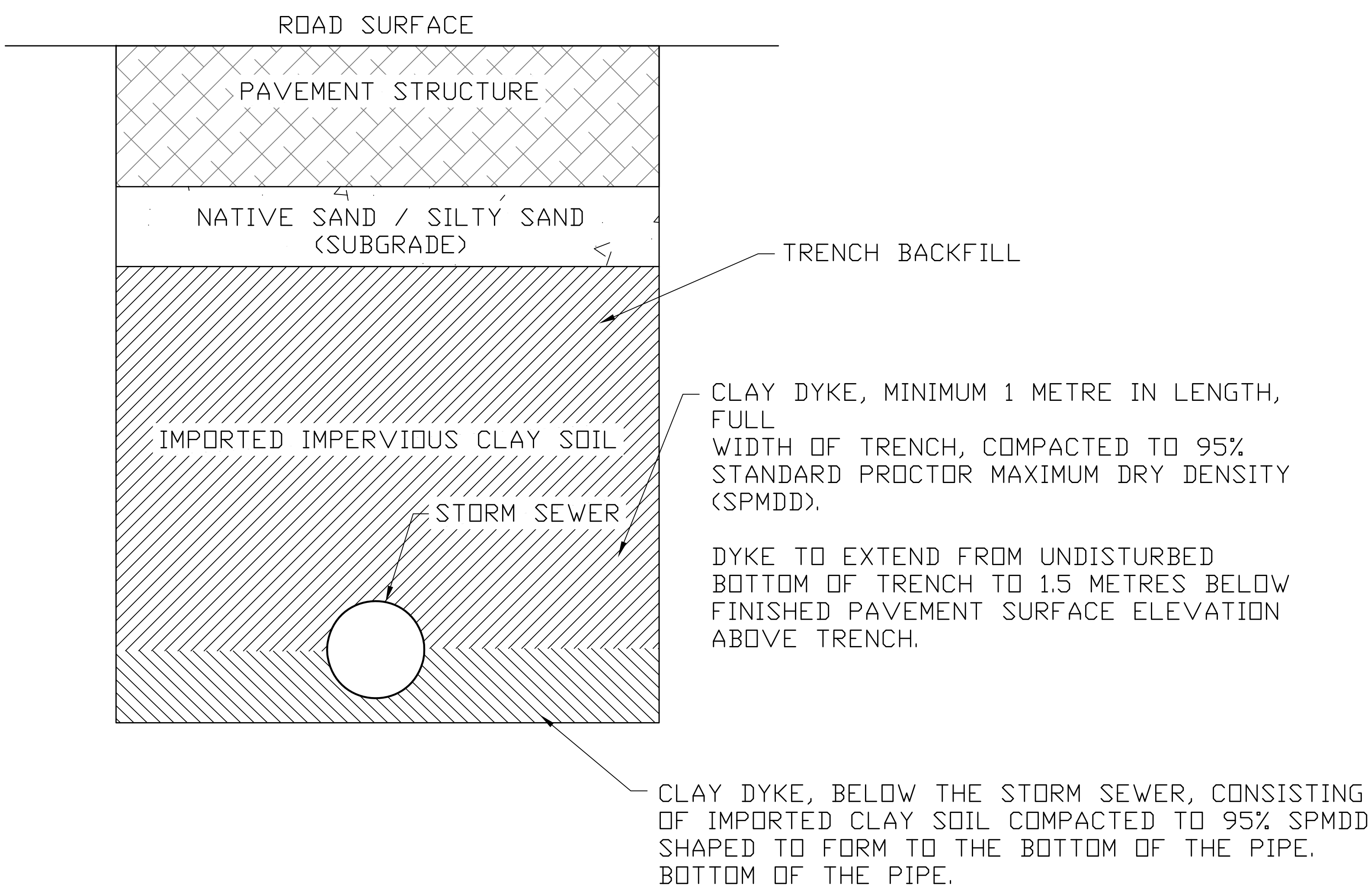
CLIENT:
IRONCLAD DEVELOPMENTS INC.

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION:
800 EAGLESON ROAD
CITY OF OTTAWA, ONTARIO

DESIGNED BY: --	*DATE:* FEBRUARY 20, 2018
DRAWN BY: DT	*SCALE:* N.T.S

KOLLAARD FILE NUMBER:
180084



The silty clay in the dyke should have a clay content over 35% percent, a liquid limit over 47 percent and a plasticity index over 20 percent.
The clay dyke may be constructed using imported clay soil that meets the above criteria.

FIGURE 3
CLAY DYKE DETAILS

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: IRONCLAD DEVELOPMENTS INC.

PROJECT: CLAY SEAL FOR STORMWATER PIPE TRENCHES

LOCATION: 800 EAGLESON ROAD, CITY OF OTTAWA, ONTARIO

DESIGNED BY: SD	DATE: FEB 26, 2018
DRAWN BY: AVB	SCALE: NTS
KOLLAARD FILE NUMBER: 180084	

TABLE I

ORDER OF WATER DEMAND FOR COMMON TREES

Some common trees in decreasing order of water demand:

Broad Leaved Deciduous

Poplar
Alder
Aspen
Willow
Elm
Maple
Birch
Ash
Beech
Oak

Deciduous Conifer

Larch

Evergreen Conifers

Spruce
Fir
Pine

Laboratory Test Results for Chemical Properties

Certificate of Analysis

Environment Testing

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0	Report Number:	1802411
		Date Submitted:	2018-02-20
		Date Reported:	2018-02-23
		Project:	180084
		COC #:	194022
Attention:	Mr. Dean Tataryn		
PO#:			
Invoice to:	Kollaard Associates Inc.		

Dear Dean Tataryn:

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

Report Comments:

APPROVAL: ______________________________

Addrine Thomas, Inorganics Supervisor

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

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

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

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

eurofins Environment Testing

Certificate of Analysis

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0	Report Number:	1802411
Attention:	Mr. Dean Tataryn	Date Submitted:	2018-02-20
PO#:		Date Reported:	2018-02-23
Invoice to:	Kollaard Associates Inc.	Project:	180084
		COC #:	194022

				Lab I.D.	1345761
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2018-02-16
				Sample I.D.	BH2 SS3 5-7
Group	**Analyte**	**MRL**	**Units**	**Guideline**	
Agri. - Soil	pH	2.00			7.90
	SO4	0.01	%		<0.01
General Chemistry	Cl	0.002	%		0.023
	Electrical Conductivity	0.05	mS/cm		0.36
	Resistivity	1	ohm-cm		2780

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

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

Certificate of Analysis

Environment Testing

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0	Report Number:	1802411
		Date Submitted:	2018-02-20
		Date Reported:	2018-02-23
		Project:	180084
Attention:	Mr. Dean Tataryn	COC #:	194022
PO#:			
Invoice to:	Kollaard Associates Inc.		

QC Summary

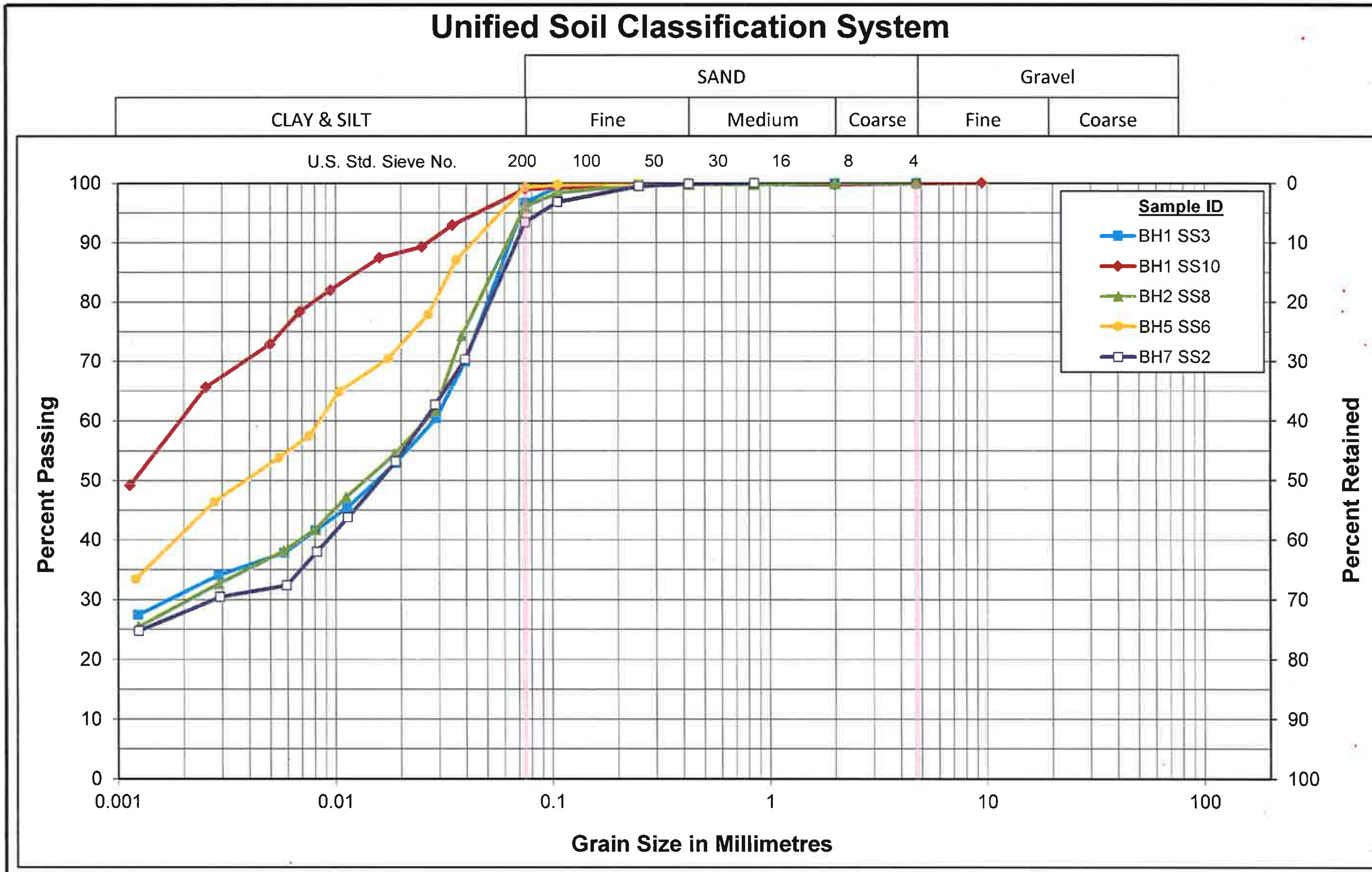
Analyte	Blank	QC % Rec	QC Limits
Run No 340984 **Analysis/Extraction Date** 2018-02-22 **Analyst** C F			
Method Ag Soil			
pH	4.50	100	90-110
SO4	<0.01 %	110	70-130
Method Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	85-115
Method Resistivity - soil			
Resistivity			
Run No 340998 **Analysis/Extraction Date** 2018-02-22 **Analyst** C F			
Method C CSA A23.2-4B			
Chloride	<0.002 %		90-110

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

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

Laboratory Test Results for Physical Properties



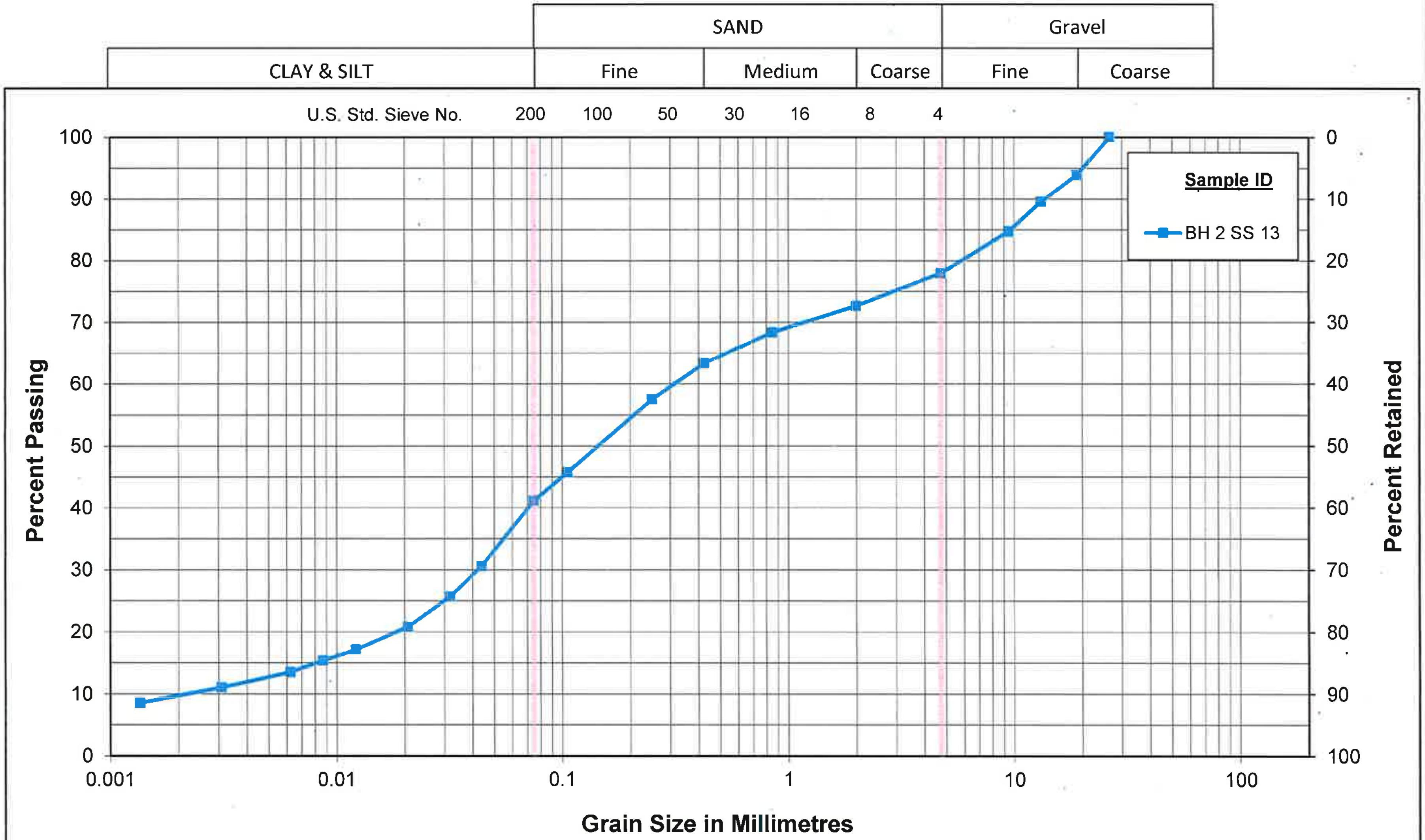
GRAIN SIZE DISTRIBUTION

Kollaard Associates Engineers, File #180084

800 Eagleson Road, Ottawa

Stantec	Figure No.
	Project No. 122410003

Unified Soil Classification System



Stantec

GRAIN SIZE DISTRIBUTION

Kollaard Associates Engineers, File #180084

800 Eagleson Road, Ottawa

Figure No.

Project No. 122410003

Stantec 2781 Lancaster Road Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASSHTO T 88

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #180084	Project No.:	122410003
Project:	800 Eagleson Road, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH1	Date Sampled:	February 20, 2018
Sample No.:	SS3	Tested By:	Denis Rodriguez
Sample Depth:	5'-7'	Date Tested:	February 21, 2018

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	51.75
Sample Weight after Hydrometer and Wash (g)	3.28
Percent Passing No. 200 Sieve (%)	93.7
Percent Passing Corrected (%)	93.59

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	258.00
Sample Weight After Sieve (g)	257.50
Percent Loss in Sieve (%)	0.19

SOIL INFORMATION

Liquid Limit (LL)	32.0	
Plasticity Index (PI)	15.5	
Soil Classification		
Specific Gravity (G_s)	2.750	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	48	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	69.09
Air Dried Mass (W_a), (g)	70.05
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9863
Air Dried Mass in Analysis (M_a), (g)	52.47
Oven Dried Mass in Analysis (M_o), (g)	51.75
Percent Passing 2.0 mm Sieve (P_{10}), (%)	99.92
Sample Represented (W), (g)	51.79

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		100.0
4.75	0.0	100.0
2.00	0.2	99.9
Total (C + F)[1]	257.50	
0.850	0.00	99.92
0.425	0.04	99.85
0.250	0.08	99.77
0.106	0.31	99.32
0.075	1.69	96.66
PAN	3.18	

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)	27.2
Meniscus Correction (H_m), (g/L)	1.0

START TIME 9:45 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
21-Feb-18	9:46 AM	1	44.0	7.0	22.5	37.0	69.90	9.39191	9.5030	0.0129	0.0395
21-Feb-18	9:47 AM	2	39.0	7.0	22.5	32.0	60.45	10.16691	9.5030	0.0129	0.0291
21-Feb-18	9:50 AM	5	35.0	7.0	22.5	28.0	52.89	10.78691	9.5030	0.0129	0.0189
21-Feb-18	10:00 AM	15	31.0	7.0	22.0	24.0	45.34	11.40691	9.6157	0.0130	0.0113
21-Feb-18	10:15 AM	30	29.0	7.0	22.0	22.0	41.56	11.71691	9.6157	0.0130	0.0081
21-Feb-18	10:45 AM	60	27.0	7.0	22.0	20.0	37.78	12.02691	9.6157	0.0130	0.0058
21-Feb-18	1:55 PM	250	25.0	7.0	21.5	18.0	34.00	12.33691	9.7308	0.0130	0.0029
22-Feb-18	9:45 AM	1440	21.5	7.0	21.5	14.5	27.39	12.87941	9.7308	0.0130	0.0012

Remarks:

Reviewed By: Brian Prevost
Date: February 26/2018

Stantec 2781 Lancaster Road
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASSHTO T 88

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #180084	Project No.:	122410003
Project:	800 Eagleson Road, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH1	Date Sampled:	February 20, 2018
Sample No.:	SS10	Tested By:	Denis Rodriguez
Sample Depth	52'6"-54'6"	Date Tested:	February 21, 2018

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	53.61
Sample Weight after Hydrometer and Wash (g)	0.46
Percent Passing No. 200 Sieve (%)	99.1
Percent Passing Corrected (%)	98.88

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	190.70
Sample Weight After Sieve (g)	190.20
Percent Loss in Sieve (%)	0.26

SOIL INFORMATION

Liquid Limit (LL)	35.8	
Plasticity Index (PI)	12.7	
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)	51.47
Air Dried Mass (W_a), (g)	52.14
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9871
Air Dried Mass in Analysis (M_a), (g)	54.31
Oven Dried Mass in Analysis (M_o), (g)	53.61
Percent Passing 2.0 mm Sieve (P_{10}), (%)	99.74
Sample Represented (W), (g)	53.75

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	0.2	99.9
2.00	0.5	99.7
Total (C + F)[1]	190.20	
0.850	0.00	99.74
0.425	0.00	99.74
0.250	0.11	99.53
0.106	0.30	99.18
0.075	0.42	98.96
PAN	0.46	

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)	27.2
Meniscus Correction (H_m), (g/L)	1.0

START TIME 9:52 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
21-Feb-18	9:53 AM	1	58.0	7.0	22.5	51.0	92.83	7.22191	9.5030	0.0129	0.0346
21-Feb-18	9:54 AM	2	56.0	7.0	22.5	49.0	89.19	7.53191	9.5030	0.0129	0.0250
21-Feb-18	9:57 AM	5	55.0	7.0	22.5	48.0	87.37	7.68691	9.5030	0.0129	0.0160
21-Feb-18	10:07 AM	15	52.0	7.0	22.5	45.0	81.91	8.15191	9.5030	0.0129	0.0095
21-Feb-18	10:22 AM	30	50.0	7.0	22.5	43.0	78.27	8.46191	9.5030	0.0129	0.0068
21-Feb-18	10:52 AM	60	47.0	7.0	22.0	40.0	72.80	8.92691	9.6157	0.0130	0.0050
21-Feb-18	2:02 PM	250	43.0	7.0	22.0	36.0	65.52	9.54691	9.6157	0.0130	0.0025
22-Feb-18	9:52 AM	1440	34.0	7.0	21.5	27.0	49.14	10.94191	9.7308	0.0130	0.0011

Remarks:

Reviewed By: Brian Prevost
Date: February/26/2018

V:\01216\active\laboratory_standing_offers\2018 Laboratory Standing Offers\122410003 Kollaard Associate Engineers\February 13-16, File # 180084, Hydr. limit and Mositures\Hydrometer Sheet_New, Calculates 20, 5 & 2 microns-May 2017.xlsx

Stantec
2781 Lancaster Road
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASSHTO T 88

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #180084	Project No.:	122410003
Project:	800 Eagleson Road, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH2	Date Sampled:	February 20, 2018
Sample No.:	SS8	Tested By:	Denis Rodriguez
Sample Depth	32'6"-34'6"	Date Tested:	February 21, 2018

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	53.95
Sample Weight after Hydrometer and Wash (g)	2.61
Percent Passing No. 200 Sieve (%)	95.2
Percent Passing Corrected (%)	95.08

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	822.10
Sample Weight After Sieve (g)	822.10
Percent Loss in Sieve (%)	0.00

SOIL INFORMATION

Liquid Limit (LL)	24.6	
Plasticity Index (PI)	8.7	
Soil Classification		
Specific Gravity (G_s)	2.750	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	48	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	69.12
Air Dried Mass (W_a), (g)	69.69
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9918
Air Dried Mass in Analysis (M_a), (g)	54.39
Oven Dried Mass in Analysis (M_o), (g)	53.95
Percent Passing 2.0 mm Sieve (P_{10}), (%)	99.91
Sample Represented (W), (g)	53.99

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		100.0
4.75	0.0	100.0
2.00	0.7	99.9
Total (C + F)[1]	822.10	
0.850	0.08	99.77
0.425	0.09	99.75
0.250	0.12	99.69
0.106	0.86	98.32
0.075	2.07	96.08
PAN	2.52	

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)	27.2
Meniscus Correction (H_m), (g/L)	1.0

START TIME 10:02 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
21-Feb-18	10:03 AM	1	48.0	7.0	22.5	41.0	74.30	8.77191	9.50295	0.012894	0.03819
21-Feb-18	10:04 AM	2	41.0	7.0	22.5	34.0	61.61	9.85691	9.50295	0.012894	0.02862
21-Feb-18	10:07 AM	5	37.0	7.0	22.5	30.0	54.36	10.47691	9.50295	0.012894	0.01866
21-Feb-18	10:17 AM	15	33.0	7.0	22.0	26.0	47.11	11.09691	9.61570	0.012970	0.01116
21-Feb-18	10:32 AM	30	30.0	7.0	22	23.0	41.68	11.56191	9.61570	0.012970	0.00805
21-Feb-18	11:02 AM	60	28.0	7.0	22.0	21.0	38.05	11.87191	9.61570	0.012970	0.00577
21-Feb-18	2:12 PM	250	25.0	7.0	21.5	18.0	32.62	12.33691	9.73081	0.013047	0.00290
22-Feb-18	10:02 AM	1440	21.0	7.0	21.5	14.0	25.37	12.95691	9.73081	0.013047	0.00124

Remarks:

Reviewed By: Brian Prevost
Date: February 26/2018

Stantec 2781 Lancaster Road
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASSHTO T 88

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #180084	Project No.:	122410003
Project:	800 Eagleson Road, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH5	Date Sampled:	February 20, 2018
Sample No.:	SS6	Tested By:	Denis Rodriguez
Sample Depth	12'6"-14'6'	Date Tested:	February 21, 2018

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	52.87
Sample Weight after Hydrometer and Wash (g)	0.40
Percent Passing No. 200 Sieve (%)	99.2
Percent Passing Corrected (%)	99.24

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	
Sample Weight After Sieve (g)	275.20
Percent Loss in Sieve (%)	

SOIL INFORMATION

Liquid Limit (LL)	31.7	
Plasticity Index (PI)	11.9	
Soil Classification		
Specific Gravity (G_s)	2.750	
Sg. Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	48	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	70.64
Air Dried Mass (W_a), (g)	71.42
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9891
Air Dried Mass in Analysis (M_a), (g)	53.45
Oven Dried Mass in Analysis (M_o), (g)	52.87
Percent Passing 2.0 mm Sieve (P_{10}), (%)	100.00
Sample Represented (W), (g)	52.87

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		100.0
4.75		100.0
2.00	0.0	100.0
Total (C + F)[1]	275.20	
0.850	0.00	100.00
0.425	0.00	100.00
0.250	0.06	99.89
0.106	0.16	99.70
0.075	0.37	99.30
PAN	0.40	

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)	27.2
Meniscus Correction (H_m), (g/L)	1.0

START TIME 10:10 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
21-Feb-18	10:11 AM	1	54.0	7.0	22.5	47.0	86.98	7.84191	9.50295	0.012894	0.03611
21-Feb-18	10:12 AM	2	49.0	7.0	22.5	42.0	77.73	8.61691	9.50295	0.012894	0.02676
21-Feb-18	10:15 AM	5	45.0	7.0	22.5	38.0	70.32	9.23691	9.50295	0.012894	0.01752
21-Feb-18	10:25 AM	15	42.0	7.0	22.5	35.0	64.77	9.70191	9.50295	0.012894	0.01037
21-Feb-18	10:40 AM	30	38.0	7.0	22.5	31.0	57.37	10.32191	9.50295	0.012894	0.00756
21-Feb-18	11:10 AM	60	36.0	7.0	22.0	29.0	53.67	10.63191	9.61570	0.012970	0.00546
21-Feb-18	2:20 PM	250	32.0	7.0	21.5	25.0	46.27	11.25191	9.73081	0.013047	0.00277
22-Feb-18	10:10 AM	1440	25.0	7.0	21.5	18.0	33.31	12.33691	9.73081	0.013047	0.00121

Remarks:

Reviewed By: Brian Prevost
Date: February 26/2018

V:\01216\active\laboratory_standing_offers\2018 Laboratory Standing Offers\122410003 Kollaard Associate Engineers\February 13-16, File # 180084, Hydr. limit and Mositures\Hydrometer Sheet_New, Calculates 20, 5 & 2 microns-May 2017.xlsx

Stantec 2781 Lancaster Road
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASSHTO T 88

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #180084	Project No.:	122410003
Project:	800 Eagleson Road, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH7	Date Sampled:	February 20, 2018
Sample No.:	SS2	Tested By:	Denis Rodriguez
Sample Depth	2'6"-4'6"	Date Tested:	February 21, 2018

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	51.54
Sample Weight after Hydrometer and Wash (g)	4.42
Percent Passing No. 200 Sieve (%)	91.4
Percent Passing Corrected (%)	91.42

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	212.50
Sample Weight After Sieve (g)	212.10
Percent Loss in Sieve (%)	0.19

SOIL INFORMATION

Liquid Limit (LL)	31.1	
Plasticity Index (PI)	14.3	
Soil Classification		
Specific Gravity (G_s)	2.750	
G_s Correction Factor (α)	0.978	
Mass of Dispersing Agent/Litre	48	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	65.69
Air Dried Mass (W_a), (g)	66.60
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9863
Air Dried Mass in Analysis (M_a), (g)	52.25
Oven Dried Mass in Analysis (M_o), (g)	51.54
Percent Passing 2.0 mm Sieve (P_{10}), (%)	100.00
Sample Represented (W), (g)	51.54

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		100.0
4.75		100.0
2.00	0.0	100.0
Total (C + F)[1]	212.10	
0.850	0.00	100.00
0.425	0.08	99.84
0.250	0.27	99.48
0.106	1.65	96.80
0.075	3.45	93.31
PAN	4.24	

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)	27.2
Meniscus Correction (H_m), (g/L)	1.0

START TIME 10:19 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
21-Feb-18	10:20 AM	1	44.0	7.0	22.5	37.0	70.24	9.39191	9.50295	0.012894	0.03951
21-Feb-18	10:21 AM	2	40.0	7.0	22.5	33.0	62.65	10.01191	9.50295	0.012894	0.02885
21-Feb-18	10:24 AM	5	35.0	7.0	22.5	28.0	53.16	10.78691	9.50295	0.012894	0.01894
21-Feb-18	10:34 AM	15	30.0	7.0	22.0	23.0	43.66	11.56191	9.61570	0.012970	0.01139
21-Feb-18	10:49 AM	30	27.0	7.0	22	20.0	37.97	12.02691	9.61570	0.012970	0.00821
21-Feb-18	11:19 AM	60	24.0	7.0	21.5	17.0	32.27	12.49191	9.73081	0.013047	0.00595
21-Feb-18	2:29 PM	250	23.0	7.0	21.5	16.0	30.37	12.64691	9.73081	0.013047	0.00293
22-Feb-18	10:19 AM	1440	20.0	7.0	21.5	13.0	24.68	13.11191	9.73081	0.013047	0.00124

Remarks:

Reviewed By: Brian Prust
Date: February 26/2018

V:\01216\active\laboratory_standing_offers\2018 Laboratory Standing Offers\122410003 Kollaard Associate Engineers\February 13-16, File # 180084, Hydr. limit and Mositures\Hydrometer Sheet_New, Calculates 20, 5 & 2 microns-I

Stantec

2781 Lancaster Road
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASSHTO T 88

PROJECT DETAILS

Client:	Kollaard Associates Engineers	Project No.:	122410003
Project:	800 Eagleson Road, Ottawa. ON. File # 180084	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH 2	Date Sampled:	February 22, 2018
Sample No.:	SS 13	Tested By:	Brian Prevost
Sample Depth:	80'-82'	Date Tested:	February 25, 2018

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	58.16
Sample Weight after Hydrometer and Wash (g)	25.42
Percent Passing No. 200 Sieve (%)	56.3
Percent Passing Corrected (%)	40.90

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	805.60
Sample Weight After Sieve (g)	803.30
Percent Loss in Sieve (%)	0.29

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)	93.04
Air Dried Mass (W_a), (g)	93.22
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9981
Air Dried Mass in Analysis (M_a), (g)	58.27
Oven Dried Mass in Analysis (M_o), (g)	58.16
Percent Passing 2.0 mm Sieve (P_{10}), (%)	72.65
Sample Represented (W), (g)	80.05

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	0.0	100.0
19.0	49.3	93.9
13.2	84.1	89.6
9.5	123.3	84.7
4.75	177.6	78.0
2.00	220.3	72.7
Total (C + F)[1]	803.30	
0.850	3.42	68.38
0.425	7.41	63.40
0.250	12.12	57.51
0.106	21.52	45.77
0.075	25.25	41.11
PAN	25.40	

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)	27.2
Meniscus Correction (H_m), (g/L)	1.0

START TIME 11:01 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
25-Feb-18	11:02 AM	1	31.0	6.0	21.5	25.0	30.56	11.40691	9.7308	0.0130	0.0441
25-Feb-18	11:03 AM	2	27.0	6.0	21.5	21.0	25.67	12.02691	9.7308	0.0130	0.0320
25-Feb-18	11:06 AM	5	23.0	6.0	21.5	17.0	20.78	12.64691	9.7308	0.0130	0.0208
25-Feb-18	11:16 AM	15	20.0	6.0	21.5	14.0	17.11	13.11191	9.7308	0.0130	0.0122
25-Feb-18	11:31 AM	30	18.5	6.0	21.5	12.5	15.28	13.34441	9.7308	0.0130	0.0087
25-Feb-18	12:01 PM	60	17.0	6.0	21.0	11.0	13.44	13.57691	9.8484	0.0131	0.0062
25-Feb-18	3:11 PM	250	15.0	6.0	21.0	9.0	11.00	13.88691	9.8484	0.0131	0.0031
26-Feb-18	8:47 AM	1320	13.0	6.0	21.0	7.0	8.56	14.19691	9.8484	0.0131	0.0014

Remarks:

Reviewed By: Brian Prevost
Date: February 26/2018

Stantec

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

February 26, 2017
File: 122410003

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

Reference: **Kollaard File #180084**
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
BH-1 SS3	5'-7'	28.0	32.0	16.5	15.5
BH-1 SS10	52'6"-54'6"	49.5	35.8	23.0	12.7
BH-2 SS8	32'6"-34'6"	36.3	24.6	15.9	8.7
BH-5 SS6	12'6"-14'6"	36.6	31.7	19.8	11.9
BH-7 SS2	2'6"-4'6"	26.4	31.1	16.8	14.3

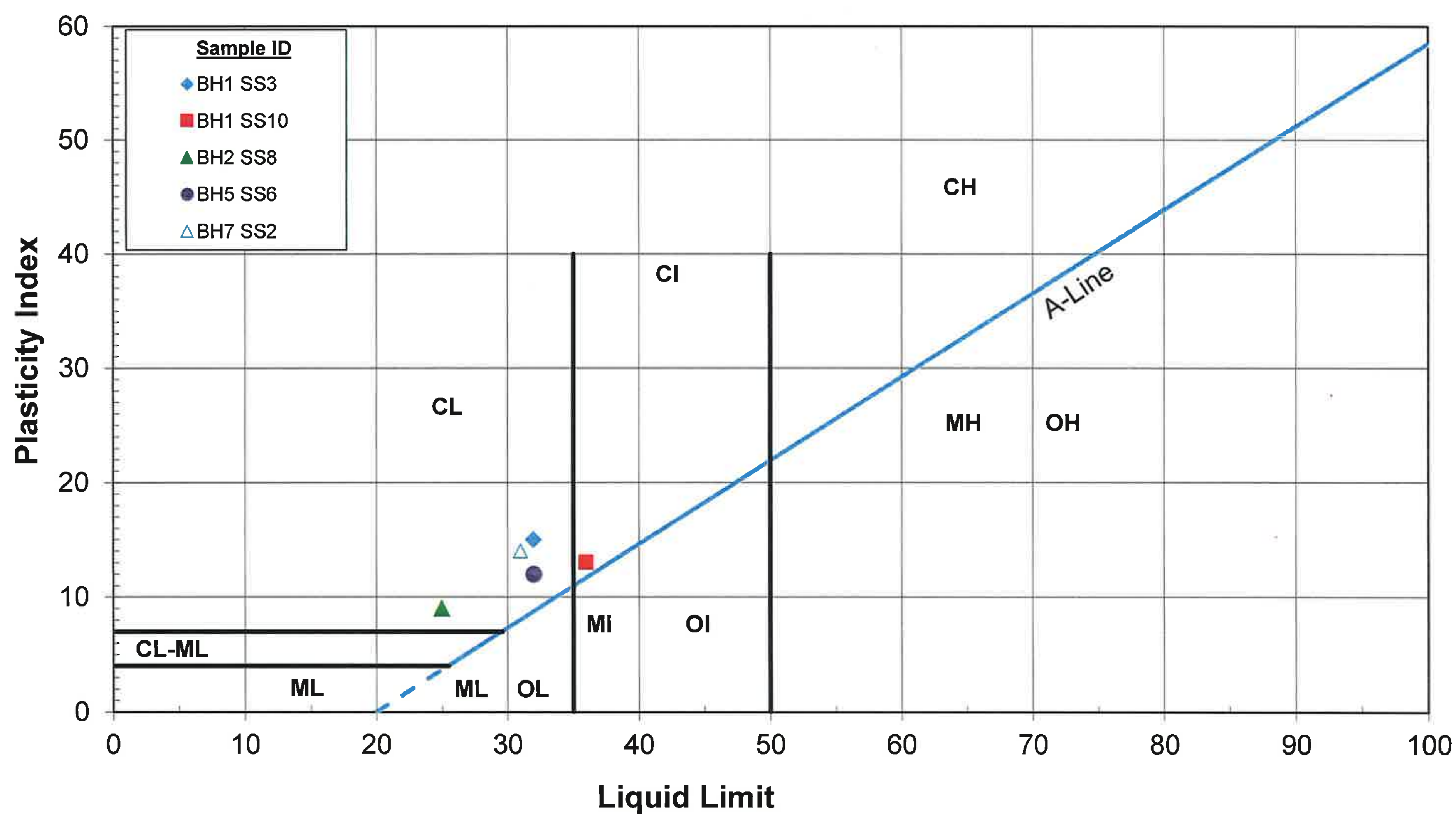
Sincerely,

Stantec Consulting Ltd

Brian Prevost

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

Attachments: Atterberg Limit Plasticity Chart



Kollaard Associates, File #180084

800 Eagleson Road, Ottawa, ON.

PLASTICITY CHART

Figure No.

Project No. 122410003

Moisture Content

LS - 701 / ASTM D 2216

PROJECT NO.:180084	DATE SAMPLED: February 13, 2018	DATE TESTED:February 20, 2018
CLIENT:Ironclad Developments	DATE RECEIVED: February 13, 2018	TESTED BY: K.L
LOCATION: 800 Eagleson Road	DATE REQUESTED:February 20, 2018	FILE NO.: 180084

METHOD A ■ METHOD B □

Water Content Recorded to +/- 1%			Water Content Recorded to +/- 0.1%		
Sieve Size, mm	Specimen Mass	Balance Readability, g	Sieve Size, mm	Specimen Mass	Balance Readability, g
75.0	5 kg	10	75.0	5 kg	10
37.5	1 kg	10	37.5	1 kg	10
19	250 g	0.1	19	250 g	0.1
9.5	50 g	0.1	9.5	50 g	0.1
4.75	20 g	0.1	4.75	20 g	0.1
2.00	20 g	0.1	2.00	20 g	0.1

ASTM D 2216 TABLE 1

Bore Hole:	1	1	1	1	1	1	1	1	1
Sample No.:	SS2	SS4	SS5	SS6	SS7	SS8	SS9	SS11	SS12
Depth:	2'6"-4'6"	10-12	15-17	20-22	25-27	32'6"-34'6"	42'6"-44'6"	62'6"-64'6"	72'6"-74'6"
Tare No.:	1	2	3	4	5	6	7	8	9
Tare +Wet Soil (gms)	44.97	66.14	63.54	47.16	71.94	66.56	59.36	61.54	74.56
Tare + Dry Soil (gms)	39.78	55.43	51.26	39.13	57.66	53.34	48.93	47.02	55.65
Mass of Water (gms)	5.19	10.71	12.28	8.03	14.28	13.22	10.43	14.52	18.91
Mass of Tare (gms)	20.89	20.88	20.89	20.87	20.87	20.89	20.98	20.91	20.89
Mass of Solids (gms)	18.89	34.55	30.37	18.26	36.79	32.45	27.95	26.11	34.76
WATER CONTENT (%)	**27**	**31**	**40**	**44**	**39**	**41**	**37**	**56**	**54**
Drying Tempterature (^{0}C), if other than 110 ±5 C									

Bore Hole:	1	2	2	2	2	2	2	2	2
Sample No.:	SS13	SS2	SS4	SS5	SS6	SS7	SS9	SS10	SS11
Depth:	82'6"-84'6"	2'6"-4'6"	10-12	15-17	20-22	25-27	42'6"-44'6"	52'6"-54'6"	62'6"-64'6"
Tare No.:	10	11	12	13	14	15	16	17	18
Tare +Wet Soil (gms)	109.48	76.25	60.08	83.4	73.79	44.9	77.9	84.87	73.75
Tare + Dry Soil (gms)	98.07	62.97	47.6	64.17	57.6	37.46	60.68	62.88	54.36
Mass of Water (gms)	11.41	13.28	12.48	19.23	16.19	7.44	17.22	21.99	19.39
Mass of Tare (gms)	20.87	20.89	20.92	20.89	20.75	20.94	20.91	20.82	20.94
Mass of Solids (gms)	77.2	42.08	26.68	43.28	36.85	16.52	39.77	42.06	33.42
WATER CONTENT (%)	**15**	**32**	**47**	**44**	**44**	**45**	**43**	**52**	**58**
Drying Tempterature (^{0}C), if other than 110 ±5 C									

Date Issued:_________________

Issued By:_________________

Moisture Content

LS - 701 / ASTM D 2216

PROJECT NO.:180084	DATE SAMPLED: February 13, 2018	DATE TESTED:February 20, 2018
CLIENT:Ironclad Developments	DATE RECEIVED: February 13, 2018	TESTED BY: K.L
LOCATION: 800 Eagleson Road	DATE REQUESTED:February 20, 2018	FILE NO.: 180084

METHOD A ■ METHOD B □

Water Content Recorded to +/- 1%			Water Content Recorded to +/- 0.1%		
Sieve Size, mm	Specimen Mass	Balance Readability, g	Sieve Size, mm	Specimen Mass	Balance Readability, g
75.0	5 kg	10	75.0	5 kg	10
37.5	1 kg	10	37.5	1 kg	10
19	250 g	0.1	19	250 g	0.1
9.5	50 g	0.1	9.5	50 g	0.1
4.75	20 g	0.1	4.75	20 g	0.1
2.00	20 g	0.1	2.00	20 g	0.1

ASTM D 2216 TABLE 1

Bore Hole:	2	2	2	3	3	3	4	4	4
Sample No.:	SS12	SS13	SS14	SS3	SS4	SS5	SS2	SS3	SS4
Depth:	72'6"-74'6"	80-82	90-92	4-6	6-8	8-10	2'6"-4'6"	5-7	7'6"-9'6"
Tare No.:	19	20	21	22	23	24	25	26	27
Tare +Wet Soil (gms)	88.53	83.93	65	65.27	93.38	72.46	58.06	81.46	55.85
Tare + Dry Soil (gms)	70.88	75.76	60.16	53.84	79.56	61.09	48.72	68.3	47.14
Mass of Water (gms)	17.65	8.17	4.84	11.43	13.82	11.37	9.34	13.16	8.71
Mass of Tare (gms)	20.89	20.94	20.93	20.9	20.95	20.98	20.86	20.86	20.96
Mass of Solids (gms)	49.99	54.82	39.23	32.94	58.61	40.11	27.86	47.44	26.18
WATER CONTENT (%)	**35**	**15**	**12**	**35**	**24**	**28**	**34**	**28**	**33**
Drying Tempterature (^{0}C), if other than 110 ±5 C									

Bore Hole:	5	5	5	5	6	6	6	7	7
Sample No.:	SS2	SS3	SS4	SS5	SS2	SS3	SS4	SS3	SS4
Depth:	2'6"-4'6"	5-7	7'6"-9'6"	10-12	2'6"-4'6"	5-7	7'6"-9'6"	5-7	7'6"-9'6"
Tare No.:	28	29	30	31	32	33	34	35	36
Tare +Wet Soil (gms)	58.91	70.7	73.08	61.97	65.74	83.21	90.44	72.4	52.67
Tare + Dry Soil (gms)	50.7	59.85	61.43	50.38	55.07	69.66	72.45	57.51	45.3
Mass of Water (gms)	8.21	10.85	11.65	11.59	10.67	13.55	17.99	14.89	7.37
Mass of Tare (gms)	20.9	20.88	20.97	20.82	20.83	20.94	20.9	20.85	20.79
Mass of Solids (gms)	29.8	38.97	40.46	29.56	34.24	48.72	51.55	36.66	24.51
WATER CONTENT (%)	**28**	**28**	**29**	**39**	**31**	**28**	**35**	**41**	**30**
Drying Tempterature (^{0}C), if other than 110 ±5 C									

Date Issued:_________________

Issued By:__________________

Moisture Content

LS - 701 / ASTM D 2216

PROJECT NO.:180084	DATE SAMPLED: February 13, 2018	DATE TESTED:February 20, 2018
CLIENT:Ironclad Developments	DATE RECEIVED: February 13, 2018	TESTED BY: K.L
LOCATION: 800 Eagleson Road	DATE REQUESTED:February 20, 2018	FILE NO.: 180084

METHOD A ■ METHOD B ☐

Water Content Recorded to +/- 1%			Water Content Recorded to +/- 0.1%		
Sieve Size, mm	Specimen Mass	Balance Readability, g	Sieve Size, mm	Specimen Mass	Balance Readability, g
75.0	5 kg	10	75.0	5 kg	10
37.5	1 kg	10	37.5	1 kg	10
19	250 g	0.1	19	250 g	0.1
9.5	50 g	0.1	9.5	50 g	0.1
4.75	20 g	0.1	4.75	20 g	0.1
2.00	20 g	0.1	2.00	20 g	0.1

ASTM D 2216 TABLE 1

Bore Hole:	8	8	8						
Sample No.:	SS2	SS3	SS4						
Depth:	2'6"-4'6"	5-7	7'6"-9'6"						
Tare No.:	37	38	39						
Tare +Wet Soil (gms)	76.4	83.48	78.27						
Tare + Dry Soil (gms)	63.85	69.13	64.06						
Mass of Water (gms)	12.55	14.35	14.21						
Mass of Tare (gms)	20.93	20.91	20.85						
Mass of Solids (gms)	42.92	48.22	43.21						
WATER CONTENT (%)	**29**	**30**	**33**						
Drying Tempterature (^{0}C), if other than 110 ±5 C									

Bore Hole:									
Sample No.:									
Depth:									
Tare No.:									
Tare +Wet Soil (gms)									
Tare + Dry Soil (gms)									
Mass of Water (gms)									
Mass of Tare (gms)									
Mass of Solids (gms)									
WATER CONTENT (%)									
Drying Tempterature (^{0}C), if other than 110 ±5 C									

Date Issued:________________

Issued By:________________

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

February 21, 2018

Site: 45.2775 N, 75.863 W   User File Reference: 800 Eagleson Road, Ottawa, ON

Requested by: ,

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.411	0.483	**0.406**	0.309	**0.220**	**0.111**	**0.053**	**0.014**	**0.0052**	**0.260**	**0.183**

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" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. 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.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.039	0.132	0.222
Sa(0.1)	0.055	0.168	0.272
Sa(0.2)	0.050	0.147	0.233
Sa(0.3)	0.040	0.115	0.180
Sa(0.5)	0.029	0.082	0.129
Sa(1.0)	0.015	0.042	0.066
Sa(2.0)	0.0058	0.020	0.031
Sa(5.0)	0.0012	0.0045	0.0077
Sa(10.0)	0.0006	0.0018	0.0031
PGA	0.030	0.092	0.149
PGV	0.020	0.063	0.103

References

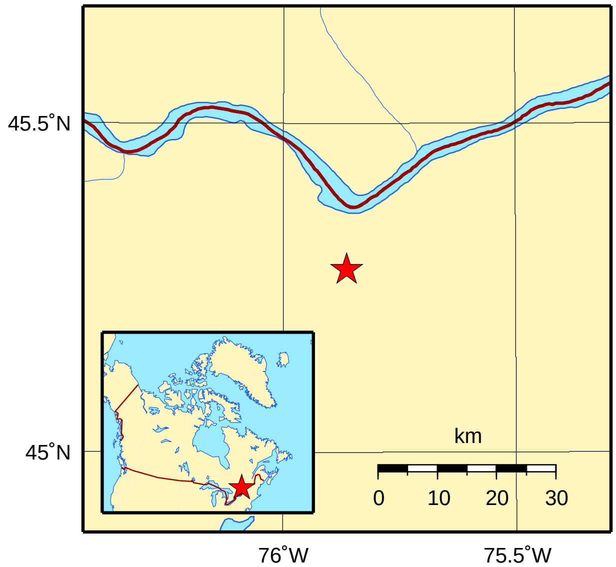
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
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

Aussi disponible en français



MOE Water Well Record for Adjacent Property

MINISTRY OF THE ENVIRONMENT
The Ontario Water Resources Act

WATER WELL RECORD

31G/5c

ONTARIO

1512480

1. PRINT ONLY IN SPACES PROVIDED
2. CHECK ☒ CORRECT BOX WHERE APPLICABLE

11 Well # 3

MUNICIP. 15008 CON. RF C 06

COUNTY OR DISTRICT	TOWNSHIP, BOROUGH, CITY, TOWN, VILLAGE	CON., BLOCK, TRACT, SURVEY, ETC.	LOT
Carleton	Nepean	V1 RF	028

OWNER (SURNAME FIRST)	ADDRESS	DATE COMPLETED
J.L. Richards & ~~Associates~~ Ass. Ltd.	864 Lady Ellen Place, Ottawa, Ont.	DAY 19 MO. 02 YR. 73

21 | ZONE 18 | EASTING 432363 | NORTHING 5013929 | RC. 4 | ELEVATION 0316 | RC. 4 | BASIN CODE 25

LOG OF OVERBURDEN AND BEDROCK MATERIALS (SEE INSTRUCTIONS)

GENERAL COLOUR	MOST COMMON MATERIAL	OTHER MATERIALS	GENERAL DESCRIPTION	DEPTH - FEET FROM	DEPTH - FEET TO
	Clay			0	80
	Sand	Gravel		80	82
	Hardpan			82	110
	Limestone			110	113

31 0080005 0082028 11 0110014 0113015

32

41 WATER RECORD

WATER FOUND AT - FEET	KIND OF WATER
~~some~~ 0080	1 ☒ FRESH 3 ☐ SULPHUR 2 ☐ SALTY 4 ☐ MINERAL
~~80~~ 82	1 ☐ FRESH 3 ☐ SULPHUR 2 ☐ SALTY 4 ☐ MINERAL
	1 ☐ FRESH 3 ☐ SULPHUR 2 ☐ SALTY 4 ☐ MINERAL
	1 ☐ FRESH 3 ☐ SULPHUR 2 ☐ SALTY 4 ☐ MINERAL
	1 ☐ FRESH 3 ☐ SULPHUR 2 ☐ SALTY 4 ☐ MINERAL

51 CASING & OPEN HOLE RECORD

INSIDE DIAM. INCHES	MATERIAL	WALL THICKNESS INCHES	DEPTH - FEET FROM	DEPTH - FEET TO
06 6¼	1 ☒ STEEL 2 ☐ GALVANIZED 3 ☐ CONCRETE 4 ☐ OPEN HOLE	.188	0	0110
	1 ☐ STEEL 2 ☐ GALVANIZED 3 ☐ CONCRETE 4 ☐ OPEN HOLE			
	1 ☐ STEEL 2 ☐ GALVANIZED 3 ☐ CONCRETE 4 ☐ OPEN HOLE			

SCREEN

SIZE(S) OF OPENING (SLOT NO.)	DIAMETER INCHES	LENGTH FEET

MATERIAL AND TYPE — DEPTH TO TOP OF SCREEN FEET

61 PLUGGING & SEALING RECORD

DEPTH SET AT - FEET FROM	TO	MATERIAL AND TYPE (CEMENT GROUT, LEAD PACKER, ETC.)

71 PUMPING TEST

PUMPING TEST METHOD: 1 ☐ PUMP 2 ☐ BAILER

PUMPING RATE: LESS 1 GPM.

DURATION OF PUMPING: HOURS MINS

STATIC LEVEL — WATER LEVEL END OF PUMPING — WATER LEVELS DURING: 1 ☐ PUMPING 2 ☐ RECOVERY — 15 MINUTES — 30 MINUTES — 45 MINUTES — 60 MINUTES (FEET)

N/A

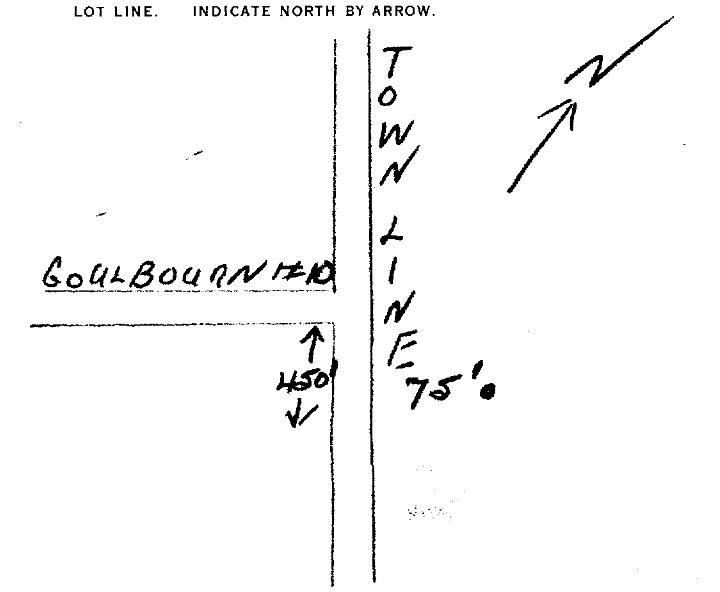
IF FLOWING, GIVE RATE GPM. — PUMP INTAKE SET AT FEET — WATER AT END OF TEST: 1 ☐ CLEAR 2 ☐ CLOUDY

RECOMMENDED PUMP TYPE: ☐ SHALLOW ☐ DEEP — RECOMMENDED PUMP SETTING FEET — RECOMMENDED PUMPING RATE GPM.

GPM./FT. SPECIFIC CAPACITY

LOCATION OF WELL

IN DIAGRAM BELOW SHOW DISTANCES OF WELL FROM ROAD AND LOT LINE. INDICATE NORTH BY ARROW.



DRILLERS REMARKS: Dry observation well

FINAL STATUS OF WELL

1 ☐ WATER SUPPLY
2 ☒ ~~OBSERVATION WELL~~
3 ☐ TEST HOLE
4 ☐ RECHARGE WELL
5 ☒ ABANDONED, INSUFFICIENT SUPPLY
6 ☐ ABANDONED, POOR QUALITY
7 ☐ UNFINISHED

WATER USE

1 ☐ DOMESTIC
2 ☐ STOCK
3 ☐ IRRIGATION
4 ☐ INDUSTRIAL
☐ OTHER
5 ☐ COMMERCIAL
6 ☐ MUNICIPAL
7 ☐ PUBLIC SUPPLY
8 ☐ COOLING OR AIR CONDITIONING
9 ☐ NOT USED

METHOD OF DRILLING

1 ☒ CABLE TOOL
2 ☒ ROTARY (CONVENTIONAL)
3 ☐ ROTARY (REVERSE)
4 ☐ ROTARY (AIR)
5 ☐ AIR PERCUSSION
6 ☐ BORING
7 ☐ DIAMOND
8 ☐ JETTING
9 ☐ DRIVING

CONTRACTOR

NAME OF WELL CONTRACTOR: McLean Water Supply Ltd — LICENCE NUMBER: 3504

ADDRESS: 1532 Raven Ave. Ottawa, Ont.

NAME OF DRILLER OR BORER: R. Lasalle — LICENCE NUMBER:

SIGNATURE OF CONTRACTOR: [signature] — SUBMISSION DATE: DAY 26 MO. 2 YR. 73

OFFICE USE ONLY

DATA SOURCE 1 — CONTRACTOR 3504 — DATE RECEIVED 240473

DATE OF INSPECTION — INSPECTOR

REMARKS: — P ✓ — WI

MINISTRY OF THE ENVIRONMENT COPY

9259

FORM 7 07-091