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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
PART OF LOT 18, CONCESSION 5
4747 BANK STREET
CITY OF OTTAWA, ONTARIO

Submitted to:

Mr. David McGann
3-15037 58th Avenue
Surrey, BC

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January 15, 2014 130708

Mr. David McGann
3-15037 58th Avenue
Surrey, BC

RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
PART OF LOT 18, CONCESSION 5
4747 BANK STREET
CITY OF OTTAWA, ONTARIO

Dear Sirs:

This report presents the results of a geotechnical investigation carried out for the above noted proposed residential development located at 4747 Bank Street, Ottawa, Ontario (see Key Plan, Figure 1). The purpose of the investigation was to identify the subsurface conditions at the site based on a limited number of test pits and 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 for the proposed residential development.

BACKGROUND SITE INFORMATION

The site has an area of approximately 7.4 hectares (18.2 acres) and is located about a kilometre south of the intersection of Bank Street and Leitrim Road in the City of Ottawa, Ontario (see Key Plan, Figure 1).

Based on initial information provided by D.G. Belfie Planning and Development Consulting Ltd, plans are being prepared to construct a residential subdivision at the site. Subsequent to the completion of the field work for the investigation, additional information was provided by the planner which indicates future mixed use/commercial development is also proposed for the portion of the property along Bank Street.

The following number of residential units are proposed:

- 68 semi-detached (34 blocks)
- 128 back to back townhouses
- 108 stacked townhouses

It is understood the proposed residential buildings will be serviced by municipal water supply and sanitary sewers. The subject site consists of woodland and former farmland. It is further understood that the proposed development will be accessed by local residential roadways. Surface drainage for the proposed development will be by means of swales, catch basins and storm sewers.

The site is bordered on the north by a residential development followed by Analdea Drive, and the northeast by a newly developed residential subdivision, on the south by vacant land and on the west by Bank Street followed by mixed commercial and residential development.

The type and number of mixed use/commercial buildings was not provided. As the types of mixed use/commercial developments and foundation requirements have not been determined at this stage, these preliminary allowable bearing pressures and factored ultimate bearing resistances are to be used for conceptual design purposes only. Kollaard Associates strongly suggests that additional subsurface investigations be carried out on a site per site basis for the final design of each of the proposed mixed use/commercial buildings along Bank Street. These preliminary allowable bearing pressures and factored ultimate bearing resistances for the mixed use/commercial are subject to change with more detailed, site specific geotechnical investigations for site specific design purposes.

Site Geology

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by shallow bedrock or possibly glacial till. Bedrock geology maps indicate that the site is underlain by grey shale, sandy shale with some dolomitic layers of the Oxford formation. Based on a review of topographical maps for the site area it is expected that the upper groundwater flow is to the southeast towards the North Castor River located approximately 600 metres south/southeast of the subject site.

PROCEDURE

The field work for this investigation was carried out in two parts with the first part consisting of putting down seventeen test pits numbered TP1 to TP17 on November 7, 2013. The second part of the field work consisted of putting down three boreholes numbered BH1 to BH3 on November 26, 2013.

Test pits

At the time of the field work, seventeen test pits were put down in various locations throughout the site. The test pits were advanced to depths ranging from about 1.6 to 3.6 metres below the existing ground surface. The subsurface conditions encountered at the test pits were classified based on visual and tactile examination of the materials exposed on the sides and bottom of the test pits. The groundwater conditions were observed in the open test pits at the time of excavating. The test pits were loosely backfilled with the excavated materials upon completion of the fieldwork.

The field work was supervised throughout by a member of our engineering staff who located the test pits in the field, logged the test pits and cared for the samples obtained. The samples obtained were delivered to the office for visual and tactile examination by the engineer to confirm field classification.

A description of the subsurface conditions encountered at the test pits given in the attached Table I, Record of Test Pits sheets following this report. The approximate locations of the test pits are shown on the attached Site Plan, Figure 2.

Boreholes

Three boreholes were put down at the site using portable drilling equipment owned and operated by OGS Drilling of Almonte, Ontario.

Sampling of the overburden materials encountered at the boreholes was carried out at on a continues basis using a 50 millimetre diameter drive open conventional split spoon sampler in conjunction with standard penetration testing to depths of about 1.37, 2.44 and 3.05 metres below the existing ground surface in BH1, BH2 and BH3, respectively. In situ vane shear testing was carried out in the cohesive materials encountered at BH3.

The subsurface soil conditions at the boreholes were identified based on visual examination of the samples recovered, the results of the in situ vane shear testing and standard penetration tests as well as laboratory test results on select samples. Groundwater conditions at the boreholes were noted at the time of drilling. The boreholes were loosely backfilled with the auger cuttings upon completion of drilling.

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 each borehole 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 locations 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 test pits and boreholes is provided in the attached Record of Test Pits and Record of Borehole Sheets following the text of this report. Here after the test pits and boreholes will collectively be referred to as test holes. The test hole logs indicate the subsurface conditions at the specific test hole locations only. Boundaries between test hole locations are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at locations other than the test hole locations may vary from the conditions encountered at the test holes.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and Kollaard Associates Inc. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

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

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

Fill

Fill materials were encountered from the surface at TP4. The fill layer was approximately 2.9 metres in thickness. The fill material consisted of topsoil, grey brown silty clay, gravel, cobbles, wood and brick.

Topsoil

Topsoil was encountered from the surface at all of the test holes. The topsoil layer was approximately 0.15 to 0.3 metres in thickness. The material was classified as topsoil based on the colour and the presence of organic materials. The identification of the topsoil layer is for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustainable plant growth.

Silty Sand/Sand and Gravel

A deposit of yellow brown silty sand with a trace to some clay and/or red brown sand and gravel with some cobbles and boulders becoming grey silty sand was encountered below the topsoil at TP5, TP9 and BH3. The silty sand and/or sand and gravel layers range in thickness from about 2.3 to 2.9 metres extending to depths of about 0.15 to 3.0 metres below the existing ground surface. The sand layer was fully penetrated at the test hole locations. The results of the standard penetration tests carried out in the silty sand at BH3 gave an N value of about 10 blows per 0.3 metres of penetration, indicating a loose to compact state of compaction.

Two soil samples (TP5 and TP16) were submitted to Stantec for grain size distribution testing. A hydrometer test was completed on the fine portion of the samples. The results of the testing are provided in the Laboratory Testing Results section at the end of this report. The hydrometer testing of two soil samples (TP5 and TP16) indicate the samples have a silt content of 57 and 20 percent and a clay content of about 25 and 6 percent, respectively. The results are located in Attachment A.

Silty Clay

A deposit of grey brown to grey silty clay was encountered below the topsoil at TP6, TP8, TP10, TP11, TP12 and TP13 and below the silty sand at BH3. A trace to some silt, gravel and cobbles was encountered within the silty clay deposit at TP11 and TP12. In situ vane shear tests carried out in the silty clay deposit gave undrained shear strength values ranging from 50 kilopascals to greater than 120 kilopascals. The results of the in situ vane shear testing and tactile examination carried out for the silty clay material indicate that the silty clay is stiff to very stiff in consistency. The silty clay layer was fully penetrated at the test holes at about 1.1 to 3.0 metres below the existing ground surface.

The results of Atterberg Limits tests conducted on a soil sample of silty clay are presented in Table I and in Attachment A at the end of the report. The tested silty clay sample classifies as inorganic clays of low to medium plasticity (CL) in accordance with the Unified Soil Classification System.

Table I – Atterberg Limit and Water Content Results

Sample	Depth(metres)	LL (%)	PL (%)	PI (%)	W (%)
BH3-SS3	1.2 – 1.8	26.8	17.5	9.3	25.5

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

CL: Clay of Low to Medium Plasticity

Glacial Till

Glacial till was encountered beneath the topsoil, fill and silty clay at TP1, TP2, TP3, TP7, TP11, TP14, TP15, TP16, TP17, BH1 and BH2. The glacial till consisted of gravel, cobbles and boulders in a matrix of red brown to grey brown to grey silty sand, with a trace to some clay. TP11, TP16 and BH2 were terminated in the glacial till on refusal to advance the test hole at a depth of approximately 3.6, 3.6 and 2.4 metres, respectively, below the existing ground surface level. Based on the standard penetration test results of 16 to 60 blows per 0.3 metres, the glacial till is indicated to be in a compact to very dense state of packing.

Two samples of the glacial till were submitted to Stantec for grain size distribution testing. The soil sample submitted consisted of a soil sample from TP1 (depth 0.5 to 1.0 metres) and TP8 (depth 2.5 metres). A hydrometer test was completed on the fine portion of the samples. The results of the testing are provided in the Laboratory Testing Results section at the end of this report. The sieve analysis for TP1 and TP8 indicated that the glacial till consists of 27 to 48 percent sand and 50 to 80 percent gravel. The results of the hydrometer test for TP1 and TP8 indicate the fines consist of about 6 and 5 percent clay and 19 to 28 percent silt and 4 to 5 percent clay. The results of the laboratory testing are presented in Attachment A.

Bedrock

All of the test holes, with the exception of TP11, TP13, TP16 and BH2 were advanced to the surface of a large boulder or weathered bedrock at depths of approximately 0.9 to 3.6 metres below existing ground surface. Where possible, the test holes were advanced through the weathered bedrock to practical refusal of advancement at depths of approximately 1.6 to 3.6 metres below the existing ground surface level. BH2 was terminated at a depth of about 2.4 metres below existing ground surface due to borehole cave in. Where bedrock was observed, a visual assessment of the bedrock indicated that the bedrock is grey/black shale.

Groundwater

A trace to some water seepage was observed in the test pits at the time of excavating at about 1.3 to 3.0 metres below the existing ground surface at TP4, TP7, TP8, TP9, TP10, TP11, TP12, TP13, TP14, TP15, TP16 and TP17. On November 26, 2013, groundwater was measured in standpipes installed in TP4, TP6 and TP11 at depths ranging between 0.1 to 3.0 metres below existing ground surface. It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

Corrosivity on Reinforcement and Sulphate Attack on Portland Cement

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

Item	Threshold of Concern	Test Result	Comment
Chlorides (Cl)	Cl > 0.04 %	<0.002	Negligible concern
pH	5.0 < pH	7.5	Neutral / Slightly Basic Negligible concern
Resistivity	R < 1500 ohm-cm	5000	Negligible 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 walls, other than ensuring minimum embedment depths are maintained.

PROPOSED RESIDENTIAL DEVELOPMENT

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.

Foundations for Proposed Residential Buildings

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

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 dwelling foundation.

For predictable performance of the proposed foundation, all fill and topsoil materials should be removed from the proposed footing areas. It is expected that the subgrade, beneath the fill and topsoil, consists of native undisturbed silty sand, silty clay, sand and gravel, glacial till or bedrock.

Conventional Concrete Spread Footing Foundation

Strip footings, a minimum 0.5 metres in width bearing on the native undisturbed silty sand, sand and gravel, silty clay, glacial till or engineered fill at a founding depth of up to 1.5 metres below the original ground surface (surface of topsoil layer) and above the groundwater level may be designed using a maximum allowable bearing pressure of 120 kilopascals for serviceability limit states and 300 kilopascals for the factored ultimate bearing resistance.

Pad footings, a minimum 0.5 metres in width bearing on the native undisturbed silty sand, sand and gravel, silty clay, glacial till or engineered fill at a founding depth of up to 1.5 metres below the original ground surface (surface of topsoil layer) and above the groundwater level may be designed using a maximum allowable bearing pressure of 150 kilopascals for serviceability limit states and 300 kilopascals for the factored ultimate bearing resistance.

The above allowable bearing pressures are subject to a maximum grade raise of 2.0 metres above the original ground surface and to maximum strip and pad footing widths of 1.5 metres.

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

For the proposed residential development with full below grade basements, a maximum allowable bearing pressure of 300 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 600 kilopascals using ultimate limit states design, may be used for the design of conventional strip footings, a minimum of 0.5 metres in width, or pad footings founded on bedrock or on a suitably constructed engineered pad founded on the bedrock.

There are no grade raise restrictions when dwellings are founded on bedrock or engineered fill placed on bedrock. Total and differential settlement of the footings designed and founded based on the above guidelines should be less than 25 millimetres and 20 millimetres, respectively. If the foundation is bearing on both bedrock and engineered fill, the foundation should be reinforced at the transition and the reinforcement should extend a minimum of 3 metres on both sides of the transition point.

Where fill material and/or topsoil is encountered below the proposed founding level, the fill and/or topsoil material should be removed and replaced with compacted granular material (engineered fill). The engineered fill should consist of granular material meeting Ontario Provincial Standards Specifications (OPSS) requirements for Granular A or Granular B Type II and should be compacted in maximum 400 millimetre thick loose lifts to at least 98 percent of the standard Proctor maximum dry density. To allow the spread of load beneath the footings, the engineered fill should extend down and out from the edges of the footing at 1 horizontal to 1 vertical, or flatter. It is considered that the engineered fill should be compacted using dynamic compaction with a large diameter vibratory steel drum roller or diesel plate compactor. If a diesel plate compactor is used, the lift thickness may need to be restricted to less than 300 mm to achieve proper compaction. Compaction should be verified by a suitable field compaction test method.

The excavations for the proposed basements should be sized to accommodate this fill placement. Currently, OPSS documents allow recycled asphaltic concrete to be used in Granular A and

Granular B Type II materials. If the source of recycled material cannot be verified, it is suggested that any granular materials used below the founding level be composed of virgin materials only.

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

Frost Protection

All exterior foundation elements and those in any unheated parts of the proposed residential buildings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated foundation elements adjacent to surfaces, which are cleared of snow cover during winter months should be provided with a minimum 1.8 metres of earth cover. Where less than the required depth of soil cover can be provided, the foundation elements should be protected from frost by using a combination of earth cover and extruded polystyrene rigid insulation. A typical frost protection insulation detail could be provided, if required.

Foundation Wall Backfill and Drainage

A conventional, perforated perimeter drain should be provided at founding level, leading by gravity flow to a sump or storm sewer. The drain should be installed at footing level and provided with a 150 millimetre thick surround of 20 millimetre minus crushed stone. The drain should be provided with a backflow preventer.

It is considered that in view of the groundwater conditions observed at the test pits and boreholes, for foundations founded no lower than 1.5 metres below the existing ground surface, the above perimeter drainage system should adequately handle any groundwater seepage to the basements.

The soils encountered at this site are considered to be frost susceptible. As such, to prevent possible foundation frost jacking, the backfill against any unheated or insulated walls or isolated walls or piers should consist of free draining, non-frost susceptible material. If imported material is required, it should consist of sand or sand and gravel meeting OPSS Granular B Type I grading requirements. Alternatively, foundations could be backfilled on the exterior 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. Where the granular backfill will ultimately support a pavement structure or walkway, it is suggested that the wall backfill material be compacted in 250 millimetre thick lifts to 95 percent of the standard Proctor dry density value.

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

Seismic Design for the Proposed Residential Buildings

For seismic design purposes, in accordance with the 2006 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class C. The subsurface conditions below the proposed footing design level consist of silty sand, sand and gravel, glacial till or bedrock. The soils have an average normalized standard penetration resistance of 40 before refusal to further penetration was encountered on underlying boulders in glacial till or bedrock at depths ranging between 1.4 to 3.1 metres. As indicated above, sound bedrock is underlying the site at a depth ranging from about 1.6 to 3.6 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)	d_i/N_i (blows/0.3m)
1	Glacial Till	0.1	0.97	14	0.029
2	Weathered Bedrock	1.07	0.3	56	0.005
3	Bedrock	1.37	29.23	100 (1)	0.292
$sum(d_i/N(60)_i)$					0.321
$d_c/(sum(d_i/N(60)_i)$					93.5

Borehole 2					
Layer	Description	Depth (m)	d_i (m)	$N(60)_i$ (blows/0.3m)	d_i/N_i (blows/0.3m)

1	Glacial Till	0.2	3.8	37	0.103
2	Bedrock	4.0 (2)	26.7	100 (1)	0.267
$sum(d_i/N(60)_i)$					0.370
$d_c/(sum(d_i/N(60)_i)$					82.3

Borehole 3					
Layer	Description	Depth (m)	d_i (m)	$N(60)_i$ (blows/0.3m)	d_i/N_i (blows/0.3m)
1	Silty Sand	0.1	0.2	15	0.027
2	Silty Clay	0.4	2.6	62	0.042
3	Glacial Till	3.0	0.1	86	0.001
4	Bedrock	3.1	27.4	100 (1)	0.274
$sum(d_i/N(60)_i)$					0.344
$d_c/(sum(d_i/N(60)_i)$					88.7

1) The blow counts N(60) in bedrock are set at the maximum value of 100.
2) Bedrock depth estimated from adjacent test pit results.

Since $50 < d_c/(sum(d_i/N(60)_i)= 82$ to 93, the seismic site response is Site Class C.

It is noted that a higher seismic classification could potentially be obtained with site specific shear vane testing.

Potential for Soil Liquefaction

Consideration for the potential for soil liquefaction was determined by considering the ratio between the cyclic resistance ratio (CRR) to the cyclic stress ratio (CSR) for the soils between the proposed underside of footing level and the depth at which refusal to further advancement using standard penetration testing was attained. The CRR value was determined from a mathematical expression as determined by Rauch (1997) of the base curve obtained from Robertson and Fear (1996). The CSR was determined from Seed and Idriss (1971). It is considered that a soil with a normalized SPT of greater than 30 is non-liquefiable. It is also considered that a soil with a CRR/CSR ratio of greater than one is not liquefiable. The average CRR / CSR ratio for the materials encountered between the silty clay and depth explored excluding the normalized SPT values above 30 is 2.4. As such the underlying soils below the proposed foundation are not considered to be liquefiable.

SITE SERVICES

Excavation

The excavations for the site services will be carried out through the fill, topsoil, silty sand, sand and gravel and glacial till. 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.

In accordance with O.Reg 213/91, s. 226, the upper soils at this site can be considered to be Type 3 soil. As such, open cut excavations within the upper soil deposits at this site above the ground water level should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter to within 1.2 metres of the bottom of the excavation. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box.

It is expected that some of the service excavations will extend below the water table in the silty clay and silty sand soils. Where this occurs, some loss of ground and groundwater inflow may occur, requiring side slopes as flat as 3 horizontal to 1 vertical or a tightly fitting, braced steel trench box to be used.

The excavations within the silty sand, sand and gravel and glacial till above the groundwater level should not present any serious constraints. In contrast, excavations below the groundwater level within the silty sand, sand and gravel and glacial till deposits encountered at all of the test holes 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.

It is considered that, depending on the climate conditions preceding and during the installation of the services pumping in excess of fifty thousand liter per day may be required. As such it is considered that a permit to take water be obtained.

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 subexcavation of any existing fill or disturbed material encountered at subgrade level. Granular material meeting OPSS specifications for Granular B Type II could be used as a sub-bedding material. The use of clear crushed stone as bedding or sub-bedding material should not be permitted.

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

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

Trench Backfill

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

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

Where native backfill is used, it should match the native materials exposed on the trench walls. Some of the native materials from the lower part of the trench excavations may be wet of optimum for compaction. Depending on the weather conditions encountered during construction, some drying of materials and/or recompaction may be required. Any wet materials that cannot be compacted to the required density should either be wasted from the site or should be used outside of existing or future 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.

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 87 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.

ROADWAY PAVEMENTS

In preparation for pavement construction at this site, all surficial topsoil and any soft, wet or deleterious materials should be removed from the proposed roadways. The exposed subgrade should be inspected and approved by geotechnical personnel and any soft areas evident should be subexcavated and replaced with suitable earth borrow approved by the geotechnical engineer. The subgrade should be shaped and crowned to promote drainage of the roadway granulars. Following approval of the preparation of the subgrade, the pavement granulars may be placed.

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

For local residential roadways the pavement should consist of:

40 mm of HL3 (or Superpave 12.5) hot mix asphaltic concrete over
40 mm of HL8 (or Superpave 19.0) hot mix asphaltic concrete over
150 millimetres of OPSS Granular A base over
400 millimetres of OPSS Granular B, Type II (100 mm minus crushed stone) subbase

Performance grade PG 58-34 asphaltic concrete should be specified. Compaction of the granular pavement materials should be carried out in maximum 300 millimetre thick loose lifts to at least 98 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment.

The above pavement structures will be adequate on an acceptable subgrade, that is, one where any roadway fill has been adequately compacted. If the roadway subgrade is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or incorporate a non-woven geotextile separator between the roadway subgrade surface and the granular subbase material. The adequacy of the design of the pavement thickness should be assessed by the geotechnical personnel at the time of construction

CONSTRUCTION CONSIDERATIONS

It is suggested that the final design drawings for the project, including the proposed site grading plan, be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

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 footing areas and any engineered fill areas for the proposed residential buildings should be inspected by Kollaard Associates Inc. to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundations should be inspected to ensure that the materials used conform to the grading and compaction specifications.

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

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

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

Regards,
Kollaard Associates Inc.

LICENSED PROFESSIONAL ENGINEER
March 11, 2014
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Attachments: Record of Boreholes
Table I - Record of Test Pits
Figures 1 and 2
Attachment A - Results of Chemical Laboratory Testing
Attachment B – Stantec Laboratory Test Results for Soils

RECORD OF BOREHOLE BH1

PROJECT: Proposed Residential Development
CLIENT: Mr. David McGann
LOCATION: 4747 Bank Street, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 130708
DATE OF BORING: November 26, 2013
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									
0	TOPSOIL		0.00							Borehole dry on November 26, 2013.
	Red brown sandy silt, trace to some gravel, cobbles (GLACIAL TILL)			1	SS	14				
1				2	SS	57				
	Weathered BEDROCK		1.07	3	SS	60				
	End of Borehole, refusal in Weathered BEDROCK		1.37							
2										
3										
4										
5										
6										
7										
8										

Kollaard Associates Engineers

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

RECORD OF BOREHOLE BH2

PROJECT: Proposed Residential Development
CLIENT: Mr. David McGann
LOCATION: 4747 Bank Street, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 130708
DATE OF BORING: November 26, 2013
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									
0	TOPSOIL		0.00							
	Red brown to grey silty sand, trace to some gravel, cobbles and boulders, trace clay (GLACIAL TILL)		0.25	1	SS	16				Borehole dry on November 26, 2013.
1				2	SS	22				
				3	SS	51				
2				4	SS	56				
	End of Borehole in GLACIAL TILL due to cave in		2.44							
3										
4										
5										
6										
7										
8										

Kollaard Associates
Engineers

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

RECORD OF BOREHOLE BH3

PROJECT: Proposed Residential Development
CLIENT: Mr. David McGann
LOCATION: 4747 Bank Street, Ottawa, Ontario
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 130708
DATE OF BORING: November 26, 2013
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									
0	TOPSOIL		0.00	1	SS	10				
	Yellow brown SILTY SAND, trace to some clay									
	Stiff to Very stiff grey brown to grey SILTY CLAY		0.40							Borehole dry on November 26, 2013.
1										
				2	SS	70				
2				3	SS	31				
				4	SS	45				
3	Grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)		3.05	5	SS	50				
	End of borehole, refusal on large boulder or bedrock									
4										
5										
6										
7										
8										

Kollaard Associates Engineers

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

TABLE I

RECORD OF TEST PITS
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL SUBDIVISION
4747 BANK STREET
CITY OF OTTAWA, ONTARIO

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP1	0.00 – 0.20	TOPSOIL
	0.20 – 1.15	Red brown sandy silt, some gravel cobbles and boulders, trace clay (GLACIAL TILL)
	1.15 – 1.75	Weathered BEDROCK
	1.75	Refusal on grey/black BEDROCK

Test pit dry, November 7, 2013.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP2	0.00 – 0.15	TOPSOIL
	0.15 – 1.00	Red brown sandy silt, some gravel cobbles and boulders, trace clay (GLACIAL TILL)
	1.00 – 2.20	Weathered BEDROCK
	2.20	Refusal on grey/black BEDROCK

Test pit dry, November 7, 2013.

TABLE I (continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP3	0.00 – 0.20	TOPSOIL
	0.20 – 0.90	Red brown sandy silt, some gravel, cobbles and boulders (GLACIAL TILL)
	0.90 – 1.70	Weathered BEDROCK
	1.70	Refusal on grey/black BEDROCK

Test pit dry, November 7, 2013.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP4	0.00 – 2.90	Grey brown silty clay, gravel, cobbles, topsoil, brick, wood debris (FILL)
	2.90 – 3.60	Grey brown silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)
	3.60	Refusal, BEDROCK

Water observed in test pit at about 2.9 metres below existing ground surface, November 7, 2013.
Water measured in standpipe at about 2.8 metres below existing ground surface, November 26, 2013.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP5	0.00 – 0.25	TOPSOIL
	0.25 – 1.30	Red brown SAND and GRAVEL, some cobbles and boulders with depth
	1.30 – 2.50	Very dense grey SILTY SAND
	2.50	Refusal on large boulder or weathered BEDROCK

Test pit dry, November 7, 2013.

TABLE I (continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP6	0.00 – 0.25	TOPSOIL
	0.25 – 1.10	Stiff grey brown SILTY CLAY
	1.10 – 1.60	Weathered BEDROCK
	1.60	Refusal on grey/black BEDROCK

Test pit dry, November 7, 2013.
Water measured in standpipe at 0.1 metres below the existing ground surface, November 26, 2013.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP7	0.00 – 0.20	TOPSOIL
	0.20 – 1.90	Red brown to grey sandy silt, some gravel, cobbles and boulders, trace clay with depth (GLACIAL TILL)
	1.90	Refusal on large boulder or weathered BEDROCK

Some water at about 1.6 metres below existing ground surface, November 7, 2013.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP8	0.00 – 0.30	TOPSOIL
	0.30 – 0.75	Stiff grey brown SILTY CLAY
	0.75 – 3.20	Yellow brown to grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)
	3.20	Refusal on grey/black BEDROCK

Some water at about 1.4 metres below existing ground surface, November 7, 2013.

TABLE I (continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP9	0.00 – 0.15	TOPSOIL
	0.15 – 1.30	Red brown SAND AND GRAVEL, some gravel, cobbles and boulders, with depth
	1.30 – 3.00	Red brown to grey SILTY SAND
	3.00	Refusal on weathered BEDROCK

Some water at about 1.3 metres below existing ground surface, November 7, 2013.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP10	0.00 – 0.20	TOPSOIL
	0.20 – 1.80	Stiff grey brown SILTY CLAY
	1.80 – 2.20	Weathered BEDROCK
	2.20	Refusal on grey/black BEDROCK

Some water at about 2.0 metres below existing ground surface, November 7, 2013.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP11	0.00 – 0.15	TOPSOIL
	0.15 – 2.90	Stiff grey brown SILTY CLAY, trace to some sand, gravel, cobbles and boulders with depth
	2.90 – 3.60	Grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)
	3.60	End of test pit

In Situ Undrained Shear Strength Test Results

Depth (metres)	Cu(kilopascal)
0.70	70
2.30	100, 80

Some water at about 3.0 metres below existing ground surface, November 7, 2013.
Water measured in standpipe at 1.3 metres below the existing ground surface, November 26, 2013.

TABLE I (continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP12	0.00 – 0.15	TOPSOIL
	0.15 – 3.00	Stiff grey brown SILTY CLAY, trace to some sand, gravel, cobbles and boulders with depth
	3.00 – 3.50	Grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)
	3.50	End of test pit on surface of weathered BEDROCK

In Situ Undrained Shear Strength Test Results

Depth (metres)	Cu(kilopascal)
0.80	>120
1.70	>120
2.80	50

Some water at about 3.0 metres below existing ground surface, November 7, 2013.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP13	0.00 – 0.25	TOPSOIL
	0.25 – 3.60	Stiff grey brown to grey SILTY CLAY, trace to some sand and gravel with depth
	3.60	End of test pit

In Situ Undrained Shear Strength Test Results

Depth (metres)	Cu(kilopascal)
1.10	90,100
1.60	90,70
2.80	70,70

Some water at about 2.1 metres below existing ground surface, November 7, 2013.

TABLE I (continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP14	0.00 – 0.15	TOPSOIL
	0.15 – 2.55	Red brown to grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)
	2.55	Refusal on weathered BEDROCK

Some water at about 1.7 metres below existing ground surface, November 7, 2013.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP15	0.00 – 0.20	TOPSOIL
	0.20 – 2.00	Red brown to grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)
	2.00	Refusal on weathered BEDROCK

Some water at about 1.7 metres below existing ground surface, November 7, 2013.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP16	0.00 – 0.25	TOPSOIL
	0.25 – 1.70	Red brown to grey silty sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)
	1.70 – 3.60	Grey fine to medium sand, some gravel, cobbles and boulders, trace to some silt, trace clay (GLACIAL TILL)
	3.60	End of test pit in GLACIAL TILL

Some water at about 1.5 metres below existing ground surface, November 7, 2013.

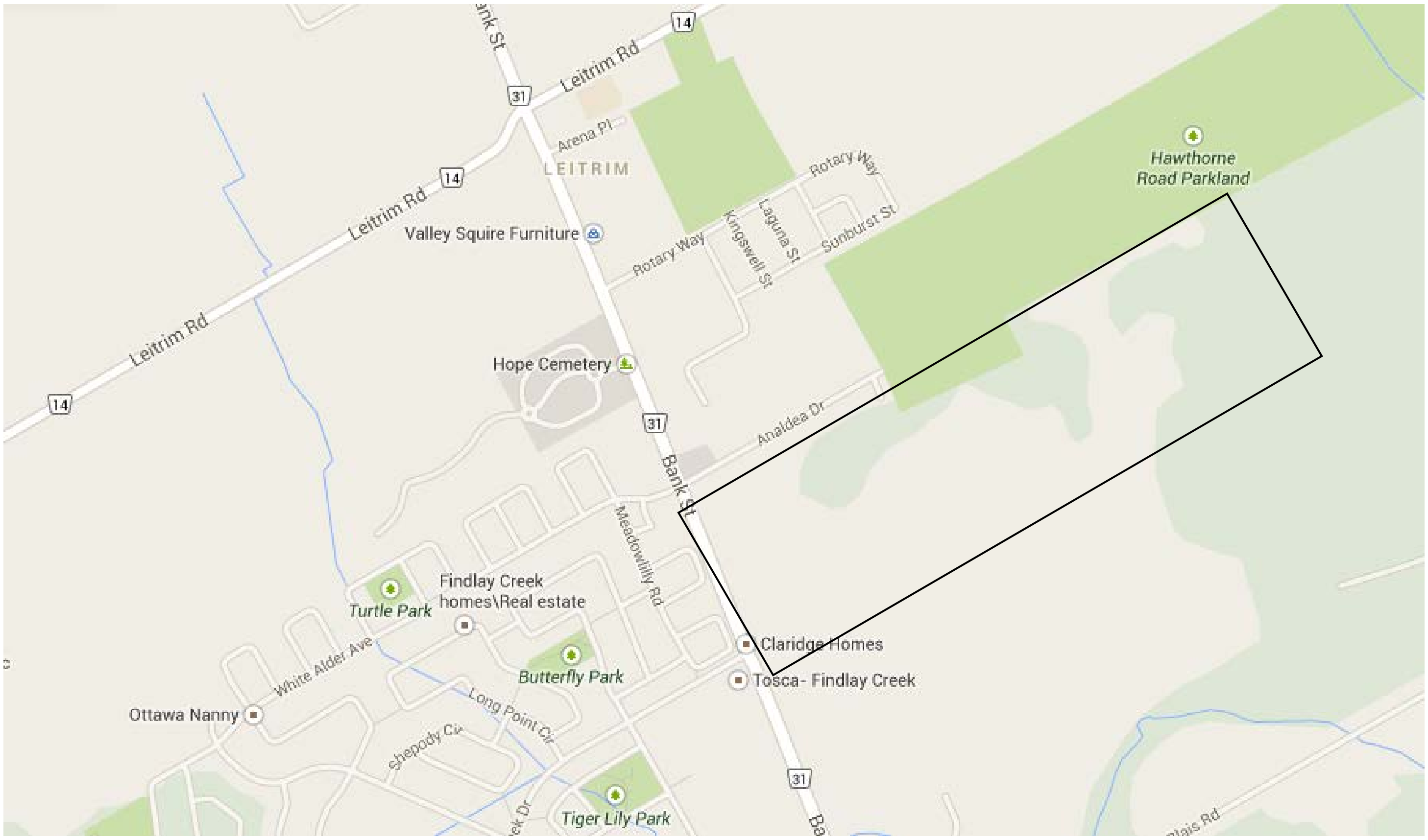
TABLE I (continued)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP17	0.00 – 0.20	TOPSOIL
	0.20 – 1.30	Red brown fine to medium sand, some gravel, cobbles and boulders, trace clay (GLACIAL TILL)
	1.30 – 2.10	Weathered BEDROCK
	2.10	Refusal on grey/black BEDROCK

Some water at about 1.3 metres below existing ground surface, November 7, 2013.

KEY PLAN

FIGURE 1



NOT TO SCALE

Kollaard Associates
Engineers

Project No. 130708

Date January 2014

DRAWING NUMBER:
SITE PLAN, FIGURE 2

LEGEND:

TP1 APPROXIMATE TEST PIT LOCATION

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:
MR. DAVID McGANN

PROJECT:
GEOTECHNICAL INVESTIGATION FOR PROPOSED NEW RESIDENTIAL SUBDIVISION

LOCATION:
4747 BANK STREET
CITY OF OTTAWA, ONTARIO

DESIGNED BY:	DATE:
--	JAN 15, 2014
DRAWN BY: DT	SCALE: N.T.S

KOLLAARD FILE NUMBER:
130708

ATTACHMENT A

Laboratory Test Results for Sulphate, Resistivity and pH

Certificate of Analysis

Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0
Attention:	Mr. Dean Tataryn
PO#:	
Invoice to:	Kollaard Associates Inc.

Report Number:	1326849
Date Submitted:	2013-12-06
Date Reported:	2013-12-12
Project:	130708
COC #:	167306

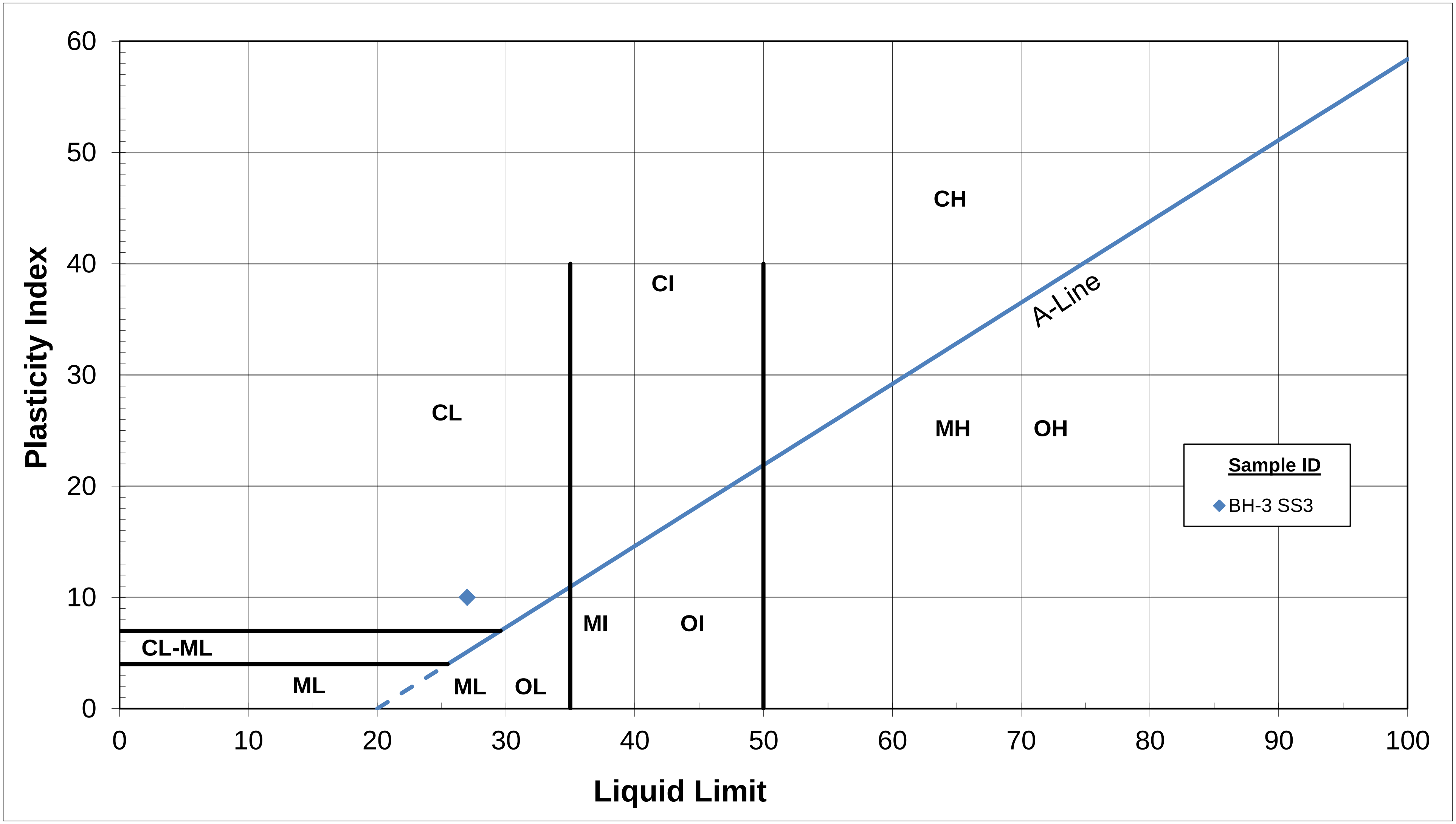
				Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1077677 Soil 2013-11-26 BH3-SS#3 -6'-8'
Group	**Analyte**	**MRL**	**Units**	**Guideline**	
Agri. - Soil	Electrical Conductivity	0.05	mS/cm		0.20
	pH	2.0			7.5
General Chemistry	Cl	0.002	%		<0.002
	Resistivity	1	ohm-cm		5000
	SO4	0.01	%		<0.01

Guideline = *** = Guideline Exceedence**
** = Analysis completed at Mississauga, Ontario.
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

ATTACHMENT B

Stantec Laboratory Test Results for Soils



Stantec

PLASTICITY CHART

4747 Bank Street, Kollaard File #130708

Figure No.

Project No. 122410003

Stantec

Stantec Consulting Ltd
101-2781 Lancaster Rd
Ottawa, ON K1B 1A7
Tel: (613) 738-6075
Fax: (613) 738-6067

December 16, 2013
File: 122410003

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

Reference: **Atterberg Limit & Water Content Results, Kollaard File #130708**

The table below summarizes ASTM D4318 Atterberg Limit & ASTM D2216 Water Content results.

Location	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Natural Water Content
BH-3 SS3	4-6'	26.8	17.5	9.3	25.5

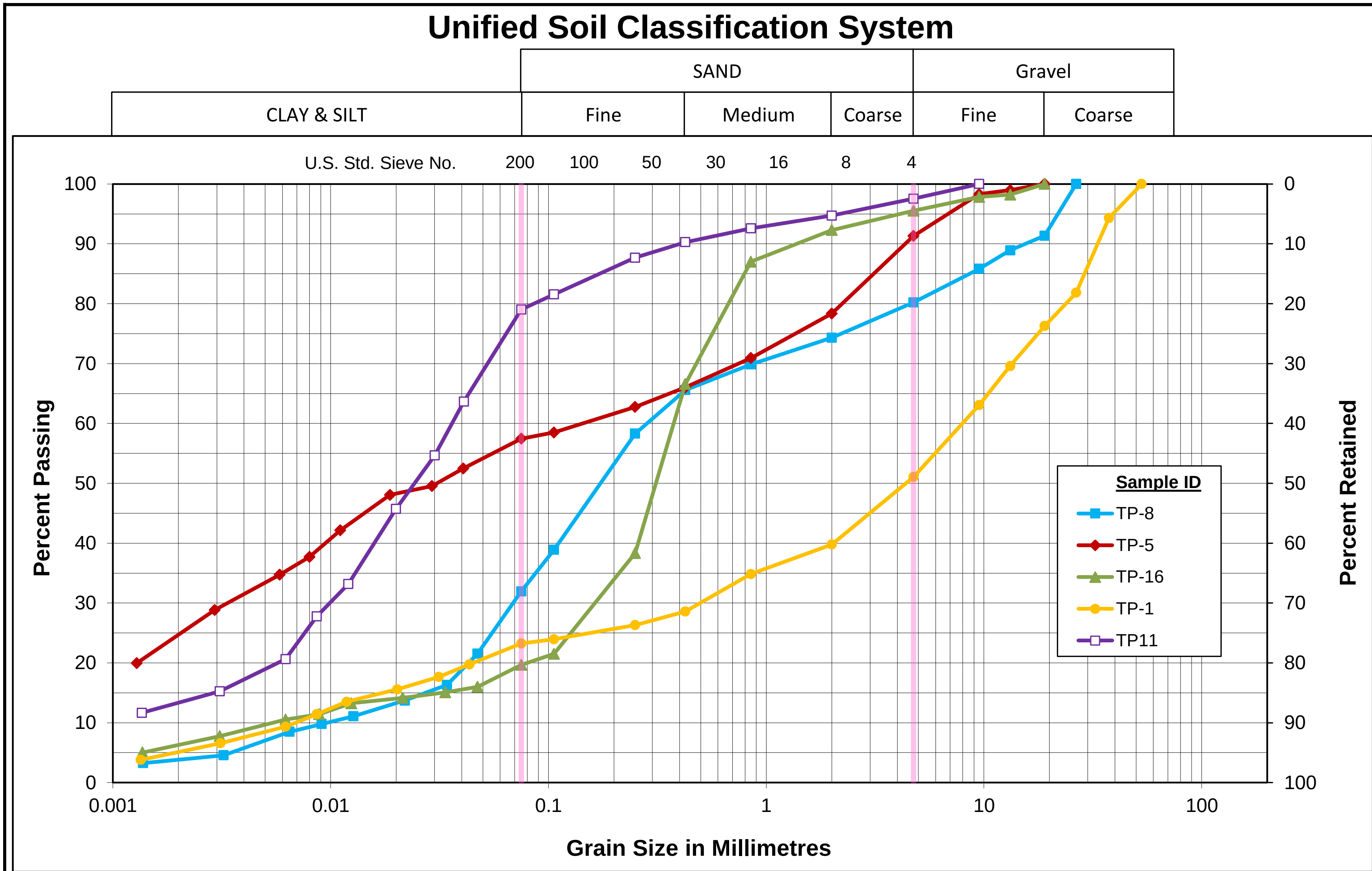
Sincerely,

Stantec Consulting Ltd

Brian Prevost

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

Attachments: Atterberg Limit Chart



Stantec

GRAIN SIZE DISTRIBUTION

4747 Bank Street, Kollaard File #130708

Figure No.

Project No. 122410003

Stantec

2781 Lancaster Road, Suite 101
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils

LS702

ASTM D422

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #130708	Project No.:	122410003
Project:	4747 Bank Street, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	TP-1	Date Sampled:	November 7, 2013
Sample No.:	N/A	Tested By:	Denis Rodriguez
Sample Depth	0.5-1.0m	Date Tested:	December 10, 2013

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	56.21
Sample Weight after Hydrometer and Wash (g)	23.59
Percent Passing No. 200 Sieve (%)	58.0
Percent Passing Corrected (%)	23.09

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	3113.10
Sample Weight After Sieve (g)	3100.20
Percent Loss in Sieve (%)	0.41

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)	67.82
Air Dried Mass (W_a), (g)	68.46
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9907
Air Dried Mass in Analysis (M_a), (g)	56.74
Oven Dried Mass in Analysis (M_o), (g)	56.21
Percent Passing 2.0 mm Sieve (P_{10}), (%)	39.78
Sample Represented (W), (g)	141.30

SIEVE ANALYSIS

Sieve Size mm	Cum. Wt. Retained	Percent Passing
75.0		100.0
63.0		100.0
53.0	0.0	100.0
37.5	176.9	94.3
26.5	565.6	81.8
19.0	738.4	76.3
13.2	946.6	69.6
9.5	1149.8	63.1
4.75	1523.9	51.0
2.00	1874.7	39.8
Total (C + F)[1]	3100.20	
0.850	6.97	34.85
0.425	15.82	28.58
0.250	19.04	26.31
0.106	22.38	23.94
0.075	23.39	23.23
PAN	23.54	

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 7:01 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
10-Dec-13	7:02 AM	1	35.0	6.5	20.5	28.5	19.73	10.78691	9.96839	0.013205	0.04337
10-Dec-13	7:03 AM	2	32.0	6.5	20.5	25.5	17.66	11.25191	9.96839	0.013205	0.03132
10-Dec-13	7:06 AM	5	29.0	6.5	20.5	22.5	15.58	11.71691	9.96839	0.013205	0.02022
10-Dec-13	7:16 AM	15	26.0	6.5	21.0	19.5	13.50	12.18191	9.84835	0.013126	0.01183
10-Dec-13	7:31 AM	30	23.0	6.5	20	16.5	11.42	12.64691	10.09098	0.013286	0.00863
10-Dec-13	8:01 AM	60	20.0	6.5	20.0	13.5	9.35	13.11191	10.09098	0.013286	0.00621
10-Dec-13	11:11 AM	250	16.0	6.5	20	9.5	6.58	13.73191	10.09098	0.013286	0.00311
11-Dec-13	6:44 AM	1423	12.0	6.5	19.5	5.5	3.81	14.35191	10.21619	0.013369	0.00134

Remarks:

Reviewed By: Brian Prevost
Date: December 16/2013

Stantec
2781 Lancaster Road, Suite 101
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils

LS702
ASTM D422

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #130708	Project No.:	122410003
Project:	4747 Bank Street, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	TP-5	Date Sampled:	November 7, 2013
Sample No.:	N/A	Tested By:	Denis Rodriguez
Sample Depth	1.5m	Date Tested:	December 10, 2013

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	51.85
Sample Weight after Hydrometer and Wash (g)	13.94
Percent Passing No. 200 Sieve (%)	73.1
Percent Passing Corrected (%)	57.29

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	564.10
Sample Weight After Sieve (g)	563.80
Percent Loss in Sieve (%)	0.05

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)	85.26
Air Dried Mass (W_a), (g)	85.96
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9919
Air Dried Mass in Analysis (M_a), (g)	52.28
Oven Dried Mass in Analysis (M_o), (g)	51.85
Percent Passing 2.0 mm Sieve (P_{10}), (%)	78.35
Sample Represented (W), (g)	66.18

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	0.0	100.0
13.2	5.8	99.0
9.5	9.6	98.3
4.75	49.0	91.3
2.00	122.1	78.4
Total (C + F)[1]	563.80	
0.850	4.92	70.92
0.425	8.16	66.02
0.250	10.33	62.75
0.106	13.14	58.50
0.075	13.84	57.44
PAN	13.90	

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 7:09 AM

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
10-Dec-13	7:10 AM	1	42.0	6.5	21.5	35.5	52.48	9.70191	9.73081	0.013047	0.04064
10-Dec-13	7:11 AM	2	40.0	6.5	21.5	33.5	49.53	10.01191	9.73081	0.013047	0.02919
10-Dec-13	7:14 AM	5	39.0	6.5	21.0	32.5	48.05	10.16691	9.84835	0.013126	0.01872
10-Dec-13	7:24 AM	15	35.0	6.5	21.5	28.5	42.13	10.78691	9.73081	0.013047	0.01106
10-Dec-13	7:39 AM	30	32.0	6.5	21.5	25.5	37.70	11.25191	9.73081	0.013047	0.00799
10-Dec-13	8:09 AM	60	30.0	6.5	20.0	23.5	34.74	11.56191	10.09098	0.013286	0.00583
10-Dec-13	11:19 AM	250	26.0	6.5	20	19.5	28.83	12.18191	10.09098	0.013286	0.00293
11-Dec-13	6:45 AM	1416	20.0	6.5	19.5	13.5	19.96	13.11191	10.21619	0.013369	0.00129

Remarks:

Reviewed By: Brian Prevost
Date: December 16/2013

Stantec

2781 Lancaster Road, Suite 101
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASTM D422

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #130708	Project No.:	122410003
Project:	4747 Bank Street, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	TP-8	Date Sampled:	November 7, 2013
Sample No.:	N/A	Tested By:	Denis Rodriguez
Sample Depth:	2.5m	Date Tested:	December 10, 2013

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	55.69
Sample Weight after Hydrometer and Wash (g)	32.15
Percent Passing No. 200 Sieve (%)	42.3
Percent Passing Corrected (%)	31.41

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	1345.40
Sample Weight After Sieve (g)	1344.10
Percent Loss in Sieve (%)	0.10

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)	114.61
Air Dried Mass (W_a), (g)	114.90
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9975
Air Dried Mass in Analysis (M_a), (g)	55.83
Oven Dried Mass in Analysis (M_o), (g)	55.69
Percent Passing 2.0 mm Sieve (P_{10}), (%)	74.32
Sample Represented (W), (g)	74.93

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	116.3	91.4
13.2	149.0	88.9
9.5	190.7	85.8
4.75	266.1	80.2
2.00	345.5	74.3
Total (C + F)[1]	1344.10	
0.850	3.33	69.88
0.425	6.54	65.59
0.250	12.00	58.31
0.106	26.55	38.89
0.075	31.76	31.93
PAN	32.05	

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 7:28 AM

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
10-Dec-13	7:29 AM	1	23.0	6.5	20.0	16.5	21.54	12.64691	10.09098	0.013286	0.04725
10-Dec-13	7:30 AM	2	19.0	6.5	20.0	12.5	16.32	13.26691	10.09098	0.013286	0.03422
10-Dec-13	7:33 AM	5	17.0	6.5	20.0	10.5	13.71	13.57691	10.09098	0.013286	0.02189
10-Dec-13	7:43 AM	15	15.0	6.5	20.5	8.5	11.10	13.88691	9.96839	0.013205	0.01271
10-Dec-13	7:58 AM	30	14.0	6.5	20.0	7.5	9.79	14.04191	10.09098	0.013286	0.00909
10-Dec-13	8:28 AM	60	13.0	6.5	20.0	6.5	8.49	14.19691	10.09098	0.013286	0.00646
10-Dec-13	11:38 AM	250	10.0	6.5	20.0	3.5	4.57	14.66191	10.09098	0.013286	0.00322
11-Dec-13	6:46 AM	1398	9.0	6.5	19.5	2.5	3.26	14.81691	10.21619	0.013369	0.00138

Remarks:

Reviewed By: Brian Prevost
Date: December 16/2013

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Stantec

2781 Lancaster Road, Suite 101
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils

ASTM D422

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #130708	Project No.:	122410003
Project:	4747 Bank Street, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	TP11	Date Sampled:	December 5, 2013
Sample No.:	N/A	Tested By:	Denis Rodriguez
Sample Depth	3'-6"	Date Tested:	December 10, 2013

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	51.70
Sample Weight after Hydrometer and Wash (g)	8.67
Percent Passing No. 200 Sieve (%)	83.2
Percent Passing Corrected (%)	78.84

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	204.60
Sample Weight After Sieve (g)	204.30
Percent Loss in Sieve (%)	0.15

SOIL INFORMATION

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

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	88.91
Air Dried Mass (W_a), (g)	89.47
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9937
Air Dried Mass in Analysis (M_a), (g)	52.03
Oven Dried Mass in Analysis (M_o), (g)	51.70
Percent Passing 2.0 mm Sieve (P_{10}), (%)	94.72
Sample Represented (W), (g)	54.59

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	5.0	97.6
2.00	10.8	94.7
Total (C + F)[1]	204.30	
0.850	1.16	92.60
0.425	2.42	90.29
0.250	3.84	87.69
0.106	7.18	81.57
0.075	8.57	79.02
PAN	8.62	

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 7:47 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
December 10, 2013	7:48 AM	1	42.0	6.5	21.0	35.5	63.63	9.70191	9.84835	0.013126	0.04088
December 10, 2013	7:49 AM	2	37.0	6.5	21.0	30.5	54.67	10.47691	9.84835	0.013126	0.03004
December 10, 2013	7:52 AM	5	32.0	6.5	20.0	25.5	45.71	11.25191	10.09098	0.013286	0.01993
December 10, 2013	8:02 AM	15	25.0	6.5	20.0	18.5	33.16	12.33691	10.09098	0.013286	0.01205
December 10, 2013	8:17 AM	30	22.0	6.5	20.5	15.5	27.78	12.80191	9.96839	0.013205	0.00863
December 10, 2013	8:47 AM	60	18.0	6.5	21.0	11.5	20.61	13.42191	9.84835	0.013126	0.00621
December 10, 2013	11:57 AM	250	15.0	6.5	21	8.5	15.24	13.88691	9.84835	0.013126	0.00309
December 11, 2013	6:47 AM	1380	13.0	6.5	19.5	6.5	11.65	14.19691	10.21619	0.013369	0.00136

Remarks:

Reviewed By: Brian Prevost
Date: December 16/2013

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Stantec

2781 Lancaster Road, Suite 101
Ottawa ON, K1B 1A7

Particle-Size Analysis of Soils
LS702
ASTM D422

PROJECT DETAILS

Client:	Kollaard Associates Engineers, File #130708	Project No.:	122410003
Project:	4747 Bank Street, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	TP-16	Date Sampled:	November 7, 2013
Sample No.:	N/A	Tested By:	Denis Rodriguez
Sample Depth	2.3m	Date Tested:	December 10, 2013

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	98.79
Sample Weight after Hydrometer and Wash (g)	78.00
Percent Passing No. 200 Sieve (%)	21.0
Percent Passing Corrected (%)	19.42

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	1184.80
Sample Weight After Sieve (g)	1184.60
Percent Loss in Sieve (%)	0.02

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)	114.94
Air Dried Mass (W_a), (g)	115.44
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9957
Air Dried Mass in Analysis (M_a), (g)	99.22
Oven Dried Mass in Analysis (M_o), (g)	98.79
Percent Passing 2.0 mm Sieve (P_{10}), (%)	92.30
Sample Represented (W), (g)	107.03

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	0.0	100.0
13.2	21.2	98.2
9.5	25.7	97.8
4.75	52.9	95.5
2.00	91.2	92.3
Total (C + F)[1]	1184.60	
0.850	5.65	87.02
0.425	27.55	66.56
0.250	57.75	38.35
0.106	75.78	21.50
0.075	77.74	19.67
PAN	77.86	

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 7:55 AM

HYDROMETER ANALYSIS

Date	Time	Elapsed Time T Mins	H_s Divisions g/L	H_c Divisions g/L	Temperature T_c °C	Corrected Reading $R = H_s - H_c$ g/L	Percent Passing P %	L cm	η Poise	K	Diameter D mm
10-Dec-13	7:56 AM	1	24.0	6.5	19.5	17.5	16.00	12.49191	10.21619	0.013369	0.04725
10-Dec-13	7:57 AM	2	23.0	6.5	19.5	16.5	15.08	12.64691	10.21619	0.013369	0.03362
10-Dec-13	8:00 AM	5	22.0	6.5	19.5	15.5	14.17	12.80191	10.21619	0.013369	0.02139
10-Dec-13	8:10 AM	15	21.0	6.5	19.5	14.5	13.25	12.95691	10.21619	0.013369	0.01242
10-Dec-13	8:25 AM	30	19.0	6.5	20	12.5	11.43	13.26691	10.09098	0.013286	0.00884
10-Dec-13	8:55 AM	60	18.0	6.5	21.0	11.5	10.51	13.42191	9.84835	0.013126	0.00621
10-Dec-13	12:05 PM	250	15.0	6.5	21	8.5	7.77	13.88691	9.84835	0.013126	0.00309
11-Dec-13	6:48 AM	1373	12.0	6.5	19.5	5.5	5.03	14.35191	10.21619	0.013369	0.00137

Remarks:

Reviewed By: Britan Prevost
Date: December 16/2013