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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1946 SCOTT STREET
CITY OF OTTAWA, ONTARIO

Project # 150263

Submitted to:

Independent Development Group Ltd.
61 Forest Hill Avenue
Ottawa, Ontario
K2C 1P7

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October 8, 2015 (rev. August 30, 2017)

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October 8, 2015 (rev. August 30, 2017) 150263

Independent Development Group Ltd.
61 Forest Hill Avenue
Ottawa, Ontario
K2C 1P7

RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1946 SCOTT 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. 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

Preliminary plans are being prepared to construct a residential development consisting of a twelve storey condominium at 1946 Scott Street in the City of Ottawa, Ontario (see Key Plan, Figure 1). The site has about 22 metres of frontage onto Scott Street and the total site area is approximately 0.16 acres (0.06 hectares).

Preliminary plans indicate that the proposed building will be a twelve storey, wooden framed structure with conventional concrete spread footing foundations. The proposed building will be serviced by municipal water and sanitary services. Surface drainage for the proposed building will be by means of swales and storm sewers.

The site is located within an area of mostly residential development with one commercial property adjacent to the site on the west side. The site is bordered on the north by Scott Street followed by residential development, on the east by a vacant lot followed by West Village Private and residential development, on the south by residential development and on the west by an industrial property (Hydro Ottawa Clifton Road transformer substation). Currently, a fenced off asphaltic concrete surfaced lot exists at the site. It is understood that the site is scheduled for a proposed multi unit residential redevelopment.

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by deposits of sandy silt to silty sand glacial till followed by shallow bedrock. Bedrock geology maps indicate that the bedrock underlying the site may consist of limestone with possible shaly partings of the Ottawa Formation.

PROCEDURE

The field work for this investigation was carried out on August 12, 2015 at which time three boreholes, numbered BH1 to BH3 were put down at the site using a truck mounted drill rig equipped with a hollow stem auger owned and operated by Capital Cutting and Coring Ltd. of Ottawa, Ontario.

Sampling of the overburden materials encountered at the boreholes was carried out at regular 0.75 metre depth intervals using a 50 millimetre diameter drive open conventional split spoon sampler in conjunction with standard penetration testing to depths ranging from about 1.4 to 2.4 metres below the existing ground surface (ASTM D-1586 – Penetration Test and Split Barrel Sampling of Soils and ASTM D-1587 – Thin Walled Tube Sampling of Soils). BH3 was continued below about 2.4 metres with coring to confirm the presence of bedrock.

The subsurface soil conditions at BH1, BH2 and BH3 were identified based on visual examination of the samples recovered, the results of the standard penetration tests as well as laboratory test results on select samples. Groundwater conditions at the boreholes were noted at the time of drilling. A standpipe was installed at BH3 for subsequent ground water level monitoring. A standpipe was installed at BH1 for subsequent ground water level monitoring. The boreholes were loosely backfilled with the auger cuttings upon completion of drilling. The drilling was conducted in conjunction with a Phase II Environmental Site Assessment being carried out by DST Consulting Engineers Inc.

One soil sample (BH3) was submitted to determine the particle size distribution analysis (ASTM D422). 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.

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 BH1, BH2 and BH3 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 boreholes is provided in the attached Record of Borehole Sheets following the text of this report. The borehole logs indicate the subsurface conditions at the specific drill locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at locations other than borehole locations may vary from the conditions encountered at the boreholes.

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

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

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

Fill

Fill material was encountered from the surface at all three boreholes at depths ranging from about 1.2 to 2.4 metres. The fill was observed to consist of asphaltic concrete followed by grey brown sand, some gravel and a trace of silt. The fill material was fully penetrated at all three borehole locations.

Glacial Till

A deposit of glacial till was encountered beneath the fill materials at all of the borehole locations. The glacial till consisted of gravel, cobbles and boulders, in a matrix of grey silty sand/sandy silt with a trace of silty clay. The results of standard penetration testing carried out in the glacial till material, which range from 19 to 66 blows per 0.3 metres with an average value of 43 blows per 0.3 metres, indicate a dense to very dense state of packing.

One soil sample (BH3 - 2.4 metres) was submitted to Stantec for hydrometer testing (ASTM D422). The results of the hydrometer testing indicated that the sample consists of about 3.0 percent clay, 26 percent silt and clay size particles, 59 percent sand and 15 percent gravel. The results are located in Attachment A.

Bedrock

Bedrock was encountered at all three boreholes ranging from 1.4, 2.1 and 2.4 metres, respectively. At BH3, the bedrock was cored to verify the quality of the upper bedrock. Two 1.5 metres core runs were performed. The total core run length was 3.07 metres. The bedrock consists of dolomite and limestone of the Oxford Formation. Fracturing of the core samples is mostly along near horizontal bedding planes.

A measure of the condition of the bedrock core obtained from the borehole can be represented as a percentage of Total Core Recovery (T.C.R.), Solid Core Recovery (S.C.R.) and Rock Quality Designation (R.Q.D.). The amount of core lost during recovery of the bedrock was low, giving a T.C.R. values of 90 and 97 percent, respectively. It is noted that the majority of the core loss occurred in the upper portion of the core runs.

The S.C.R. values vary from about 16 to 96 percent as follows:
From the bedrock surface to 1.5 meters below the bedrock surface the S.C.R. = 89 percent.
Between 1.5 and 3.0 m below the bedrock surface the S.C.R. = 97 percent.

The R.Q.D. values are as follows:
From the bedrock surface to 1.5 meters below the bedrock surface the R.Q.D = 89 percent.
Between 1.5 m and 3.0 m below the bedrock surface the R.Q.D = 95 percent.

Groundwater

All of the boreholes were dry at the time of the field work. On September 4, 2015, groundwater was measured in a standpipe installed in BH3 at a depth of about 7.3 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 %	0.005	Negligible concern
pH	5.0 < pH	9.1	Slightly Basic Negligible concern
Resistivity	R < 1500 ohm-cm	3030	Negligible concern
Sulphates (SO_4)	SO_4 > 0.1%	0.01	Negligible concern

Based on the chemical test results, ASTM C 150 Type II cement may be used for this proposed development. No special protection is required for reinforcement steel within the concrete walls. The laboratory results are present at the end of this report.

PROPOSED RESIDENTIAL DEVELOPMENT BUILDING FOUNDATIONS

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 Building

With the exception of any surficial fill and/or topsoil, the subsurface conditions encountered at the boreholes advanced during the investigation are suitable for the support of the proposed residential building on conventional spread footing foundations. It is suggested that the building be founded either directly on the underlying bedrock or on engineered fill place on the underlying bedrock.

For the proposed below grade basement foundation, a maximum allowable bearing pressure of 450 kilopascals using serviceability limit states design and a factored ultimate bearing resistance of 1200 kilopascals using ultimate limit states design, may be used for the design of conventional strip footings, a minimum of 0.6 metres in width, or pad footings founded on suitably constructed engineered fill which has been placed on bedrock free of deleterious material.

There are no grade raise restrictions associated with the above allowable bearing pressures.

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 predictable performance of the proposed foundation, all existing fill, topsoil, and/or any debris or deleterious materials and any disturbed subgrade material should be removed from within the proposed foundation areas. The exposed subgrade should consist of bedrock. The subgrade surface should then be inspected and approved by geotechnical personnel.

Any fill required to raise the footings for the proposed residential building to founding level should consist of imported granular material (engineered fill). The engineered fill should consist of granular material meeting Ontario Provincial Standards Specifications (OPSS) requirements for Granular A or Granular B Type II and should be compacted in maximum 300 millimetre thick loose lifts to 98 percent of the standard Proctor maximum dry density. It is considered that the engineered fill should be compacted using dynamic compaction with a large diameter vibratory steel drum roller or diesel plate compactor. If a diesel plate compactor is used, the lift thickness may need to be restricted to less than 300 mm to achieve proper compaction. Compaction should be verified by a suitable field compaction test method.

To allow the spread of load beneath the footings, the engineered fill should extend horizontally from the edges of the footing a minimum distance of 0.3 m and then down and out at 1 horizontal to 1 vertical, or flatter. The excavations for the proposed residential building should be sized to accommodate this fill placement. Currently, OPSS documents allow recycled asphaltic concrete to be used in Granular A and Granular B Type II materials. Since the source of recycled material cannot be determined, it is suggested that any granular materials used below the founding level be composed of virgin materials only.

The native glacial till 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.

All shallow exterior footings if present and those in any unheated parts of the structures should be provided with at least 1.5 metres of earth cover for frost protection purposes. A conventional, perforated perimeter drain, with a 150 millimetre surround of 20 millimetre minus crushed stone, should be provided at founding level and should lead by gravity flow to a sump and/or to a storm sewer. The proposed basement should also be provided with under floor drains consisting of perforated pipe with a surround of 20 millimetre minus crushed stone to reduce the potential for buildup of hydrostatic pressure below the basement floor.

Frost Protection Requirements for Spread Footing Foundations

Where the native subgrade consists of material other than bedrock, 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.

Where the foundations for the proposed residential buildings will be place on either sound bedrock or on an engineered granular pad placed on sound bedrock frost protection for the foundation is not required. Both the native bedrock and the engineered granular pad are considered non-frost susceptible when provided with good drainage.

2012 OBC Part 9.12.2.2 provides the following information:

Table 9.12.2.2 - Minimum Depths of Foundation

Type of Soil: Rock, Coarse grained soils with good soil drainage
Minimum Depth of Foundation (Containing No Heated Space): No Limit

Because the subgrade and granular pad will consist of non-frost-susceptible bedrock and compacted granular 'B' Type II or granular ‘A’, no frost protection is required for this proposed residential building placed on bedrock or on an engineering pad over bedrock.

Building Basement Foundation Walls 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 boreholes, the above perimeter drainage system should adequately handle any groundwater seepage to the basements.

Excluding the bedrock at the site, the native 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.

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

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

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

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

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.

Building Structure Floor Slab

As stated above, it is expected that the proposed building will be founded on engineered fill or bedrock. For predictable performance of the proposed concrete floor slab all existing fill material, topsoil and any otherwise deleterious material should be removed from below the proposed floor slab areas. The exposed native subgrade surface should then be inspected and approved by geotechnical personnel. Any soft or disturbed areas evident should be subexcavated and replaced with suitable engineered fill.

The fill materials beneath the proposed concrete floor slab should consist of a minimum of 150 millimetre thickness of crushed stone meeting OPSS Granular A immediately beneath the concrete floor slab followed by sand, or sand and gravel meeting the OPSS for Granular B Type I, or crushed stone meeting OPSS grading requirements for Granular B Type II, or other material approved by the Geotechnical Engineer. The fill materials should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage of the concrete. The saw cut depth should be about one quarter of the thickness of the slab. The crack control cuts should be placed at a grid spacing not exceeding about 5 metres.

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

If the building basement will be unheated, the footings/grade beams, foundation walls and floor slabs will require protection from frost effects. Should the building basement not be heated we will be pleased to provide guidelines for suitable frost protection.

Seismic Design for the Proposed Residential Buildings

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 B Rock. The subsurface conditions below the proposed footing design level consists of a thin veneer of glacial till over bedrock at a depth of about 1.4 to 2.4 metres. As indicated above, the bedrock is sound at a depth of 1.5 metres below the bedrock surface with an RQD of 95%. The bedrock consists of carbonic rock and is either limestone or dolostone.

The shear wave velocity of dolostone typically ranges between 1.5km/s and 3.9km/s and of limestone between 1.5km/s and 3.7km/s. As such specific seismic site classification using shear wave velocity testing for the site could result in a seismic site classification of Site Class A, if required.

Potential for Soil Liquefaction

The information obtained from boreholes indicate that the native deposits underlying the site consist of a relatively thin layer of glacial till then bedrock. Based on the native soil and bedrock conditions at the site, it is considered that no damage to the proposed residential building should occur due to liquefaction of the native subgrade under seismic conditions.

SITE SERVICES

Excavation

The excavations for the site services will be carried out through fill, glacial till and/or bedrock. 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. Where space constraints dictate, the excavation and backfilling operations should be carried out within a tightly fitting, braced steel trench box. Excavation walls within bedrock may be made near vertical.

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

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

Pipe Bedding and Cover Materials

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

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

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

Trench Backfill

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

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

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

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

ACCESS ROADWAY PAVEMENTS

In preparation for pavement construction at this site the topsoil and any soft, wet or deleterious materials should be removed from the proposed access roadway area. The exposed sub-grade should be inspected and approved by geotechnical personnel and any soft areas evident should be sub-excavated and replaced with suitable earth borrow approved by the geotechnical engineer. The sub-grade should be shaped and crowned to promote drainage of the roadway area granular. Following approval of the preparation of the sub-grade, the pavement granular may be placed.

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

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

50 millimetres of hot mix asphaltic concrete (HL3) over
150 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase
(50 or 100 millimetre minus crushed stone)

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

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

CONSTRUCTION CONSIDERATIONS

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

The subgrade for the site services, access roadway and parking 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 silty sand and silty clay deposits within the glacial till material 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
Aug.30.2017
S.E. deWit
100079612
PROVINCE OF ONTARIO

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

Steve deWit, P.Eng.

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

RECORD OF BOREHOLE BH1

PROJECT: Proposed Residential Development
CLIENT: Independent Development Group Ltd.
LOCATION: 1946 Scott Street, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 150263
DATE OF BORING: August 12, 2015
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80 / REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80	DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface									
0	ASPHALTIC CONCRETE		0.00							
	Grey brown sand, some gravel, trace silt (FILL)			1	SS	15				Borehole dry August 12, 2015.
1				2	SS	14				
				3	SS	50				
	End of Borehole, refusal on BEDROCK		1.40							
2										
3										
4										
5										
6										
7										
8										

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: Independent Development Group Ltd.
LOCATION: 1946 Scott Street, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 150263
DATE OF BORING: August 12, 2015
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	ASPHALTIC CONCRETE		0.00							
	Grey brown sand, some gravel, trace silt (FILL)			1	SS	18				Borehole dry August 12, 2015.
1				2	SS	26				
	Grey brown silty sand, trace clay and gravel (GLACIAL TILL), some weathered bedrock		1.20	3	SS	19				
2				4	SS	50				
	End of Borehole, refusal on BEDROCK		2.10							
3										
4										
5										
6										
7										
8										

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: Independent Development Group Ltd.
LOCATION: 1946 Scott Street, Ottawa, ON
PENETRATION TEST HAMMER: 63.5kg, Drop, 0.76mm

PROJECT NUMBER: 150263
DATE OF BORING: August 12, 2015
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
	Ground Surface					
0	ASPHALTIC CONCRETE	0.00				
	Grey brown sand, some gravel, trace silt (FILL)		1	SS	25	
1			2	SS	20	
	Grey brown silty sand, trace clay and gravel (GLACIAL TILL), becoming weathered bedrock with depth	1.20	3	SS	24	
2			4	SS	66	
	Advanced corehole through BEDROCK	2.44	5	RC		
4	Advanced corehole through BEDROCK (Limestone)	3.98	6	RC		
6–9						Borehole dry, August 12, 2015. Water level measured in standpipe at about 7.3 metres below existing ground surface, September 4, 2015.
10	End of corehole in BEDROCK (Limestone)	10.20				

UNDIST. SHEAR STRENGTH × Cu, kPa × 20 40 60 80
REM. SHEAR STRENGTH ○ Cu, kPa ○ 20 40 60 80
DYNAMIC CONE PENETRATION TEST blows/300 mm 10 30 50 70 90
ADDITIONAL LAB TESTING
STRATA PLOT

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

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

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

PENETRATION RESISTANCE

Standard Penetration Resistance, N

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

Dynamic Penetration Resistance

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

WH

_Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drih rig.

PM

Sampler advanced by manual pressure.

SOIL TESTS

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

SOIL DESCRIPTIONS

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

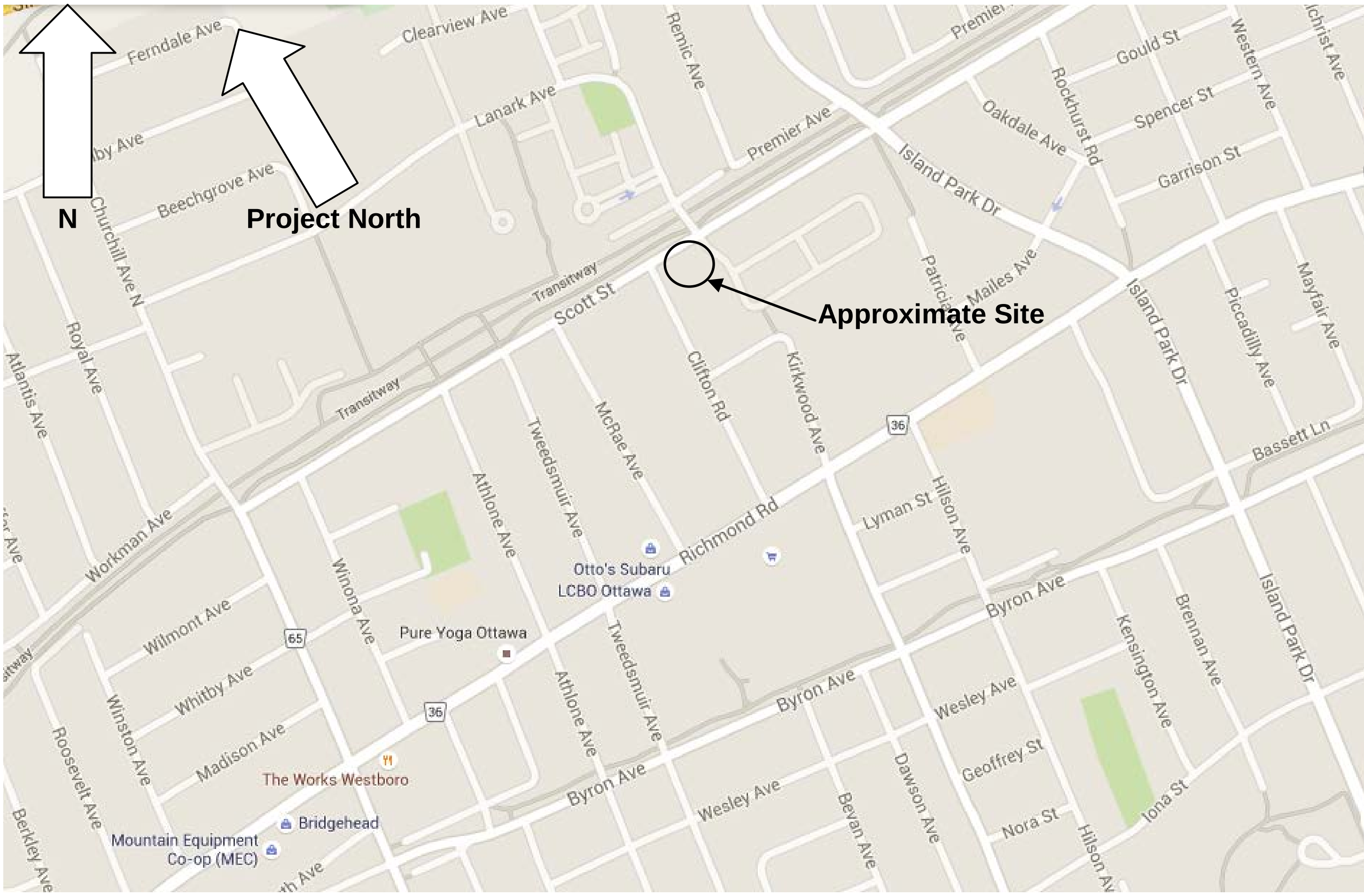
Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to100
Very Stiff	over100

LIST OF COMMON SYMBOLS

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

KEY PLAN

FIGURE 1



NOT TO SCALE

Kollaard Associates
Engineers

Project No. 150263

Date Oct 2015 rev.Aug 30/17

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:
INDEPENDENT DEVELOPMENT GROUP LTD.

PROJECT:
GEOTECHNICAL INVESTIGATION FOR RESIDENTIAL BUILDING

LOCATION:
1946 SCOTT STREET
CITY OF OTTAWA, ONTARIO

DESIGNED BY:	DATE:
--	AUG. 28, 2015
DRAWN BY:	SCALE:
DT	N.T.S

KOLLAARD FILE NUMBER:
150263

Laboratory Test Results for Chemical Properties

EXOVA ENVIRONMENTAL ONTARIO

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:	1515900
Date Submitted:	2015-08-13
Date Reported:	2015-08-21
Project:	150263
COC #:	175690

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: ______________________________

Shyla Monette
Team Leader, Inorganics

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

Exova 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.

Exova (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.

Exova (Mississauga) is accredited for specific parameters by SCC, Standards Council of Canada (to ISO 17025)

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. Exova recommends consulting the official provincial or federal guideline as required.

EXOVA ENVIRONMENTAL ONTARIO

Certificate of Analysis

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Attention:	Mr. Dean Tataryn
PO#:	
Invoice to:	Kollaard Associates Inc.

Report Number:	1515900
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Date Reported:	2015-08-21
Project:	150263
COC #:	175690

Group	Analyte	MRL	Units	Guideline	Lab I.D. 1195070 Sample Matrix Soil Sample Type Sampling Date 2015-08-12 Sample I.D. BH 15-02
Agri. - Soil	pH	2.0			9.1
General Chemistry	Cl	0.002	%		0.005
	Electrical Conductivity	0.05	mS/cm		0.33
	Resistivity	1	ohm-cm		3030
	SO4	0.01	%		0.01

Guideline = *** = Guideline Exceedence**

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in 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

Certificate of Analysis

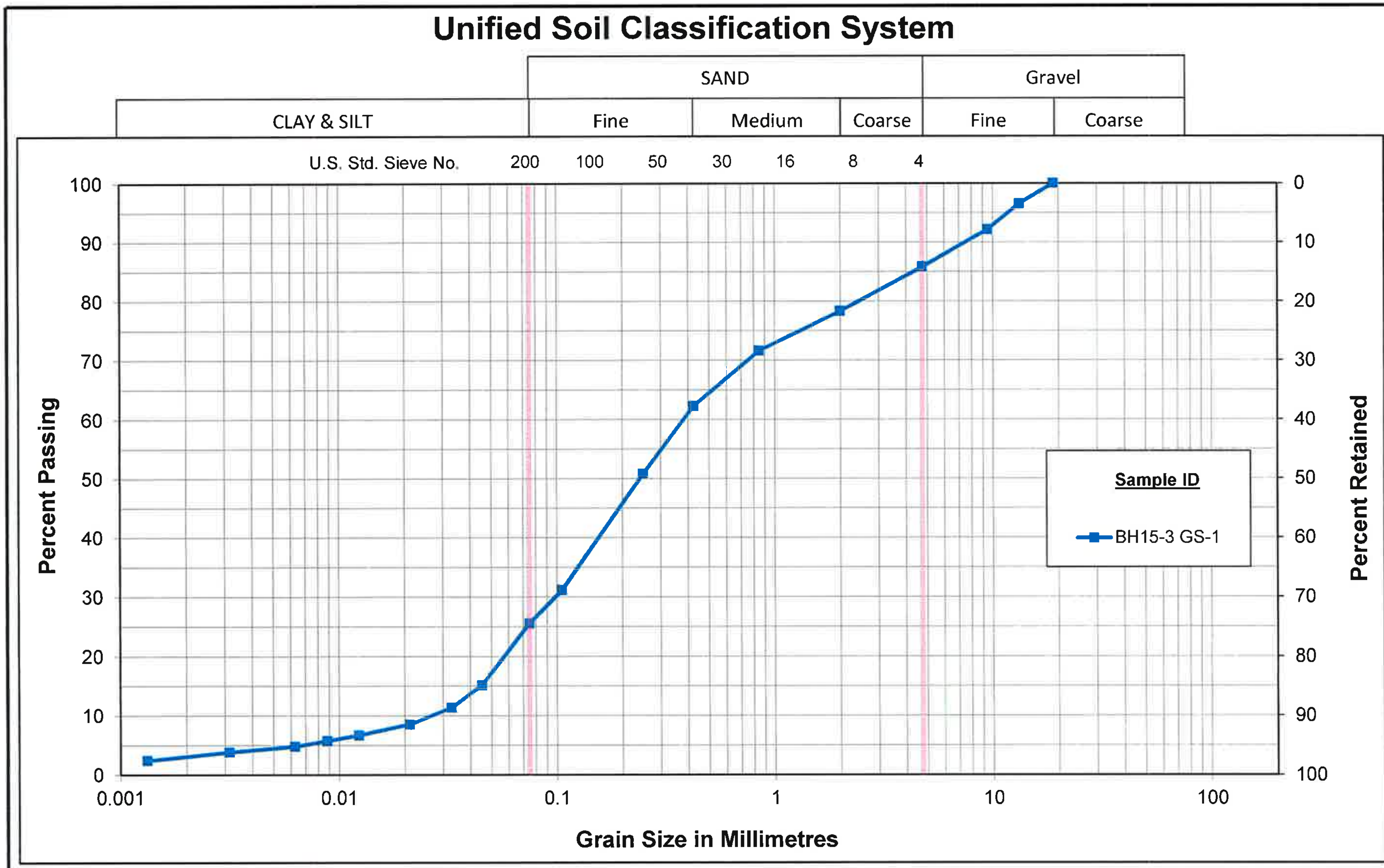
Client:	Kollaard Associates Inc. 210 Prescott St., Box 189 Kemptville, ON K0G 1J0
Attention:	Mr. Dean Tataryn
PO#:	
Invoice to:	Kollaard Associates Inc.

Report Number:	1515900
Date Submitted:	2015-08-13
Date Reported:	2015-08-21
Project:	150263
COC #:	175690

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 292290 **Analysis/Extraction Date** 2015-08-14 **Analyst** DML			
Method Ag Soil			
pH			90-110
Method Cond-Soil			
Electrical Conductivity			85-115
Method Resistivity - soil			
Resistivity			
Run No 292836 **Analysis/Extraction Date** 2015-08-20 **Analyst** NP			
Method C SM4500-SO4--D			
SO4	<0.01 %	98	70-130
Run No 292839 **Analysis/Extraction Date** 2015-08-20 **Analyst** NP			
Method C CSA A23.2-4B			
Chloride	<0.002 %	100	90-110

Laboratory Test Results for Physical Properties



Stantec	**GRAIN SIZE DISTRIBUTION** Kollaard Associates Engineers, File #150263 1946 Scott Street, Ottawa	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 #150263	Project No.:	122410003
Project:	1946 Scott Street, Ottawa	Test Method:	LS702
Material Type:	Soil	Sampled By:	Kollaard Associates Engineers
Source:	BH15-3	Date Sampled:	August 12, 2015
Sample No.:	GS-1	Tested By:	Denis Rodriguez
Sample Depth:	±8'1"	Date Tested:	August 31, 2015

WASH TEST DATA

Oven Dry Mass In Hydrometer Analysis (g)	80.90
Sample Weight after Hydrometer and Wash (g)	55.94
Percent Passing No. 200 Sieve (%)	30.9
Percent Passing Corrected (%)	24.16

PERCENT LOSS IN SIEVE

Sample Weight Before Sieve (g)	621.50
Sample Weight After Sieve (g)	620.60
Percent Loss in Sieve (%)	0.14

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	24	g

CALCULATION OF DRY SOIL MASS

Oven Dried Mass (W_o), (g)	157.80
Air Dried Mass (W_a), (g)	158.11
Hygroscopic Corr. Factor ($F=W_o/W_a$)	0.9980
Air Dried Mass in Analysis (M_a), (g)	81.06
Oven Dried Mass in Analysis (M_o), (g)	80.90
Percent Passing 2.0 mm Sieve (P_{10}), (%)	78.31
Sample Represented (W), (g)	103.31

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	21.9	96.5
9.5	48.9	92.1
4.75	88.2	85.8
2.00	134.8	78.3
Total (C + F)[1]	620.60	
0.850	6.94	71.59
0.425	16.57	62.27
0.250	28.38	50.84
0.106	48.69	31.18
0.075	54.54	25.52
PAN	55.58	

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
31-Aug-15	10:11 AM	1	21.0	5.0	24.5	16.0	15.15	12.95691	9.07441	0.012599	0.04535
31-Aug-15	10:12 AM	2	17.0	5.0	24.5	12.0	11.36	13.57691	9.07441	0.012599	0.03283
31-Aug-15	10:15 AM	5	14.0	5.0	24.5	9.0	8.52	14.04191	9.07441	0.012599	0.02111
31-Aug-15	10:25 AM	15	12.0	5.0	24.5	7.0	6.63	14.35191	9.07441	0.012599	0.01232
31-Aug-15	10:40 AM	30	11.0	5.0	24.0	6.0	5.68	14.50691	9.17830	0.012671	0.00881
31-Aug-15	11:10 AM	60	10.0	5.0	24.0	5.0	4.74	14.66191	9.17830	0.012671	0.00626
31-Aug-15	2:20 PM	250	9.0	5.0	22.5	4.0	3.79	14.81691	9.50295	0.012894	0.00314
01-Sep-15	10:10 AM	1440	7.5	5.0	22.0	2.5	2.37	15.04941	9.61570	0.012970	0.00133

Remarks:

Reviewed By: Brian Prevost
Date: September 8/2015