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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL SUBDIVISION
PART 1, PLAN 5R-10284
2050 DUNROBIN ROAD
WEST CARLETON WARD
CITY OF OTTAWA, ONTARIO

Submitted to:

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PROJECT #: 200977

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EXECUTIVE SUMMARY

Kollaard Associates Inc. (Kollaard) is pleased to present the results of the geotechnical investigation completed for the proposed Residential Subdivision Development to be located at 2050 Dunrobin Road, City of Ottawa Ontario.

The geotechnical investigation was completed in conjunction with a Phase One Environmental Site Assessment, Hydrogeological Study, Topographic Survey, Stormwater Management Plan Report as well as civil engineering drawings which are reported under separate covers.

The draft plan and civil engineer drawings indicate that the proposed residential development will consist of 8 lots to be developed for single family residential purposes. The development will occupy a 9 hectare tract of land on the northeast side of Dunrobin Road. The development will be serviced by a single road extended perpendicularly from Dunrobin Road and terminated with a Cul-de-sac.

Since the proposed structures will be relatively light (conventional wood frame housing), the bedrock surface is fairly shallow and the soil overburden is not highly compressive, the subsurface investigation was completed by means of test pits in keeping with Section 2.3 of the Geotechnical Investigation and Reporting Guidelines for the City of Ottawa.

The fieldwork for this subsurface investigation was carried on July 31, 2007 at which time fourteen test pits numbered TP1 to TP14, were put down at the site using a tire mounted backhoe supplied and operated by a local excavating contractor. The field work was supervised on a full time basis by Kollaard. The test pits revealed that the subsurface conditions are, in general, comprised of a layer of topsoil followed by a layer of fine to medium sand and/or silty sand followed by glacial till, then bedrock. A thin layer of weather silty clay crust was encountered below the silty sand at two test pit locations. In general, the ground surface slopes downward from Dunrobin Road to the northeast ranging in elevation from about 79.0 metres to about 75.0 metres.

Ground water was encountered at depths of between 0.6 and 1.6 metres below the existing ground surface at the northeastern or lower end of the site (elevations between 73.9 and 75.27 m. With the exception of test pits TP13 and TP14, the groundwater, where encountered, was slightly above the bedrock surface.

Based on the findings of the subsurface investigation, there is no sensitive marine clay deposits present at the site or other subsurface geotechnical conditions that would preclude normal

residential construction. There is no potential that the development of the site will cause adverse effects or aggravate a hazard either on site or elsewhere.

The site has been classified as seismic site Class C. The on-site soils are not considered to be liquefiable during a seismic event.

The geotechnical investigation has revealed that conditions are suitable for the construction of the proposed residential buildings on spread and strip footing foundations founded on engineered fill or on a native silty sand / sand / silty clay or glacial till subgrade. Footings prepared as per the geotechnical recommendations in the report may be designed using a serviceability limit state bearing pressure (SLS) of 100 kPa when founded on the native soils or an SLS of 150 kPa when founded on bedrock or engineered fill placed on bedrock.

Based on lot grading considerations, the proposed underside of footing (USF) elevation for each dwelling will be set between about 0.3 metres below the existing ground surface to about 0.3 above the existing ground surface at the proposed dwelling location. Where the USF is above the native subgrade surface, the foundation will be supported by engineered fill. The proposed grading has resulted in a grade raise approaching 3 or more metres at some locations. This grade raise is considered acceptable from a geotechnical point of view.

Excavation of bedrock or deep excavations are not expected at the site. As such, seepage of groundwater into the excavations is not expected. Surface water flowing into excavations during rainfall or snow melt events should controlled by redirecting surface drainage and by pumping.

The roadway should be constructed following the minimum structure for local residential roadways and should consist of 90 mm of asphaltic concrete underlaing by 150 mm of OPSS Granular A base over 300 mm of OPSS Granular B Type II sub-base. A non-woven 6 ounce per square yard geotextile fabric should be placed between the native subgrade and the granular sub-base.

The above and other related considerations are discussed in greater detail in the main body of the report.

RECORD OF TEST PIT SHEETS

LIST OF FIGURES

LIST OF ATTACHMENTS

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March 12, 2025 200977

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Attention: Zbigniew and Teresa Hauderowicz

RE: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL SUBDIVISION
PART 1, PLAN 5R-10284, 2050 DUNROBIN ROAD
WEST CARLETON WARD
CITY OF OTTAWA, ONTARIO

1 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at the site of the proposed residential subdivision at 2050 Dunrobin Road in the City of Ottawa, Ontario. Plans are being prepared to construct a residential subdivision within about a 9 hectare tract of land located on the northeast side of Dunrobin Road approximately 340 metres southeast of Constance Lake Road, West Carleton Ward in the City of Ottawa, Ontario (see Key Plan, Figure 1).

The purpose of the investigation was to:

- Identify the general subsurface conditions at the site by means of a limited number of test pits.
- Based on the factual information obtained, provide engineering guidelines for the geotechnical aspects of the design of the project together with construction considerations, which could influence design decisions.

2 BACKGROUND INFORMATION AND SITE GEOLOGY

2.1 Existing Site Conditions

The proposed development has in general a rectangular shape and extends from Dunrobin Road to the former CN railway tracks located along the northeast side of the site. The ground surface at the site, in general, slopes downward from Dunrobin Road at about 0.2 to 2 percent to the rear property line at the northeast side. The proposed development site is part of the Harwood Creek watershed. Harwood Creek is a tributary to Constance Lake and is located about 80 metres southeast of the rectangular portion of the proposed development.

A former single family dwelling existed in about the centre of the site some 25 metres from Dunrobin Road. There are some matures trees in the area of the former dwelling, along the property lines within the northeast portion of the site and along a fence line located in about the centre of the site. The vegetative communities on the southwest portion of the site predominately consisted of Forb Meadow which transitions to Buckthorn Deciduous Shrub Thickets through the central portion of the site. The northeast end of the site adjacent the railway corridor is occupied by fresh-moist poplar deciduous woodland. A tailwater section of the Flood Plain of the Harwood Creek extends onto the site covering a significant portion of the eastern about 100 metres of the site.

2.2 Proposed Development

It is understood that the proposed residential development will consist of eight lots ranging in size from about 0.8 to 1.9 hectares in plan area for single family dwelling construction purposes. It is understood that the proposed construction will consist of light residential single family dwellings of wood frame construction with full depth conventional concrete foundations. A portion of the dwellings may be faced with brick or stone. Dwellings will be serviced with private wells and septic systems. Surface drainage will be by means of sheet flow, swales and drainage ditches.

2.3 Site Geology

Based on a review of the surficial geology map for the site area, it is expected that the site is underlain by a relatively thin veneer of overburden material over shallow bedrock. The bedrock geology map indicates that the bedrock underlying the site consists of limestone and dolomite of the Oxford formation and sandstone of the Nepean formation.

A review of Ministry of Environment Well Records for drinking water wells put down on the site indicates that the overburden thickness varies from about 0.3 metres to about 4.6 metres. The underlying bedrock is indicated by the well records to consist of limestone and/or limestone with interbedded sandstone followed by granite.

3 SUBSURFACE INVESTIGATION

The fieldwork for this subsurface investigation was carried on July 31, 2007 at which time fourteen test pits numbered TP1 to TP14, were put down at the site using a tire mounted backhoe supplied and operated by a local excavating contractor. The field work for this present investigation was carried out in conjunction with our previous hydrogeological investigation and terrain analysis for the site the results of which are reported in the Kollaard Associates Report No. 070415 dated October 25, 2007

The test pits put down during the subsurface investigation were for geotechnical and terrain analysis purposes only. Identification of the presence or absence of surface or subsurface contamination was outside the scope of work for the investigation. As such, an environmental technician was not on site for environmental sampling or assessment purposes.

The test pits were advanced to depths of about 0.2 to about 1.8 metres below the existing ground surface. The subsurface conditions encountered at the test pits were classified based on visual and tactile examination of the samples recovered and of the materials exposed on the sides and bottom of the test pits (ASTM D2488 - Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).

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 fieldwork was supervised throughout by a member of our engineering staff who directed the test pitting operations, cared for the samples obtained and logged the test pits.

Three samples (TP5 0.23 to 1.35, TP9 0.25 to 0.71, TP10 (0.2 to 1.07) were submitted for sieve analysis LS-602 to verify the grain size distribution and classification of the native soils at the site.

A detailed account of the subsurface conditions encountered at each of the test pits is provided in the attached Table I, Record of Test Pits following the text of this report. The approximate locations of the test pits are shown on the attached Site Plan, Figure 2.

4 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, the soil and groundwater conditions encountered at the test pits put down for this investigation are given in the attached Table I, Record of Test Pits following the text of this report. The test pit logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at other than the test pit locations may vary from the conditions encountered at the test pits. In addition to soil and bedrock variability, fill of variable physical and chemical composition may be present over portions of the site.

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 date of observations noted in the report and on the test pit logs. Groundwater conditions may vary seasonally, or may be affected by construction activities on or in the vicinity of the site.

The subsurface conditions encountered at the test pit locations are indicated to consist, in general, of topsoil followed by a layer of fine to medium sand and/or silty sand glacial till, then bedrock.

There are no sensitive marine clay deposits present at the site or other subsurface geotechnical conditions that would preclude normal residential construction. The subsurface soils encountered are not considered to be sensitive to fluctuating groundwater levels at the thickness and consistency / relative density present at the site.

4.2 Topsoil

About a 0.2 to 0.4 metre thick layer of topsoil was encountered from the ground surface at all of the test pit locations. The surface soil layer was classified as topsoil based on colour and the presence of organic materials and is intended for geotechnical description purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustaining plant growth.

4.3 Sand/Silty Sand

About a 0.4 to 1.5 metres thickness of grey brown sand/silty sand was encountered beneath the topsoil at test pits TP2 and TP3 and Test pits TP7 to TP12. Based on the difficulty of advancement of the test pits within the sand/silty sand, the sand/silty sand is indicated to be in a compact to dense state of packing. The sand was fully penetrated at all of the test pit locations where it was encountered.

The grain size distribution analysis for samples recovered from test pits TP9 and TP10 indicate that: The silty sand is in general fine to medium grained with trace quantities of gravel and some silt/clay particle sizes; The sand is general fine to medium grained with trace silty/clay particle sizes.

4.4 Silty Clay

A deposit of silty clay was encountered beneath the topsoil at test pit TP14, and beneath the sand/silty sand at test pits TP3 and TP7. The silty clay has been weathered to a grey brown crust. Based on visual and tactile examination of the silty clay exposed on the sides and bottom of the test pits, the silty clay encountered at the test pit locations is considered to be stiff to very stiff in consistency. Based on the blocky structure and difficulty to mould, the silty clay was considered to have a relatively low moisture content. Test pit TP3 was terminated within the silty clay at a depth of about 1.2 metres below the existing ground surface. The silty clay was fully penetrated at Test pits TP7 and TP14 at depths of about 1.2 to 1.4 metres below the existing ground surface.

4.5 Glacial Till

Glacial till was encountered below the topsoil at test pits TP5, TP6, and TP13 at depths of about 0.2 to 0.3 metres below the existing ground surface, below the sand/silty sand at test pits TP10 and TP11 at depths of about 0.7 to 1.1 metres below the existing ground surface, and below the silty clay at test pit TP14 at about 1.2 metres below the existing ground surface. Based on the difficulty of advancement of the test pits within the glacial till, the glacial till is indicated to be in a compact to dense state of packing. Test pits TP6, TP13 and TP14 was terminated within the glacial till at depths of about 1.7 to 1.8 metres below the existing ground surface. The glacial till was fully penetrated, where encountered, at the remainder of the test pit locations.

The grain size distribution analysis for samples recovered from test pit TP5 confirm that the material consists of sand and gravel in a matrix of silt and clay and is correctly identified as glacial till.

4.6 Weathered Bedrock/Bedrock

Weathered bedrock and/or relatively sound bedrock was encountered at all of the test pit locations except test pits TP3, TP6, TP13 and TP14 at depths of about 0.2 to 2.0 metres below the existing ground surface.

4.7 Groundwater

Seepage was encountered into test pits TP5, TP6, TP8, TP10, TP13 and TP14 during excavating on July 31, 2007 at depths of about 1.3, 1.2, 1.6, 1.5, 0.6 and 0.8 metres below the existing ground surface, respectively. The remaining eight test pits were dry upon completion of excavating.

The water infiltration into the test pits was encountered either in very close proximity to the surface of the bedrock indicating that water is being perched immediately above the bedrock (TP5 and TP10) or it was encountered at or below an elevation of 74.70 metres within the low lying areas of the site.

Since no groundwater was encountered in above the bedrock in several of the test pits put down in lower lying areas of the site, it is expected that the groundwater level will be below the surface of the bedrock during years with less than normal amounts of precipitation.

It should be noted that the water may be encountered at higher levels during wet periods of the year such as the early spring or immediately following significant rain fall events. The elevated water level will be a function of the downward migration of surface water and will not represent an elevated groundwater table.

4.8 Corrosivity on Reinforcement and Sulphate Attack on Portland Cement

The results of the laboratory testing of a soil sample 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 %	< 0.0005	Negligible
pH	pH < 5.5	6.34	Negligible concern
Resistivity	R < 20,000 ohm-cm	16600	Mildly Corrosive
Sulphates (SO_4)	SO_4 > 0.1%	<0.0020	Negligible concern

The results of the laboratory testing of a soil sample for sulphate gave a percent sulphate of less than 0.0020. The National Research Council of Canada (NRC) recognizes four categories of potential sulphate attack of buried concrete based on percent sulphate in soil. From 0 to 0.10 percent the potential is negligible, from 0.10 to 0.20 percent the potential is mild but positive, from 0.20 to 0.50 percent the potential is considerable and 0.50 percent and greater the potential is severe. Based on the above, the soils are considered to have a negligible potential for sulphate attack on buried concrete materials and accordingly, conventional GU or MS Portland cement may be used in the construction of the proposed concrete elements.

The pH value for the soil sample was reported to be at 6.34, indicating a durable condition against corrosion. This value was evaluated using Table 2 of Building Research Establishment (BRE) Digest 362 (July 1991).The pH is greater than 5.5 indicating the concrete will not be exposed to attack from acids.

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

Corrosivity Rating for soils ranges from extremely corrosive with a resistivity rating <1000 ohm-cm to non-corrosive with a resistivity of >20,000 ohm-cm as follows:

Soil Resistivity (ohm-cm)	Corrosivity Rating
> 20,000	non- corrosive
10,000 to 20,000	mildly corrosive
5,000 to 10,000	moderately corrosive
3,000 to 5,000	corrosive
1,000 to 3,000	highly corrosive
< 1,000	extremely corrosive

The soil resistivity was found to be 16600 ohm-cm for the sample analyzed making the soil mildly corrosive for buried steel. Increasing the specified strength and increasing concrete cover or increasing the specified strength and adding air entrainment into any reinforced concrete in contact with the soil is recommended. Additional special protection, other than listed above, is not required for reinforcement steel within the concrete foundation walls.

Based on the chemical test results, Type GU General Use Hydraulic Cement may be used for this proposed development. Special protection in the form of air entrainment and minimum cover is required for reinforcement steel within the concrete walls.

The laboratory results are presented in the Attachments following the text of this report.

4.9 Permeability of Native Soils along Proposed Swales

Permeability testing was completed on the native glacial till materials along the north and south sides of the proposed development and within the area of the proposed stormwater storage swale. The test results are included in the attachments following the text of this report. The test results indicate that the permeability k for the native soils at the site at the north and south sides of the site ranges from 1.26×10^{-6} m/s to 9.44×10^{-7} m/s. The permeability of the native soils in the area of the proposed stormwater storage swale was determined to be 1.64×10^{-7} m/s. It is noted that the permeability of a soil and the hydraulic conductivity of water in the same soil have the same value.

The following table obtained from Appendix C of the CVC LID guide indicates the relationship between the Percolation Time, Coefficient of Permeability and Infiltration Rate.

Table C1: Approximate relationships between hydraulic conductivity, percolation time and infiltration rate

Hydraulic Conductivity, K_{fs} (centimetres/second)	Percolation Time, T (minutes/centimetre)	Infiltration Rate, 1/T (millimetres/hour)
0.1	2	300
0.01	4	150
0.001	8	75
0.0001	12	50
0.00001	20	30
0.000001	50	12

Source: Ontario Ministry of Municipal Affairs and Housing (OMMAH). 1997. Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions. Toronto, Ontario.

From the above comparison, the native soils at the site would have an estimated infiltration rate of 30 to 50 mm/hr and a Percolation Time T of 12 to 20 minutes.

5 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

5.1 General

This section of the report provides engineering guidelines on the geotechnical aspects of the project based on our interpretation of the test pit information and the project requirements. It is stressed that the information in the following sections is provided for the guidance of the designers for the design of the project 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 retained 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 from materials from off site sources are outside the terms of reference for this report and have not been investigated or addressed.

5.2 Foundations for the Proposed Single Family Dwellings

With the exception of the topsoil, the subsurface conditions encountered at the test pits advanced during the investigation are suitable for the support of the proposed single family dwellings on conventional spread footing foundations. It is noted that fill has been placed on the site at several locations since the date at which test pits were advanced on the site. These fill materials are also not considered suitable for the support of the proposed foundations. The excavations for the foundations should be taken down through any topsoil or otherwise deleterious material to expose the native, undisturbed sand/silty sand, silty clay, glacial till, or bedrock.

It is expected that the excavations to remove the topsoil, fill and any other deleterious material will likely result in an approved subgrade level below the founding elevation for the majority of the development. Where this occurs, the subgrade will have to be raised using engineered fill as discussed in more detail in the following sections.

5.2.1 Allowable Bearing Capacity and Grade Raise Restrictions

The allowable bearing pressure for any footings depends on the depth of the footings below original ground surface, the width of the footings, and the height above the original ground surface of any landscape grade raise adjacent to the foundations and the thickness of the soils deposit beneath the footings.

For the proposed single family dwellings founded in the sand/silty sand, silty clay or glacial till, a geotechnical reaction at serviceability limit state (SLS) of 100 kilopascals and a factored geotechncial resistance at ultimate limit state (ULS) of 300 kilopascals could be used for the design of conventional strip or pad footings a minimum of 0.5 metres in width. The exposed subgrade surface should be inspected and approved by a qualified geotechnical person prior to the placement of any engineered fill or foundation installation.

For the proposed single family dwellings founded all on the weathered bedrock, relatively sound bedrock or engineered fill placed directly over the bedrock, a geotechnical reaction at serviceability limit state (SLS) of 150 kilopascals and a factored geotechncial resistance at ultimate limit state (ULS) of 450 kilopascals could be used for the design of both conventional strip and pad footings.

Provided that any loose and disturbed soil is removed from the bearing surfaces prior to placement of engineered fill or pouring concrete the total and differential footing settlements are expected to be less than 25 and 20 millimetres, respectively, using the above allowable resistances.

To minimize the potential for foundation cracking where footings will be founded on both overburden materials and bedrock, it is suggested that the foundations walls in the transition zone be suitably reinforced. Suggested foundation treatment for overburden/bedrock transition areas are provided in the attached Figure 3.

The above bearing pressures are suitable for strip and pad footings up to 1.5 metres in width and 2.5 metres square, respectively. Due to their limited thickness and stiff to very stiff consistency or compact to dense state of compaction, the soils at the site are not present considered to be significantly susceptible to consolidation under the loading expected for the development. As such there are no grade raise restrictions related to the above allowable bearing pressures.

5.2.2 Engineered Fill

It is expected that the removal of topsoil and deleterious material will likely result in an approved subgrade level below the proposed founding elevation of a majority of the proposed dwellings. Where this occurs, the subgrade could be raised to the proposed founding level using suitable imported engineered fill. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular A or Granular B Type II and should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. To allow the spread of load beneath the footings, the engineered fill should extend at least 0.5 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter. The excavations for the foundation 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 founding level be composed of virgin material only.

Any engineered fill materials provided to support the concrete basement floor slabs should consist of sand, or sand and gravel meeting the Ontario Provincial Standards Specifications (OPSS) for Granular B Type I or crushed stone meeting OPSS grading requirements for Granular B Type II. A minimum 150 millimetre thickness of crushed stone meeting OPSS Granular A should be provided immediately beneath the concrete floor slab. The engineered fill materials should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density.

5.2.3 Foundation Excavations

Any excavation for the proposed structures will be carried out through topsoil, fill or any otherwise deleterious material to expose the underlying native sand/silty sand, silty clay, glacial till or bedrock. The sides of the excavations should be sloped in accordance with the requirements of Ontario Regulation 213/91, s. 226 under the Occupational Health and Safety Act. According to the Act, the native soils at the site can be classified as Type 3 soil, however this classification should be confirmed by qualified individuals as the site is excavated and if necessary, adjusted.

Bedrock was encountered at relatively shallow depths at most of the test pits. However, most of the foundations are expected to be founded at or near the existing ground surface elevation at the dwelling location. As such, it is expected that significant bedrock removal will likely not be required. Small amounts of bedrock removal, if required, can most likely be carried out by hoe ramming. It is recommended that pre-construction condition surveys of nearby structures and existing utilities are completed before and bedrock removal.

5.2.4 Ground Water in Excavation and Construction Dewatering

Groundwater was encountered within the test pits put down within the east portion of the site, occupied by the tailwater section of the Harwood Creek Flood Plain, at depths of between 0.6 and 1.6 metres below the existing ground surface. The based on the proposed site grading and drainage plan Drawing No. 200977-GRD prepared by Kollaard Associates Inc, the proposed underside of footing elevation for the dwellings in this area are at or above the existing ground surface. As such it is considered unlikely that excavations for the proposed foundations will encounter significant groundwater. As such a permit to take water is will not be required prior to excavation.

Groundwater and surface water inflow into the excavations during construction, if any should be handled by pumping from sumps within the excavation.

5.2.5 Effect of Dewatering of Foundation Excavations

Since the existing groundwater level at the site will be below the expected underside of footing elevations, dewatering of the excavation will not remove water from historically saturated soils. As such dewatering of the foundations or excavations, if required, will not have a detrimental impact on any adjacent structures.

5.2.6 Effect of Lowering GWL on Grade Raise, Settlement and Consolidation

The colour of the soils encountered within the test pits indicate that oxidation is ongoing within the soils encountered at the site. As such, all of the soils encountered at the site are above the normal groundwater level and are above any long term low groundwater level (GWL).

The water infiltration into the test pits was encountered either in very close proximity to the surface of the bedrock indicating that water is being perched immediately above the bedrock or it was encountered below an elevation of 74.70 metres within the low lying areas of the site. As such, all of the development will be above the levels at which water was encountered in proximity to the development locations.

Consolidation and potential ground settlement due to fluctuating groundwater levels are not a legitimate possibility at the site given the existing subsurface conditions. Therefore no further discussion with respect to this concern is merited in this report.

5.2.7 Frost Protection Requirements for Spread Footing Foundations

In general, all exterior foundation elements and those in any unheated parts of the proposed buildings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated foundation elements adjacent to surfaces, which are cleared of snow cover during winter months should be provided with a minimum 1.8 metres of earth cover for frost protection purposes.

5.2.8 Foundation Wall Backfill

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

Backfilling should be completed in accordance with Part 9 of the Ontario Building Code. It is noted that backfill of the foundation should not commence until the ground level floor system has been installed unless the foundation has been structurally reinforced as an unsupported wall system.

Where the backfill material will ultimately support a pavement structure or walkway, it is suggested that the foundation wall backfill material be compacted in 250 millimetre thick lifts to 95 percent of the standard Proctor dry density value. In that case any native material proposed for foundation backfill should be inspected and approved by the geotechnical engineer.

5.2.9 Foundation Drainage

The foundation should be covered with a drainage layer as specified by the Ontario Building Code. A conventional, perforated perimeter drain, with a 150 millimetre surround of 20 millimetre minus crushed stone, should be provided at the founding level for the cast-in-place concrete basement floor slab and should lead by gravity flow to a sump/sump pump. The sump pit should be equipped with an emergency backup pump. The sump discharge should be equipped with a backup flow protector. The sump should discharge to the ground surface within the limits of the lot. The sump pump and sump pump discharge should be in keeping with Ottawa Sewer Design Guidelines Section (ISTB 2018-04). Section 5.12.2.1, 5.12.2.2, 5.12.2.3 sentences 1-9, 5.12.2.7.

5.2.10 Basement Floor Slab Support

As stated above, it is expected that the proposed residential buildings will be founded on native subgrade or on an engineered pad placed on the native subgrade. For predictable performance of the proposed concrete basement floor slab all existing fill material, topsoil and any otherwise deleterious material should be removed from below the proposed floor slab area. The exposed native subgrade surface should then be inspected and approved by geotechnical personnel. Any soft areas evident should be subexcavated and replaced with suitable engineered fill. Any fill materials consisting of granular material, removed from the proposed concrete floor slab area, could be stockpiled for possible reuse with approval from the geotechnical engineer.

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

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

The slab should be structurally independent from walls and columns, which are supported by the foundations. This is to reduce any structural distress that may occur as a result of differential soil movement. If it is intended to place any internal non-load bearing partitions directly on the slab-on-grade, such walls should also be structurally independent from other elements of the building founded on the conventional foundation system so that some relative vertical movement between the floor slab and foundation can occur freely.

The concrete floor slab should be saw cut at regular intervals to minimize random cracking of the slab due to shrinkage of the concrete. The saw cut depth should be about one quarter of the thickness of the slab. The crack control cuts should be placed at a grid spacing not exceeding the lesser of 25 times the slab thickness or 4.5 metres. The slab should be cut as soon as it is possible to work on the slab without damaging the surface of the slab.

5.3 Stormwater Management Weir Wall

The outlet control structure for the primary storage in the stormwater storage swale will consist of V-notch weirs. This weir will be formed in a cast in place concrete weir wall. The weir wall should be placed on a minimum 0.4 metre wide footing founded 1.5 metres below the invert of the swale bottom. The weir wall may be designed for a geotechnical reaction at serviceability limit state (SLS) of 100 kilopascals and a factored geotechncial resistance at ultimate limit state (ULS) of 300 kilopascals. The exposed subgrade surface should be inspected and approved by a qualified geotechnical person prior to the placement of the footing. Dewatering if required can be completed as described above.

5.4 Noise Barrier Foundation

The Environmental Noise Impact Assessment requires that a 2.0 metre tall noise barrier fence be erected along the rear lot line of Lots 3, 5 and 7. The noise barrier fence is to have a minimum density of 20 kg/m^2. A review of common commercially available noise barriers meeting these requirements indicates that these barriers are typically supported by posts embedded in a concrete foundation. The concrete foundation typically consists of cast in place concrete caissons

Due to relatively shallow depth to bedrock, it is considered that there will be insufficient embedment depth above the bedrock to provide lateral support for the caissons without the caissons becoming excessively large. It is recommended that the foundation system consist of a pad and pier foundation system. The concrete pads should be cast in place and bear on the sound bedrock surface. When bearing on the sound bedrock surface, the foundation footings may be designed for a geotechnical reaction at serviceability limit state (SLS) of 500 kilopascals and a factored geotechncial resistance at ultimate limit state (ULS) of 1000 kilopascals.

If caissons are used instead of a pad and pier foundation systems, the caissons may be designed assuming a lateral geotechnical resistance resist, P, resisting the overturning of the caisson at any depth, h, calculated using the following equation.

$$\mathbf{P = k_p \gamma h}$$

Where:	P	=	the pressure, at any depth, h, below the finished ground surface (kN/m^2)
	K_p	=	passive earth pressure coefficient, equal to 3
	γ	=	native unit weight of soil adjacent caisson, estimated at 19 kN/m^3
	h	=	the depth, in metres, below the finished ground surface at which the pressure, P, is being computed

The area of resistance is assumed to be equal to the diameter of the caisson should the caisson be direct bury or equal to the diameter of the granular material compacted around the caisson or pier if granular backfill is used and properly compacted.

The pads and piers should be backfilled with a non frost susceptible granular material such as OPSS Granular B Type 1 or Type II to 0.3 metres from the ground surface. The granular material should be compacted to a minimum of 95% standard proctor maximum dry density. The sound bedrock surface is considered to be non frost susceptible and no additional frost protection will be required.

5.5 Swimming Pools

In-ground and above-ground swimming pools can be constructed on the Lots; provided the following precautions are respected. In addition to the below precautions; no swimming pool of any kind shall be constructed within the flood plain or flood plain setback of the Harwood Creek.

Furthermore, swimming pool construction is not recommended to be constructed in any other easement/setback as outlined by other consultants.

5.5.1 In-ground Swimming Pools

The installation of an in-ground swimming pool will result in a negligible net gain of any increased loading to the site's underlying soil conditions. It is recommended that a site specific geotechnical investigation be completed before the installation of an in-ground pool to determine the depth to the underlying bedrock. Small amounts of bedrock can be removed, if required, by hoe ramming techniques. Since significant hoe ramming immediately adjacent to a structure can negatively impact the structure, an alternative pool structure or pool location should be considered if significant quantities of shallow bedrock are encountered. It is not recommended to stockpile any excavated material within 3 meters of any building.

Since there are no grade raise restrictions, any site re-grading to accommodate the pool will have no impact on the underlying soil conditions.

5.5.2 Above-ground Pools

The addition of an above-ground pool will result in a net gain of loading imposed on the site's underlying soils due to the weight of water above-ground surface. Since there are no grade raise restrictions, the depth of the above-ground pool is not limited by the underlying soil conditions. It is recommended to install above-ground pools a minimum of 2.5 m from the foundation of the dwelling in order to avoid any lateral loading on the foundation from the above-ground pool.

Any site re-grading to accommodate the pool will have no impact on the underlying soil conditions.

5.6 Slope Stability Considerations

The City of Ottawa Slope Stability guidelines provide the following requirement: A report addressing the stability of slopes, prepared by a qualified geotechnical engineer licensed in the Province of Ontario, should be provided wherever a site has slopes (existing or proposed) steeper than 5 horizontal to 1 vertical (i.e., 11 degree inclination from horizontal) and/or more than 2 metres in height.

5.6.1 Existing Slopes

There are no existing slopes on the site that are steeper than 5 horizontal to 1 vertical and exceed a height of 1 metre. There are no existing slopes on site that exceed a height of 2 metres. As such, there are no existing slopes on the site that could be considered unstable.

5.6.2 Proposed Slopes

5.6.2.1 Roadside Ditches and Swales

The proposed slopes of the roadside ditches have been designed to be in keeping with the City of Ottawa Standard details for Roads. The ditch side slopes are limited in height to much less than 1.3 metres. As such, there are no stability concerns with respect to the roadside ditch slopes. The swale side slopes are designed with side slopes not exceeding 3 horizontal to 1 vertical and a height not exceeding 1 metre. The side slopes of the swales should consist of native soils or fill that has been at least moderately well compacted using heavy construction equipment during placement. The slopes of both the roadside ditches and swales should be protected with vegetation as soon as possible. The proposed roadside ditch and swale slopes are not considered to be at risk of being unstable.

5.6.2.2 Slopes from Proposed Onsite Septic Systems

The slopes from the proposed septic system will not exceed 4 horizontal to 1 vertical in keeping with the specifications in Part 8 of the Ontario Building Code. The slopes from the onsite septic systems will be less than 1.5 metres in total height. The slopes from the proposed septic systems are not considered to be at risk of being unstable.

5.6.2.3 Rear Yard Slopes to the existing undisturbed grade

The rear yard slopes from the landscaped area adjacent to the proposed dwellings to the undisturbed grade at the back of each lot are designed with a maximum slope of 5 horizontal to 1 vertical. The maximum height of these slopes is less than 2 metres. The rear yard slopes are not considered to be at risk of being unstable.

5.7 Seismic Design for the Proposed Residential Buildings

5.7.1 Seismic Site Classification

Based on the information obtained from the test pits, The subsurface conditions consist of a thin layer of overburden having in general a thickness of less than 3 metres followed by bedrock. Based on these subsurface conditions, 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 for foundation design purposes can be assumed to be Site Class C.

5.7.2 National Building Code Seismic Hazard Calculation

The online 2015 National Building Code Seismic Hazard Calculation was used to verify the seismic conditions at the site. The design Peak Ground Acceleration (PGA) for the site was calculated as 0.181 with a 2% probability of exceedance in 50 years based on the interpolation of the 2015 National Building Code Seismic Hazard calculation. The seismic site classification for the site is indicated to be Seismic Site Class C. The results of the calculation are attached following the text of this report.

5.7.3 Potential for Soil Liquefaction

As indicated above, the results of the test pits indicate that the native deposits within the area of the proposed residential subdivision consist of compact to dense silty sand/sand, stiff silty clay, compact to dense glacial till and bedrock. Accordingly there is no potential for liquefaction of the native subgrade under seismic conditions.

5.8 Site Services

As stated previously the proposed residential subdivision will be serviced with private drilled wells and septic systems. In addition, storm water runoff is being managed with surface flow. As such no significant excavations for services are expected. However, any excavation for the installation of such services as gas, telephone, hydro etc. should be backfilled in a manner compatible with the future use of the area above the service excavation.

If excavations extend below the water table in silty sand or sandy soil, some loss of ground and groundwater inflow may occur, requiring flatter side slopes to be used. Cobbles and boulders, some of which could be large may exist within the glacial till. As noted above, bedrock was encountered at the site at relatively shallow depths, as such excavating through weathered bedrock/bedrock may be require for the installation of the services and can be completed as outlined above.

In areas where the service trench will be located below or in close proximity to the proposed roadways or driveways, acceptable native materials should be used as backfill between the roadway subgrade level and the lesser of the depth of excavation or the depth of seasonal frost penetrations (i.e. 1.8 metres below finished grade) in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent section of roadway. Where native 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. Any boulders larger than 300 millimetres in size should not be used as service trench backfill. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I.

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

5.9 Roadways

5.9.1 Subgrade Preparation

In preparation for roadway construction, the topsoil, fill and any soft, wet or deleterious material should be removed from the roadway area. 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 roadway granulars may be placed.

Fill sections along the proposed roadway should be brought up to proposed roadway subgrade level using acceptable earth borrow material or granular material consisting of OPSS select subgrade material or OPSS Granular B Type I or Type II. The earth borrow should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment. Any of the native materials proposed for this use should be approved by the geotechnical engineer before placement within the roadway.
The subgrade surface 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.

5.9.2 Pavement Structure

It is suggested that provision be made for the following minimum pavement structure for local residential roadways:

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

Non-woven geotextile fabric (6oz/sqy) such as Soleno TX-110 or Thrace-Ling 150EX or approved alternative.

Performance grade PG 58-34 asphaltic concrete should be specified. The pavement granular materials should be compacted in maximum 300 millimetre thick lifts to at least 100 percent of standard Proctor maximum dry density using suitable vibratory compaction equipment.

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

The above pavement structure assumes that the trench backfill is adequately compacted and that the roadway subgrade surface is prepared as described in this report. If the roadway subgrade surface 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 to incorporate a non-woven geotextile separator between the roadway subgrade surface and the granular subbase material. The adequacy of the design pavement thickness should be assessed by geotechnical personnel at the time of construction.

5.10 TREES

The upper soils at the site consist of compact to dense silty sand/sand, stiff silty clay, compact to dense glacial till and bedrock. As previously indicated, the silty clays encountered were not considered to consist of sensitive marine deposited silty clays due to their consistency and relatively low moisture content. In addition, the thickness of the silty clay deposits, where fully penetrated ranged from about 0.15 to 0.9 metres.

Where silty clay soils are encountered at a proposed building location, in keeping with the City of Ottawa, Tree Planting in Sensitive Marine Clay Soils - 2017 Guidelines small and medium sized trees can be planted as close as 4.5 metres from the proposed dwelling provided sufficient soil volume is available around the proposed tree location (a minimum of 25 m^3 for small trees and 30 m^3 for medium trees must be available in the upper 1.5 metres below finished grade).

Where silty clay is present at a proposed building location and where the thickness of the silty clay deposit exceeds 0.4 metres, large trees should be planted no closer than 15 metres from the proposed building

Excluding the areas where the silty clay deposits exceed 0.4 metres, the remainder of the subsurface soils encountered at the site are not considered particularly sensitive to depletion of moisture by trees. There are no planting restrictions from a geotechnical perspective for small and medium trees with respect to planting distance from the proposed buildings. Large trees should be planted no closer than 10 metres from a proposed dwelling where no silty clay is present on the lot.

Tree planting guidelines provided by a landscape architect, arborist, urban forest manager or other qualified professional with respect to species, distance to building requirements, moisture requirements etc should be obtained and followed in addition to the geotechnical recommendations.

6 CONSTRUCTION OBSERVATIONS

It is suggested that the final design drawings for the site, 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 engineer 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.

Any native or imported earth borrow material proposed to be used as engineered fill below the pavement areas should be approved by Kollaard Associates Inc. prior to use.

All footing areas and any engineered fill areas for the proposed dwellings 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 foundation should be inspected and in situ density testing should be carried out to ensure that the materials used meet 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 trench backfill where the service trench will be located below or in close proximity to the proposed roadways or driveways, and on the pavement granular materials to ensure the materials meet the specifications from a compaction point of view.

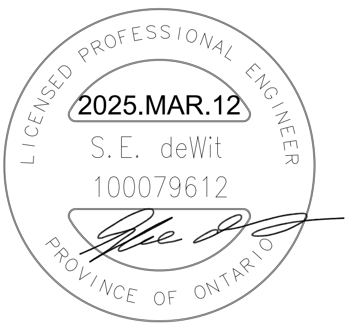
Any blasting should be carried out under the supervision of a blasting specialist engineer. Pre-blast condition surveys of nearby structures and existing utilities are essential. Monitoring of the blasting should be carried out throughout the blasting period to ensure that the blasting meets the limiting vibration criteria established by the specialist engineer.

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 any further service to you, please do not hesitate to contact our office.

Sincerely,
Kollaard Associates Inc.

Written by:



Steven deWit, P. Eng.

RECORD OF TEST PIT SHEETS

TABLE I

RECORD OF TEST PITS
PART 1, PLAN 5R - 10284
WEST CARLETON WARD
CITY OF OTTAWA, ONTARIO

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP1	0.00 – 0.15	TOPSOIL
	0.15	Refusal, BEDROCK

Test pit dry, July 31, 2007.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP2	0.00 – 0.18	TOPSOIL
	0.18 – 0.46	Grey brown, silty sand, trace clay, some gravel, weathered bedrock (GLACIAL TILL)
	0.46 – 0.71	Weathered BEDROCK
	0.71	Refusal, BEDROCK

Test pit dry, July 31, 2007.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP3	0.00 – 0.38	TOPSOIL
	0.38 – 0.84	Grey brown SILTY SAND, some gravel
	0.84 – 1.17	Grey brown SILTY CLAY
	1.17	End of test pit

Test pit dry, July 31, 2007.

TABLE I (CONTINUED)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP4	0.00 – 0.18	TOPSOIL
	0.18 – 0.79	BOULDERS and weathered BEDROCK
	0.79	Refusal, BEDROCK

Test pit dry, July 31, 2007.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP5	0.00 – 0.23	TOPSOIL
	0.23 – 1.35	Grey brown silty sand, trace clay, gravel, cobbles (GLACIAL TILL)
	1.35	Refusal, BEDROCK

Water observed in test pit at about 1.3 metres below existing ground surface, July 31, 2007.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP6	0.00 – 0.30	TOPSOIL
	0.30 – 1.83	Grey brown silty sand, trace clay, gravel, cobbles (GLACIAL TILL)
	1.83	End of test pit

Water observed in test pit at about 1.2 metres below existing ground surface, July 31, 2007.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP7	0.00 – 0.30	TOPSOIL
	0.30 – 1.02	Grey brown SILTY SAND, trace clay
	1.02 – 1.22	Red brown SILTY SAND
	1.22 – 1.37	Grey brown SILTY CLAY
	1.37	Refusal, BEDROCK

Test pit dry, July 31, 2007.

TABLE I (CONTINUED)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP8	0.00 – 0.18	TOPSOIL
	0.18 – 0.51	Grey brown SILTY SAND, some gravel
	0.51 – 1.98	Grey brown to grey fine to medium SAND
	1.98	Refusal, BEDROCK

Water observed in test pit at about 1.6 metres below existing ground surface, July 31, 2007.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP9	0.00 – 0.25	TOPSOIL
	0.25 – 0.71	Grey brown fine to medium SAND, some silt
	0.71 – 1.45	Grey brown SILTY SAND, gravel, cobbles, boulders
	1.45	Refusal, BEDROCK

Test pit dry, July 31, 2007.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP10	0.00 – 0.20	TOPSOIL
	0.20 – 1.07	Grey brown fine to medium SAND
	1.07 – 1.65	Grey brown silty sand, trace clay, gravel, cobbles (GLACIAL TILL)
	1.65	Refusal, BEDROCK or large boulder

Water observed in test pit at about 1.5 metres below existing ground surface, July 31, 2007.

TABLE I (CONTINUED)

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP11	0.00 – 0.30	TOPSOIL
	0.30 – 0.74	Red brown to grey brown fine to medium SAND, some gravel and cobbles
	0.74 – 1.04	Grey silty sand, trace clay, gravel, cobbles, boulders (GLACIAL TILL)
	1.04	Refusal, BEDROCK

Test pit dry, July 31, 2007.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP12	0.00 – 0.20	TOPSOIL
	0.20 – 0.51	Grey brown SILTY SAND, gravel, cobbles
	0.51	Refusal, BEDROCK

Test pit dry, July 31, 2007.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP13	0.00 – 0.23	TOPSOIL
	0.23 – 1.68	Grey brown silty sand, trace clay, some gravel, cobbles, boulders (GLACIAL TILL)
	1.68	End of Test Pit

Water observed in test pit at about 0.6 metres below existing ground surface, July 31, 2007.

TEST PIT NUMBER	DEPTH (METRES)	DESCRIPTION
TP14	0.00 – 0.30	TOPSOIL
	0.30 – 1.22	Grey brown SILTY CLAY
	1.22 – 1.83	Grey brown to grey silty sand, trace clay, gravel, cobbles, boulders (GLACIAL TILL)
	1.83	End of test pit

Water observed in test pit at about 0.8 metres below existing ground surface, July 31, 2007.

LIST OF FIGURES

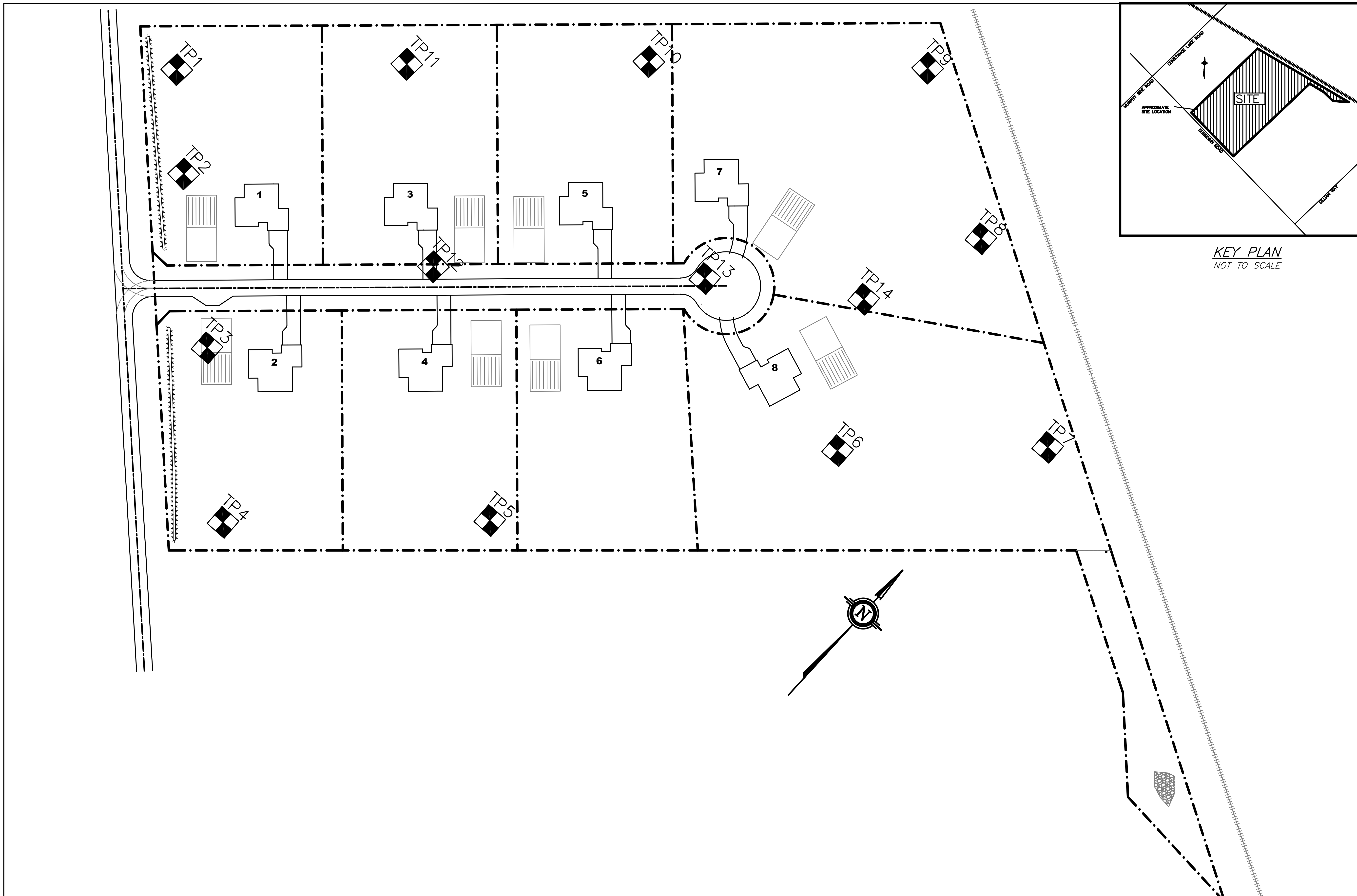
KEY PLAN	FIGURE 1

NOT TO SCALE

Kollaard Associates
Engineers

Project No. 200977

Date November 12, 2021



NOTE:

1. All dimensions are in metres.
2. All elevations are in metres and are based on a geodetic benchmark. TBM = Bell kiosk located sout/westh side of Dunrobin Road, across from proposed lot #2 , elevation = 79.82 m (geodetic)
3. This drawing does not represent a legal survey.
4. Finished grade to slope away from proposed building at a minimum of 2%. Grade elevations are as indicated.
5. All dimensions to be verified on site by contractor prior to construction.
6. All materials and construction methods to be in accordance with City of Ottawa Standards and Ontario Provincial Standards and Specifications.
7. All disturbed areas to be reinstated to the satisfaction of the engineer and the City of Ottawa.
8. The owner (and/or Contractor) agrees to prepare and implement an erosion and sediment control plan at least equal to the stated minimum requirements and to the satisfaction of the City of Ottawa, appropriate to the site conditions, prior to undertaking any site alterations (filling, grading, removal of vegetation, etc.) and during all phases of site preparation and construction in accordance with the current Best Management Practices for Erosion and Sediment Control
9. Any changes made to this plan must be verified and approved by Kollaard Associates Inc.

No.	REVISION	DATE	BY
0	ISSUED FOR SUBDIVISION APPROVAL	12.NOV2021	SD

Kollaard Associates
Engineers

BOX 189
210 PRESCOTT STREET
KEMPTVILLE, ONTARIO
K0G 1J0
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LICENCED PROFESSIONAL ENGINEER
12.NOV.2021
S.E. deWit
100079612
PROVINCE OF ONTARIO

DESIGN	SD
CHECKED	SD
DRAWN	SD
CHECKED	SD
APPROVED	SD

SCALE

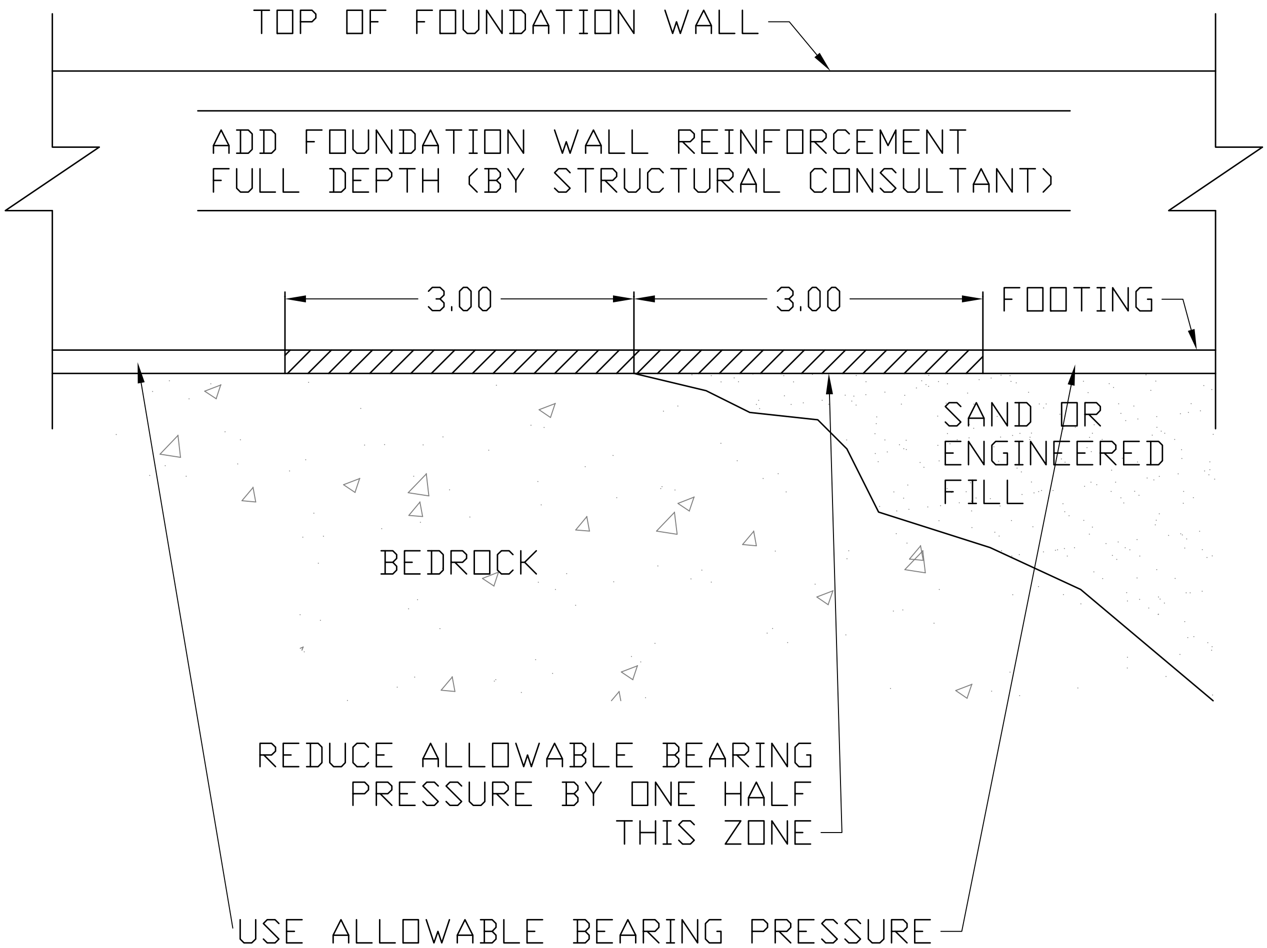
1: 750

0 5 10 20 30

PROJECT LOCATION	2050 DUNROBIN ROAD, CITY OF OTTAWA, ONTARIO
CLIENT NAME	HAUDEROWICZ, ZBIGNIEW AND TERESA
PROJECT NAME	PROPOSED RESIDENTIAL SUBDIVISION
DRAWING	SITE PLAN – FIGURE 2
PROJECT No.	200977
DRAWING No.	200977–SP
DATE	12–NOV–2021
SHEET SET	1 of 1

FOOTING BEDROCK - OVERBURDEN TRANSITION TREATMENT

FIGURE 3



NOT TO SCALE

THIS DRAWING TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL SUBDIVISION DEVELOPMENT 2050 DUNROBIN ROAD
PROJECT NO. 200977
DATE: NOVEMBER 2021

KOLLAARD ASSOCIATES INC.

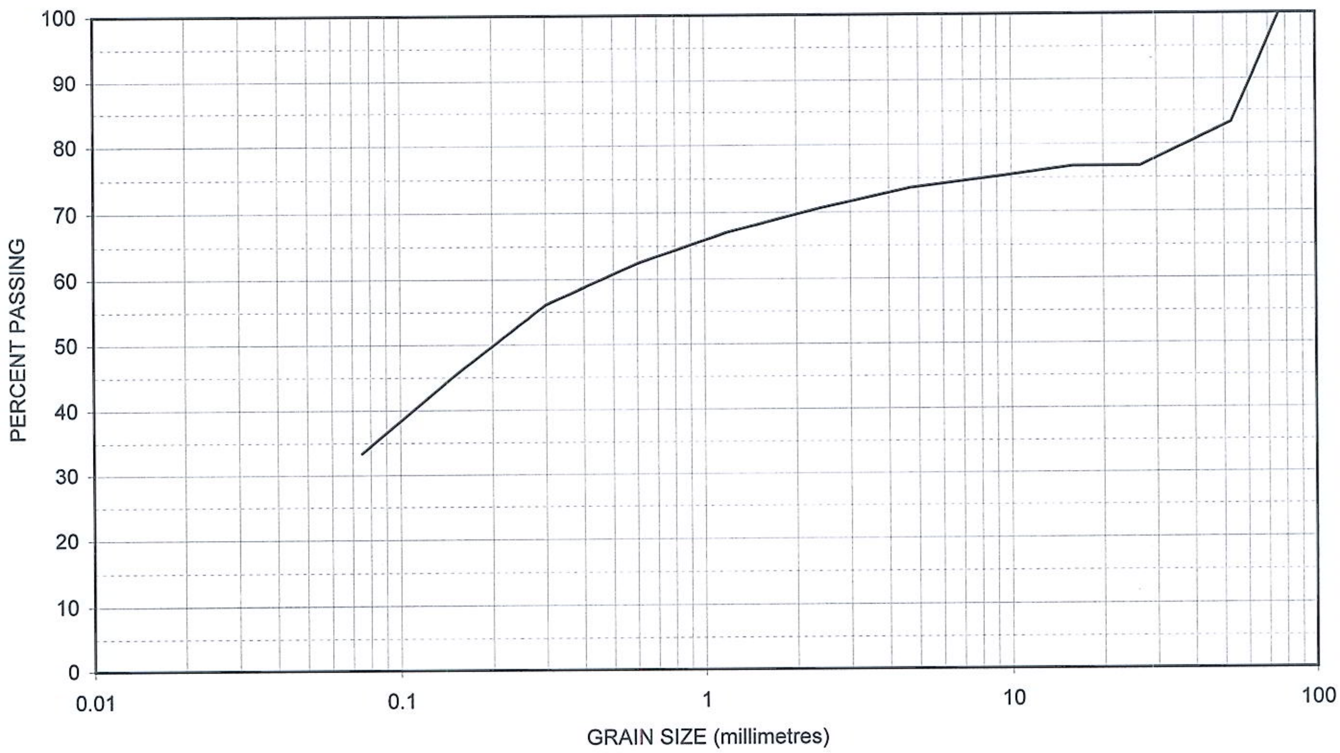
LIST OF ATTACHMENTS
Sieve Analysis Test Results
Laboratory Testing Results for Chemical Testing for Corrosivity
Permeability Test Results
National Building Code Seismic Hazard Calculation
Response to Geotechnical Review Comments

Sieve Analysis Test Results

Grain Size Distribution Analysis

Kollaard Associates
Engineers

Test Pit 5 (0.23 - 1.35)



SIEVE SIZE (mm)	76.2	53	26.5	19.0	16	13.2	9.5	4.75	2.36	1.180	0.600	0.300	0.15	0.075
SAMPLE PASSING	100.0	83.2	76.7	76.7	76.7	76.2	75.3	73.6	70.5	67.0	62.4	56.1	45.2	33.4

CLIENT: The Design Group

PROJECT: Constance Lake and Dunrobin Subdivision

OUR REF.: 070415

TYPE OF MATERIAL: silty sand glacial till

INTENDED USE: N/A

DATE SAMPLED: July 31, 2007

DATE TESTED: August 14, 2007

SOURCE: TP-5

SAMPLE NO: 1

REMARKS: Based on the above grain size distribution analysis and information published by the MOE relating grain size and percolation rate, it is estimated that the percolation rate for the sample is about 12-13 minutes per centimetre.

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Kemptville, Ontario K0G 1J0
(613) 860-0923, FAX: (613) 258-0475

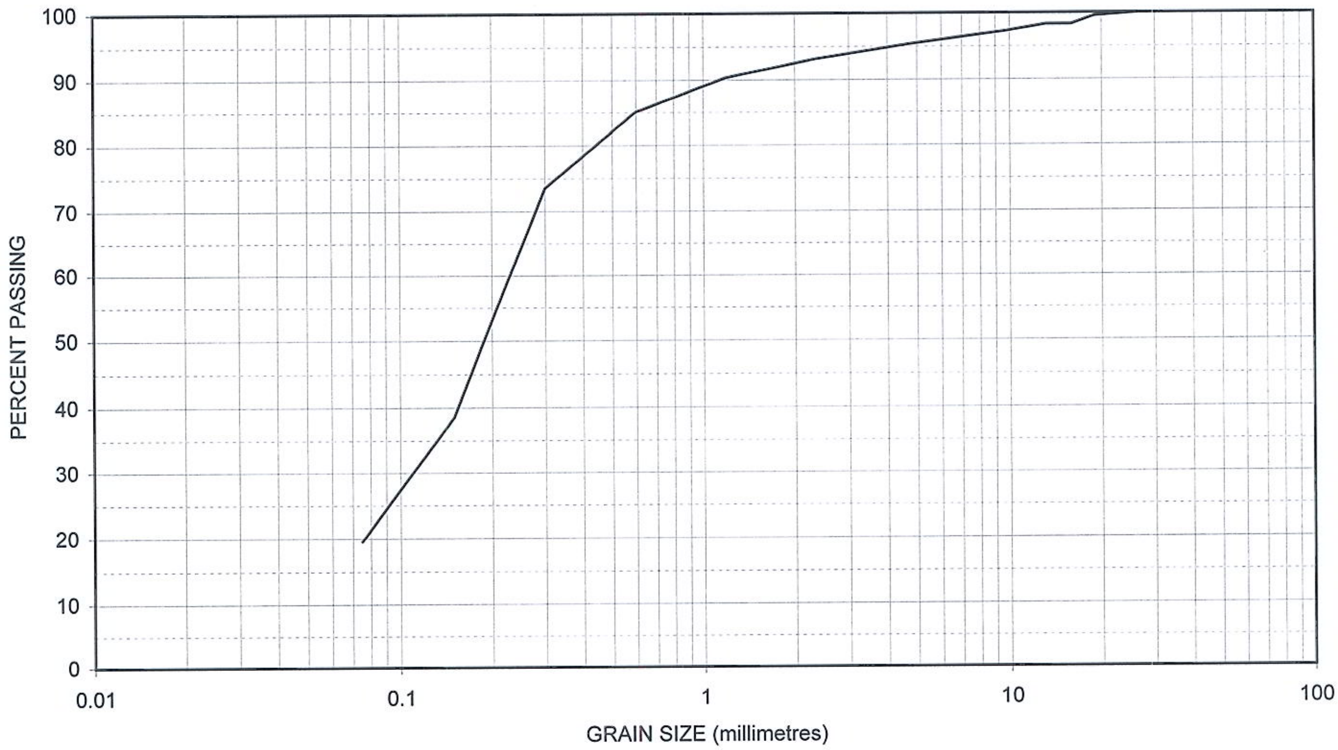
Issued by: C. R. Morey, P. Eng.

Date: October 12, 2007

Grain Size Distribution Analysis

Kollaard Associates Engineers

Test Pit 9 (0.25 - 0.71)



SIEVE SIZE (mm)	76.2	53	26.5	19.0	16	13.2	9.5	4.75	2.36	1.180	0.600	0.300	0.15	0.075
SAMPLE PASSING			100.0	99.5	98.3	98.3	97.2	95.4	93.2	90.3	85.1	73.5	38.5	19.5

CLIENT: The Design Group

PROJECT: Constance Lake and Dunrobin Subdivision

OUR REF.: 070415

TYPE OF MATERIAL: fine to medium sand, some silt

INTENDED USE: N/A

DATE SAMPLED: July 31, 2007

DATE TESTED: August 9, 2007

SOURCE: TP-9

SAMPLE NO: 2

REMARKS: Based on the above grain size distribution analysis and information published by the MOE relating grain size and percolation rate, it is estimated that the percolation rate for the sample is about 12-13 minutes per centimetre.

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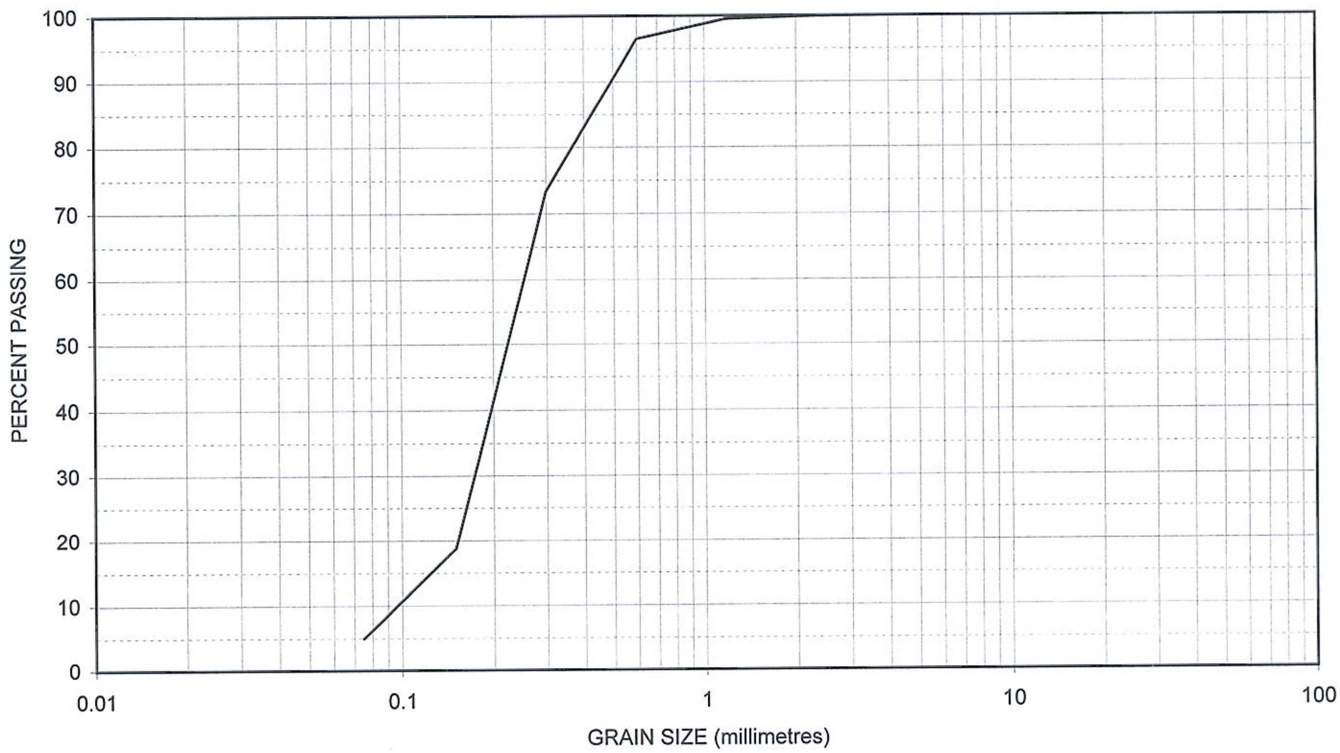
Issued by: C. R. Morey, P. Eng.

Date: October 12, 2007

Grain Size Distribution Analysis

Kollaard Associates Engineers

Test Pit 10 (0.20 - 1.07)



SIEVE SIZE (mm)	76.2	53	26.5	19.0	16	13.2	9.5	4.75	2.36	1.180	0.600	0.300	0.15	0.075
SAMPLE PASSING								100.0	99.9	99.5	96.5	73.3	18.7	5.0

CLIENT: The Design Group

PROJECT: Constance Lake and Dunrobin Subdivision — OUR REF.: 070415

TYPE OF MATERIAL: Fine to medium sand — INTENDED USE: N/A

DATE SAMPLED: July 31, 2007 — DATE TESTED: August 9, 2007

SOURCE: TP-10 — SAMPLE NO: 3

REMARKS: Based on the above grain size distribution analysis and information published by the MOE relating grain size and percolation rate, it is estimated that the percolation rate for the sample is about 12-13 minutes per centimetre.

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Engineers
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Kemptville, Ontario K0G 1J0
(613) 860-0923, FAX: (613) 258-0475

Issued by: C. R. Morey, P. Eng.

Date: October 12, 2007

Laboratory Testing Results for Chemical Testing for Corrosivity

CERTIFICATE OF ANALYSIS

Work Order	: **WT2305798**	Page	: 1 of 3
Client	: **Kollaard Associates Inc.**	Laboratory	: Waterloo - Environmental
Contact	: Dean Tataryn	Account Manager	: Costas Farassoglou
Address	: 210 Prescott Street Unit 1 Kemptville ON Canada K0G1J0	Address	: 60 Northland Road, Unit 1 Waterloo ON Canada N2V 2B8
Telephone	: 613 860 0923	Telephone	: 613 225 8279
Project	: 200977	Date Samples Received	: 09-Mar-2023 11:15
PO	: ----	Date Analysis Commenced	: 12-Mar-2023
C-O-C number	: ----	Issue Date	: 20-Mar-2023 14:27
Sampler	: CLIENT		
Site	: ----		
Quote number	: SOA 2022		
No. of samples received	: 1		
No. of samples analysed	: 1		

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted. This document shall not be reproduced, except in full.

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results

Additional information pertinent to this report will be found in the following separate attachments: Quality Control Report, QC Interpretive report to assist with Quality Review and Sample Receipt Notification (SRN).

Signatories

This document has been electronically signed by the authorized signatories below. Electronic signing is conducted in accordance with US FDA 21 CFR Part 11.

Signatories	*Position*	*Laboratory Department*
Greg Pokocky	Supervisor - Inorganic	Inorganics, Waterloo, Ontario

General Comments

The analytical methods used by ALS are developed using internationally recognized reference methods (where available), such as those published by US EPA, APHA Standard Methods, ASTM, ISO, Environment Canada, BC MOE, and Ontario MOE. Refer to the ALS Quality Control Interpretive report (QCI) for applicable references and methodology summaries. Reference methods may incorporate modifications to improve performance.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis.

Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

Please refer to Quality Control Interpretive report (QCI) for information regarding Holding Time compliance.

Key : CAS Number: Chemical Abstracts Services number is a unique identifier assigned to discrete substances
LOR: Limit of Reporting (detection limit).

Unit	*Description*
µS/cm	microsiemens per centimetre
mg/kg	milligrams per kilogram
ohm cm	ohm centimetres (resistivity)
pH units	pH units

<: less than.

>: greater than.

Surrogate: An analyte that is similar in behavior to target analyte(s), but that does not occur naturally in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery.

Test results reported relate only to the samples as received by the laboratory.

UNLESS OTHERWISE STATED on SRN or QCI Report, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.

Analytical Results

Sub-Matrix: **Soil/Solid**

(Matrix: **Soil/Solid**)

Analyte	CAS Number	Method	LOR	Unit	TEST PIT 4-B 2050 DUNROBIN RD	----	----	----	----
				Client sampling date / time	09-Mar-2023 08:00	----	----	----	----
					WT2305798-001	--------	--------	--------	--------
					Result	----	----	----	----
Physical Tests									
Conductivity (1:2 leachate)	----	E100-L	5.00	µS/cm	60.4	----	----	----	----
pH (1:2 soil:CaCl2-aq)	----	E108A	0.10	pH units	6.34	----	----	----	----
Resistivity	----	EC100R	100	ohm cm	16600	----	----	----	----
Leachable Anions & Nutrients									
Chloride, soluble ion content	16887-00-6	E236.Cl	5.0	mg/kg	<5.0	----	----	----	----
Sulfate, soluble ion content	14808-79-8	E236.SO4	20	mg/kg	<20	----	----	----	----

Please refer to the General Comments section for an explanation of any qualifiers detected.

Permeability Test Results

SoilMoisture
Soil Hydrology Instruments for Research Since 1965

Guelph Permeameter

Input

Result

Test #1

Reservoir Cross-sectional area in cm^2

(enter "35.22" for Combined and "2.16" for Inner reservoir): **35.22**
Enter water Head Height ("H" in cm): **15**
Enter the Borehole Radius ("a" in cm): **6**

Enter the soil texture-structure category (enter one of the below numbers): **2**

1. Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.
2. Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.
3. Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.
4. Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macropors, etc

Steady State Rate of Water Level Change ("R" in cm/min): **0.1000**

Res Type 35.22
H 15
a 6
H/a 2.5
a* 0.04
C0.01 1.033
C0.04 1.085
C0.12 1.063
C0.36 1.063
C 1.085
R 0.100
Q 0.059
pi 3.142

α^* = 0.04 cm^{-1}

C = 1.08468
Q = 0.0587

K_{fs} = 1.64E-05 cm/sec
9.81E-04 cm/min
1.64E-07 m/sec
3.86E-04 inch/min
6.44E-06 inch/sec

Φ_m = 4.09E-04 cm^2/min

Test #2

Reservoir Cross-sectional area in cm^2

(enter "35.22" for Combined and "2.16" for Inner reservoir): **35.22**
Enter water Head Height ("H" in cm): **5**
Enter the Borehole Radius ("a" in cm): **6**

Enter the soil texture-structure category (enter one of the below numbers): **2**

1. Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.
2. Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.
3. Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.
4. Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macropors, etc

Steady State Rate of Water Level Change ("R" in cm/min): **0.3000**

Res Type 35.22
H 5
a 6
H/a 0.83333
a* 0.04
C0.01 0.52372
C0.04 0.53755
C0.12 0.48911
C0.36 0.48911
C 0.53755
R 0.300
Q 0.1761
pi 3.1415

α^* = 0.04 cm^{-1}

C = 0.53755
Q = 0.1761

K_{fs} = 9.44E-05 cm/sec
5.66E-03 cm/min
9.44E-07 m/ses
2.23E-03 inch/min
3.71E-05 inch/sec

Φ_m = 2.36E-03 cm^2/min

Test #3

Reservoir Cross-sectional area in cm^2

(enter "35.22" for Combined and "2.16" for Inner reservoir): **35.22**
Enter water Head Height ("H" in cm): **5**
Enter the Borehole Radius ("a" in cm): **6**

Enter the soil texture-structure category (enter one of the below numbers): **2**

1. Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.
2. Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.
3. Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.
4. Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macropors, etc

Steady State Rate of Water Level Change ("R" in cm/min): **0.4000**

Res Type 3.52E+01
H 5
a 6
H/a 0.833333333
a* 0.04
C0.01 0.523723554
C0.04 0.537554004
C0.12 0.489114302
C0.36 0.489114302
C 0.537554004
R 0.400
Q 2.35E-01
pi 3.1415

α^* = 0.04 cm^{-1}

C = 0.53755
Q = 2.35E-01

K_{fs} = 0.00013 cm/sec
7.55E-03 cm/min
1.26E-06 m/ses
2.97E-03 inch/min
4.95E-05 inch/sec

Φ_m = 3.15E-03 cm^2/min

Calculation formulas related to shape factor (C). Where H_1 is the first water head height (cm), H_2 is the second water head height (cm), a is borehole radius (cm) and α^* is microscopic capillary length factor which is decided according to the soil texture-structure category. For one-head method, only C_1 needs to be calculated while for two-head method, C_1 and C_2 are calculated (Zang et al., 1998).

Soil Texture-Structure Category	$\alpha^*(cm^{-1})$	Shape Factor
Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.	0.01	$C_1 = \left(\frac{H_2/a}{2.081 + 0.121\left(H_2/a\right)}\right)^{0.672}$
Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.	0.04	$C_1 = \left(\frac{H_1/a}{1.992 + 0.091\left(H_1/a\right)}\right)^{0.683}$ $C_2 = \left(\frac{H_2/a}{1.992 + 0.091\left(H_2/a\right)}\right)^{0.683}$
Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.	0.12	$C_1 = \left(\frac{H_1/a}{2.074 + 0.093\left(H_1/a\right)}\right)^{0.754}$ $C_2 = \left(\frac{H_2/a}{2.074 + 0.093\left(H_2/a\right)}\right)^{0.754}$
Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macro pores, etc.	0.36	$C_1 = \left(\frac{H_1/a}{2.074 + 0.093\left(H_1/a\right)}\right)^{0.754}$ $C_2 = \left(\frac{H_2/a}{2.074 + 0.093\left(H_2/a\right)}\right)^{0.754}$

Calculation formulas related to one-head and two-head methods. Where R is steady-state rate of fall of water in reservoir (cm/s), K_{fs} is Soil saturated hydraulic conductivity (cm/s), Φ_m is Soil matric flux potential (cm²/s), α^* is Macroscopic capillary length parameter (from Table 2), a is Borehole radius (cm), H_1 is the first head of water established in borehole (cm), H_2 is the second head of water established in borehole (cm) and C is Shape factor (from Table 2).

One Head, Combined Reservoir	$Q_1 = \bar{R}_1 \times 35.22$	$K_{fs} = \frac{C_1 \times Q_1}{2\pi H_1^2 + \pi a^2 C_1 + 2\pi\left(\frac{H_1}{\alpha^*}\right)}$
One Head, Inner Reservoir	$Q_1 = \bar{R}_1 \times 2.16$	$\Phi_m = \frac{C_1 \times Q_1}{(2\pi H_1^2 + \pi a^2 C_1)\alpha^* + 2\pi H_1}$
Two Head, Combined Reservoir	$Q_1 = \bar{R}_1 \times 35.22$ $Q_2 = \bar{R}_2 \times 35.22$	$G_1 = \frac{H_2 C_1}{\pi\left(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1)\right)}$ $G_2 = \frac{H_1 C_2}{\pi\left(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1)\right)}$ $K_{fs} = G_2Q_2 - G_1Q_1$ $G_3 = \frac{(2H_2^2 + a^2C_2)C_1}{2\pi\left(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1)\right)}$
Two Head, Inner Reservoir	$Q_1 = \bar{R}_1 \times 2.16$ $Q_2 = \bar{R}_2 \times 2.16$	$G_4 = \frac{(2H_1^2 + a^2C_1)C_2}{2\pi\left(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1)\right)}$ $\Phi_m = G_3Q_1 - G_4Q_2$

National Building Code Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

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

Site: 45.394N 75.982W

2021-11-12 15:50 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.405	0.218	0.129	0.039
Sa (0.1)	0.477	0.268	0.165	0.055
Sa (0.2)	0.401	0.230	0.145	0.050
Sa (0.3)	0.305	0.177	0.113	0.040
Sa (0.5)	0.217	0.127	0.081	0.029
Sa (1.0)	0.110	0.065	0.042	0.014
Sa (2.0)	0.053	0.031	0.019	0.006
Sa (5.0)	0.014	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.257	0.146	0.090	0.029
PGV (m/s)	0.181	0.102	0.063	0.020

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

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

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

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

Response to Geotechnical Review Comments

Kollaard Associates
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The following geotechnical review comments were received August 14, 2023 Kollaard Associates Inc.'s response is provided in italics immediately after each comment for clarity:

Development Review Comments – Derek Kulyk (Project Manager)
c) Geotechnical Investigation, Proposed Residential Subdivision; Part 1, Plan 5R-10184, 2050 Dunrobin Road, West Carleton Ward, City of Ottawa, Ontario; prepared by Kollaard Associates, dated Nov 12, 2021, Revision 1 – dated May 5, 2023.

1. (Originally Comment #4) Please identify potential long term low groundwater level and how it was considered in the overall permitted ground raise and its contribution to consolidation and potential ground settlement, including for the proposed subdivision road, due to soil's shear strength loss - comment was not addressed as requested, report needs to be updated directly.
Response to the original comment, from Kollaard Associates, dated May 5, 2023, is acceptable but the rationale needs to be added to the revised report, not as an external commentary.

The response was added to the report in section 5.2.6.

2. Section 5.5.2 Pavement Structure recommends 300 mm of Granular B, while the City of Ottawa Rural Local Roadway Cross-Section Over Earth, Drawing Number R-27, March 2016, specifies minimum Granular B layer of 400 mm.

The cross section has been modified in the report and on the drawings.

3. Section 4.8 of the report makes a reference to Attachment C following the report, however none of the attachments are labelled.

The wording has been modified to read "The laboratory results are presented in the Attachments following the text of this report"

4. Concrete retaining wall (headwall) is proposed on site, however the report makes no reference to its geotechnical design and construction requirements. Slope Stability Assessment needs to be provided that addresses the global stability of the proposed wall.

The outlet has been modified. The outlet control will consist of a V-notch weir in a cast in place concrete wall. Geotechnical recommendations with respect to the weir wall have been added to the report.

A slope stability assessment does not need to be provided for the weir walls as the soil height on either side of the wall is similar which means that the weir walls are not acting as retaining walls.

The footing of the proposed structure appears to be below the GWT which was identified in TP 14 as 0.8 m below ground surface. Footing/foundation geotechnical considerations need to be provided.

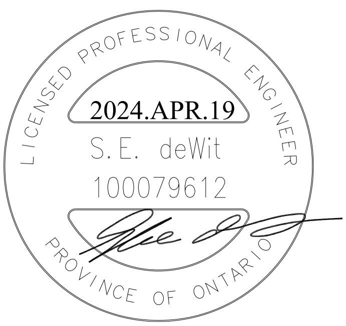
Please see response to comment 1 above, (original question 4.) The geotechnical report *already provides recommendations related to excavation, construction dewatering, subgrade preparation, backfill and site services. A separate section in the report to provide the same information again is not necessary.*

5. Environmental Noise Impact Assessment 20 Dunrobin Road prepared for Zbigniew Hauderowicz (Prepared by Arcadis IBI Group, dated May 20,2022; updated June 21, 2022; updated November 11, 2022), proposed 2m tall noise barrier with 20 kg/m_2 density at the north limit of the site to mitigate the kennel noise for lots 3, 5 and 7, however the Geotechnical report did not provide ant geotechnical considerations for the proposed structure.

Section added to report

We trust that this response provides sufficient information for your present purposes. If you have any questions concerning this response please do not hesitate to contact our office.

Sincerely,



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Kollaard File # 200977
City of Ottawa File No. D02-02-22-0018
MVCA File No. PMRZA-43

March 12, 2025

The following geotechnical review comments were received dated February 10, 2025 Kollaard Associates Inc.'s response is provided in italics immediately after each comment for clarity:

i. Geotechnical Investigation, Proposed Residential Subdivision Part 1, Plan 5R-10284 2050 Dunrobin Road West Carleton Ward City of Ottawa, Ontario prepared by Kollaard Associates Engineers, dated and sealed April 19, 2024.

Comments:
1. Section 5.3 must be revised to reference only the one weir wall for the SWM block since the design has changed.

Corrected in report

2. Provide a copy of the geotechnical report within the next submission.

Noted. The most recent version of the report is revision 4 dated March 12, 2025

The following two comments are based upon satisfying the Geotechnical condition as part of the Draft Plan of Subdivision:

3. The report must discuss design and construction provisions of swimming pools.

Discussion of swimming pools added to the report as Section 5.5

4. The report must discuss any slopes (existing or proposed) which are considered to be unstable and require a slope stability analysis. If none exist or are proposed on-site, briefly state this assertion within the report

Discussion of slope stability added to the report as Section 5.6. Numbering of the subsequent sections was adjusted accordingly.