Kollaard Associates
Engineers
215 Sanders Street, Unit 1
P.O. Box 189
Kemptville, Ontario K0G 1J0

Civil • Geotechnical •
Structural • Environmental •
Hydrogeological

(613) 860-0923

FAX: (613) 258-0475

REPORT ON

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

Submitted to:

Hauderowicz, Zbigniew and Teresa
165 Constance Lake Road
Kanata, Ontario
K2K 1X7

PROJECT #: 200977

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Issued in support of the Subdivision Agreement — November 12, 2021
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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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May 5, 2023 200977

Hauderowicz, Zbigniew and Teresa
165 Constance Lake Road
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K2K 1X7

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 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 Attachment C following this report.

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 ground water 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 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.

5.2.7 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.

5.2.8 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.

5.2.9 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 Seismic Design for the Proposed Residential Buildings

5.3.1 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.

5.3.2 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.

5.3.3 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.

5.4 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.

5.5 Roadways

5.5.1 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.

5.5.2 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
300 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.

5.6 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.

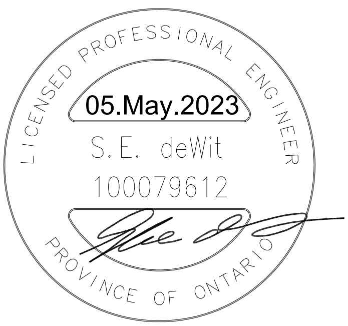
Any blasting should be carried out under the supervision of a blasting specialist engineer. Pre-blast condition surveys of nearby structures and existing utilities are essential. Monitoring of the blasting should be carried out throughout the blasting period to ensure that the blasting meets the limiting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

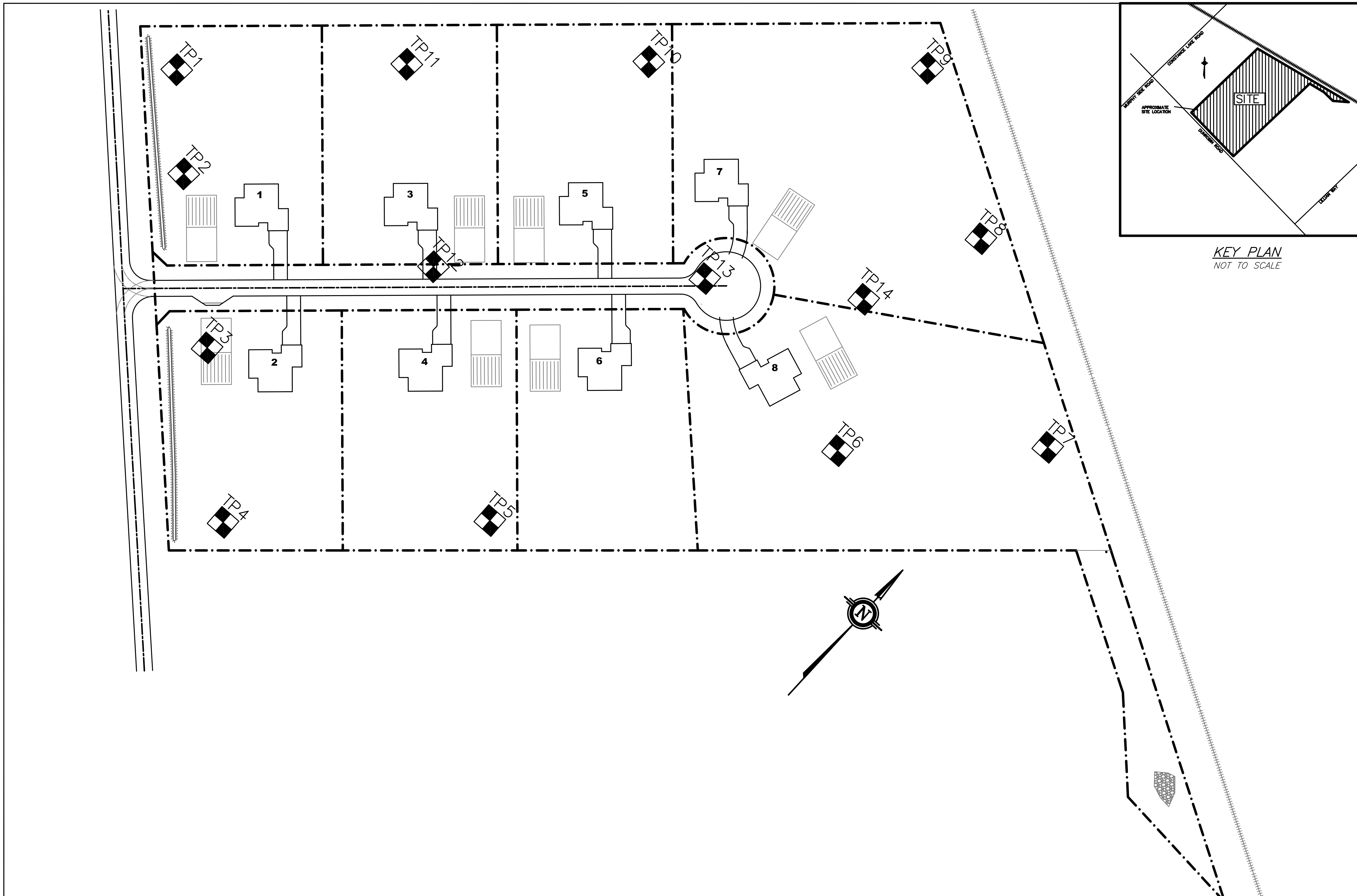
KEY PLAN	FIGURE 1

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
FACSIMILE (613) 258-0475

(613) 860-0923

LICENCED PROFESSIONAL ENGINEER
12.NOV.2021
S.E. deWit
100079612
PROVINCE OF ONTARIO

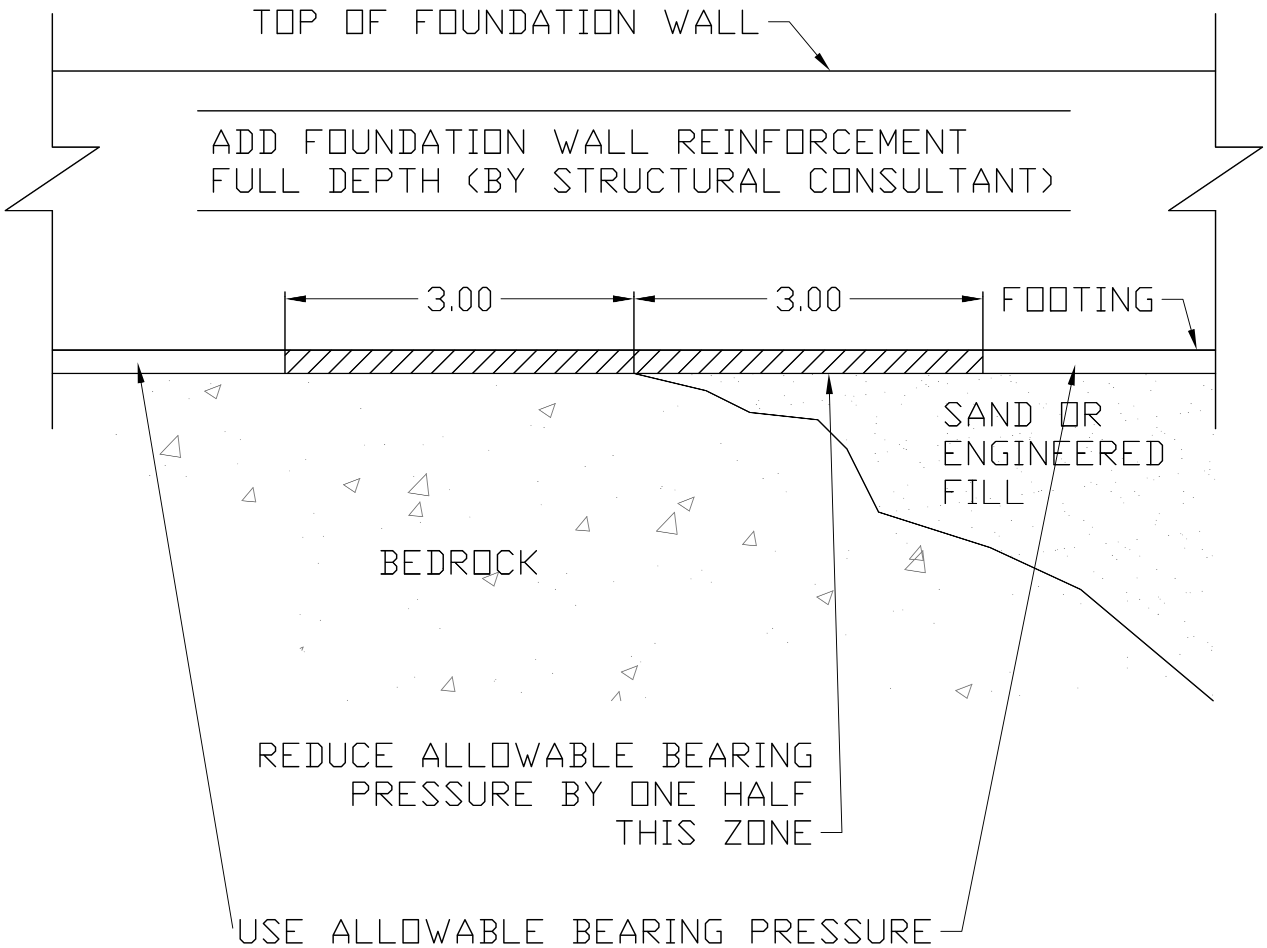
DESIGN	SD
CHECKED	SD
DRAWN	SD
CHECKED	SD
APPROVED	SD

SCALE
1: 750
0 5 10 20 30

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

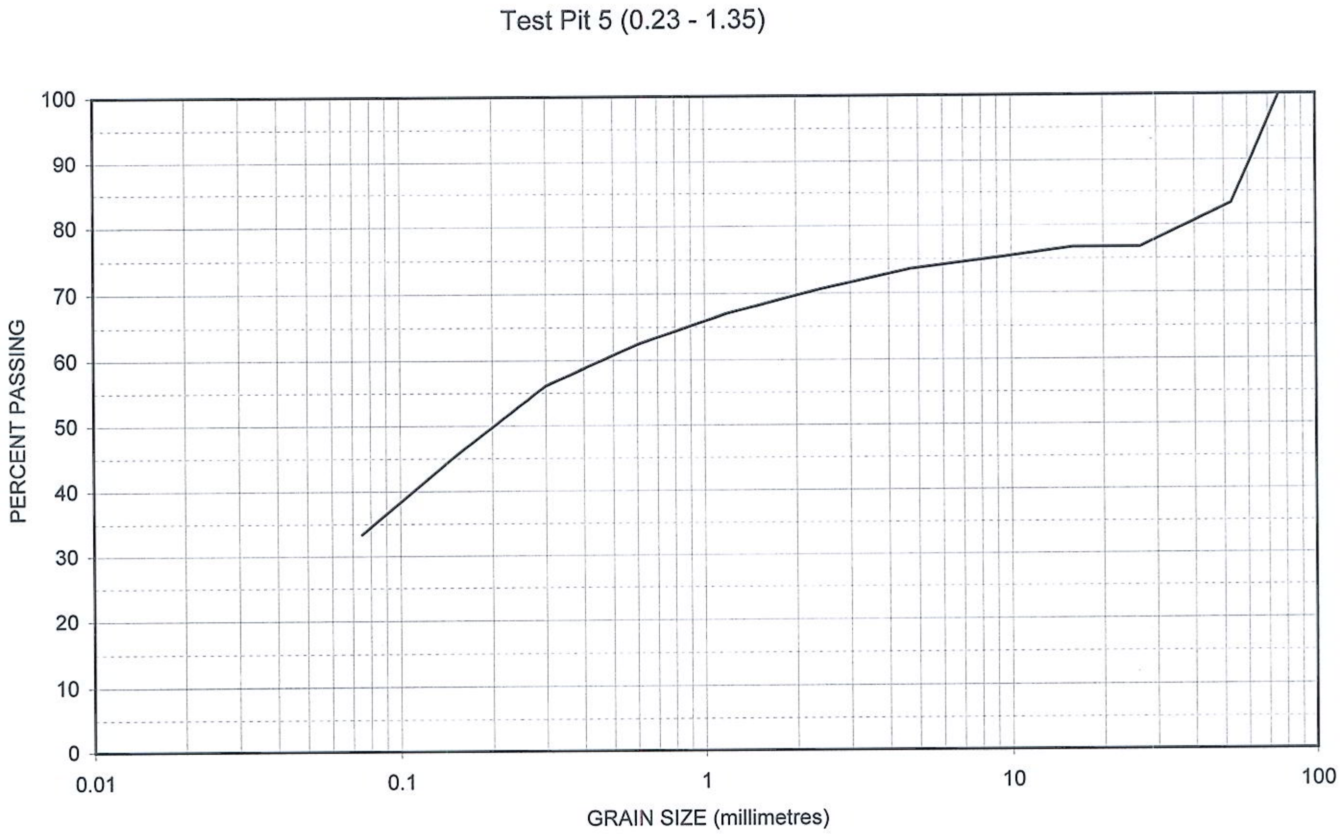
KOLLAARD ASSOCIATES INC.

LIST OF ATTACHMENTS

Sieve Analysis Test Results

Grain Size Distribution Analysis

Kollaard Associates
Engineers



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.

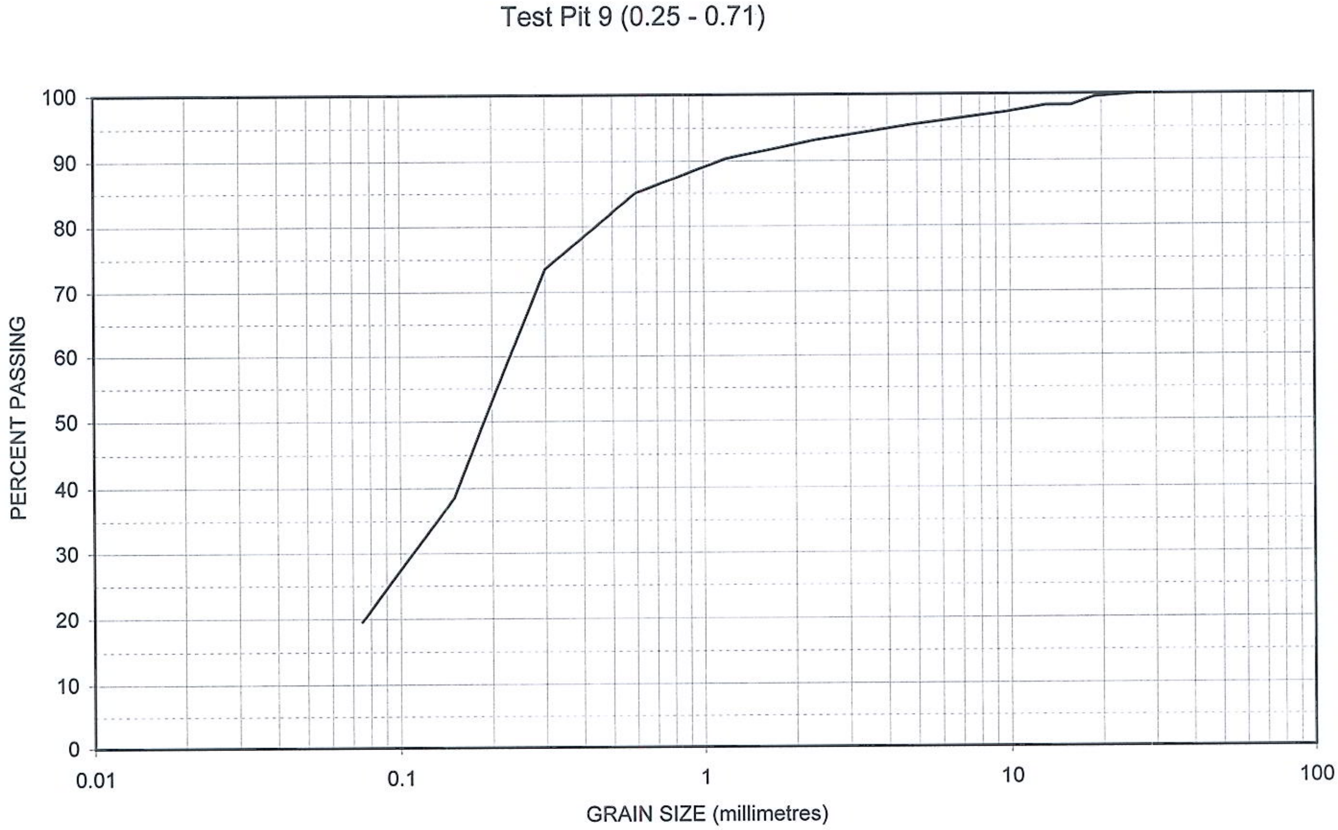
Kollaard Associates
Engineers
Box 189, 215 Sanders Street, Unit 1
Kemptville, Ontario K0G 1J0
(613) 860-0923, FAX: (613) 258-0475

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

Date: October 12, 2007

Grain Size Distribution Analysis

Kollaard Associates Engineers



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.

Kollaard Associates
Engineers
Box 189, 215 Sanders Street, Unit 1
Kemptville, Ontario K0G 1J0
(613) 860-0923, FAX: (613) 258-0475

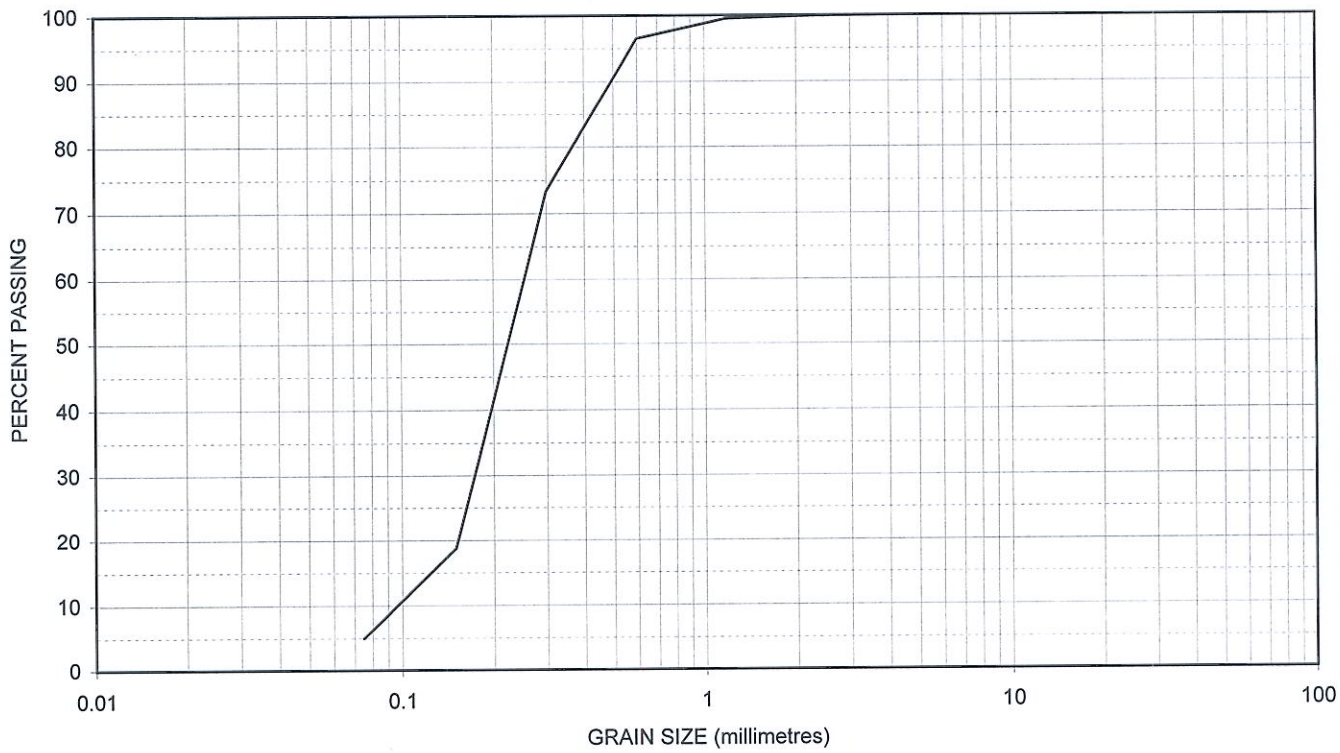
Issued by: C. R. Morey, P. Eng.

Date: October 12, 2007

Grain Size Distribution Analysis

Kollaard Associates Engineers

Test Pit 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.

Kollaard Associates
Engineers
Box 189, 215 Sanders Street, Unit 1
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

ALS Canada Ltd.

right solutions.
right partner.

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)**

				Client sample ID	TEST PIT 4-B 2050 DUNROBIN RD	----	----	----	----
				Client sampling date / time	09-Mar-2023 08:00	----	----	----	----
Analyte	CAS Number	Method	LOR	Unit	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.

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 Geotechincal Review Comments

Kollaard Associates
Engineers
210 Prescott Street, Unit 1
P.O. Box 189
Kemptville, Ontario K0G 1J0

Civil • Geotechnical •
Structural • Environmental •
Materials Testing •
(613) 860-0923
FAX: (613) 258-0475

The following geotechnical review comments were received May, 2022. Kollaard Associates Inc.'s response is provided in italics immediately after each comment for clarity:

a. 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)

1. PEO logo is not allowed to be used in a report and needs to be removed, as per their requirements.

Please note: Kollaard Associates has confirmed with PEO that the use of the Logo is acceptable.

2. Please state clearly if there are sensitive marine clays present on site and in what areas

Statement added to Report
There was no sensitive marine clay encountered at the site. Silty clay was encountered in 2 of the 14 test pits. However the silty clay present was weathered into a stiff to very stiff crust and is not considered to be sensitive in that condition.

3. Annual high-water level derived from spring time investigation needs to be provided and discussed in the report in detail, as the groundwater level appears to be at depth as shallow as 0.6 m below the surface, when measured in July. Please provide a clear analysis, if seasonal ground water level fluctuations will have any short- or long-term effects on the subgrade seasonal condition, considering that all proposed houses and the road will be placed on a localized substantial amount of fill and within close proximity to ditches that might seasonally alter groundwater levels.

Seasonal Groundwater level fluctuations will not have any short or long term effects on the subgrade seasonal condition, especially considering that proposed houses and the road will be placed on a localized substantial amount of fill.

Water infiltrating downward from the surface does not constitute groundwater. If it did, then the seasonally high groundwater level would be at the surface everywhere in Ottawa as this phenomenon occurs every time it rains.

As stated in the geotechnical guideline, sandy soils and glacial till are generally not susceptible to shrinkage due to moisture depletion. This also means that sandy soils and glacial till are not *susceptible to expansion with added moisture. Further, the sandy soils and glacial till encountered at the site were observed to be in a compact to dense state of backing based on difficulty of excavation and tactile examination. Since these soils are not susceptible to expansion or shrinkage, the soil structure, which will support the houses and roads, will not be susceptible to seasonal fluctuation of the groundwater level as it occurs.*

It is commonly recommended that the allowable bearing capacity in sandy soils is reduced by a factor of 2 below the groundwater level. The allowable bearing capacity provided in the reports account for this.

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.

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.

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. As such the potential for these occurrences do not merit specific discussion given the light loading of the proposed residential development.

5. It appears that most footings are being proposed on a layer of fill, above the existing ground level containing topsoil, consequently, please address the statements regarding fill and fill material types more explicitly towards the proposed design condition, not as a potential generic option. The recommendations should be clear and evident (there are some statements that hint at potential topsoil and organic matter removals but they are offered as potential solutions, if such conditions are encountered) The report needs to address the proposed design conditions more adequately, not as generic statements. This also pertains to the public road that will be constructed on fill.

*The report says "*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." *There is nothing unclear about this requirement. This statement explicitly states that the topsoil and deleterious materials should be removed from the foundation excavations.*

The word generic means "characteristic of or relating to a class or group of things; not specific". As this comment points out, most of the development that consists of the proposed 8 lots and the roadway share similar conditions in terms of expected subgrade elevation in relation to the existing ground surface. Since the majority of the development is similar it would be reasonable to group the development together. Any reference relating to the group would by definition then be generic. Since the geotechnical report is intended to address the proposed development and not one specific lot or element of the development, a generic recommendation would be most suited for the report.

The geotechnical report requires that a subgrade inspection be completed by a qualified geotechnical person before the addition of material in the roadway, or building envelope or placement of the foundation footing. There inspections are intended to ensure that the subgrade surface is properly prepared. It is acknowleged that the statement, **"could be raised to the proposed founding level using suitable imported engineered fill"**, *leaves the possibility that there are other options. This makes the statement true, not unclear or a hint. There are other options, such as filling the entire area with lean mix concrete that, while not common practise due to cost, are valid options and will not detract from the development.*

The wording has been modified in the report.

6. Since most of the footings will be placed above the existing ground level and backfilled with close to 3.0 m of fill, the report needs to adequately and unambiguously discuss and provide a conclusion if the pre-consolidation pressure of the underlying ground is in excess of the final stress level that it will be exposed to, after the fill above it is placed and compacted. Compaction impact should also be discussed. As the pre-consolidation stress might be different for different material types and depths, reasonable discussion and conclusions need to be provided. Slope stability of the fill needs to be discussed and safe fill slopes need to be defined, as this will determine the minimum grading limits. This analysis should also include the road fill considerations.

This level of discussion could be reasonably expected if the development included large or heavy buildings on a site with either significant soil thicknesses or sensitive soils. Since none of these conditions are present at the site and the development will consist of light residential construction, detailed discussion is not warranted.
Discussion with respect to the stability of the fill slopes has been added to the report.

7. Please provide a clear recommendation how the ground needs to be prepared for the footings and the foundation walls and process of backfilling.

This is already clear in the report where it states that the topsoil, fill and deleterious materials must be removed and the subgrade should be approved.
The process of backfilling for a single family residential building of the nature that will be constructed within the proposed development is governed by Part 9 the Ontario Building Code.

8. Please provide clear foundation wall drainage recommendations. If sump pump is proposed, please make a reference to the "Ottawa Sewer Design Guidelines" Section 5.12 (ISTB 2018-04). Discussion of the foundation drainage should also include if there is any 100-year storm flood level influence and if so, how it should be dealt with.

A review of the "Ottawa Sewer Design Guidelines" Section 5.12 (ISTB 2018-04) *as a whole indicates that this section was, in general, written for infill and serviced Greenfield development.*

This section begins with

> Screening Criteria for Areas Where Sump Pump Systems May be Considered:
> "The use of sump pump systems for the purpose of foundation drainage will require all of the following conditions to be satisfied prior to acceptance and implementation:
> 1. The area under consideration is on full services.
> 2. The area under consideration is underlain by clay soils subject to grade raise restrictions.
> 3. The finished grades that would be required to allow gravity drainage would exceed permissible grade raises,

Sentence 1 is sufficient to indicate that this section of the guideline is not really applicable to a rural residential development without a storm sewer.

Sentence 2 and 3 indicate that this guideline is most suited to development in clay soils. This does not appear to be particularly relevant to a rural construction where lot drainage is directed to a roadside ditch with the consideration that raising the underside of footing sufficiently high to ensure gravity drainage from the USF to the roadside ditch at a level above any potential HGL is not realistic.

In general, the proposed founding level for a rural dwelling is set above the high ground water level with the intent that there is minimal demand on the foundation drain. Generally this places the proposed founding level slightly above the adjacent roadside ditch level. As indicated with the present proposed development, placing the proposed founding level above the invert of the ditch typically results in a founding level in close proximity to the existing ground surface. Since there are no storm sewers the roadside ditch is typically the outlet for discharge from the foundation drainage system. The limited difference in elevation between the foundation drainage system and the invert of the ditch means that elevated level in the ditch during significant storm events would cause backup in a gravity system from the ditch to the dwelling foundation. Since the demand on the foundation system will occur during a significant storm event, this backup would likely cause flooding of the foundation. This is the reason sump pumps are recommended and standard in rural un-serviced construction.

The purpose of discharging the foundation drainage system by means of a sump pump is to completely remove the foundation system from any potential connection to the ditch drainage system and any flows in the ditches during a storm event.

Reference to section 5.12.2.1, 5.12.2.2, 5.12.2.3 sentences 1-9, 5.12.2.7 have been added to the report. A request for a hydrogeological assessment in keeping with section 5.12.2 for this particular proposed development is ludicrous.

9. Please provide the recommended insulation detail, mentioned in Section 5.2.6 of the report.

The option for insulation has been removed from the report

10. Are there any grade raise restrictions for the placement of the berm adjacent Dunrobin Road? There is no consideration given to the design of that berm in the report. Please provide.

The intent of the berm was to provide back slope for the ditch along Dunrobin Road in order to prevent runoff originating from a public road flowing onto private property. The design has been slightly revised to reduce the height of the berm and blend the berm with the lot grading.
There is no special consideration required for the new design.

11. All grade raise locations and fill, including in the MVCA area need to be discussed and clear recommendations are to be provided. Slope stability needs to be discussed, also for temporary conditions.

Discussion added to the report. There are no temporary slope stability concerns.

12. In the section of the report discussing soil bearing capacity, please include the Unit weight of the soil and soil's undrained shear strength in the discussion and conclusion, as it relates to the overall soil's bearing capacity, especially that excessive amounts of fill will be used. Please justify how 20 mm and 15 mm settlement was determined. Please also discuss the engineered fill bearing capacity.

While it is acknowledged that unit weight as well as the other mentioned soil properties all contribute to the overall soil bearing capacity, there was no definitive statement provided in the report as to what the capacity of the soil is. The report states

> ",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 actual capacity of the soil at the thicknesses present that would result in settlement of 25 mm is many times higher than the given value of 100 kPa. It is common knowledge that the stiff clay crust and glacial tills in the Ottawa Valley when present in limited thickness are suitable to support *footings designed for 100 kPa SLS. As such, justification is not required. Anything greater than 100 kPa is typically not required in the design of normal residential development. The laboratory tests and design calculations required to verify the actual capacity of the soil will cost many times more in terms of laboratory and engineering fees than any saving in foundation costs that would result from a higher bearing capacity. As such, sound engineering judgement would indicate that further detailed investigation and analysis given the subsurface conditions and proposed development is not warranted.*

The report does not say that there will be 20 mm and 15 mm of settlement. It states that the settlement will be less than these values. These values were provided in the context of foundations bearing on bedrock or on engineered fill placed on bedrock and compacted to a minimum of 95 percent SPMDD. Bedrock is typically not prone to settlement under loading from residential dwellings. As such the expected settlement would be negligible. Any settlement more than 20 mm would be extremely unlikely and extremely unexpected. For the situation to occur which leads to granular fill being placed on bedrock, the bedrock would have to be relatively close to the founding level. Even if the engineered fill was 2 m thick, at 95% SPMDD, there is a minimal total void volume remaining within the fill. This means that granular would essentially have to be compressed into a solid for significant settlement to occur. The loading from a residential dwelling and any amount of grade raise is not sufficient to accomplish this. As such the expected settlement will be less than 20mm.

Differential settlement of 20 mm and total settlement of 25 mm is used as a general guide to design foundations. A statement in the report that the expected total settlement will be less than 25 mm and differential settlement will be less than 20 mm allows any person design the foundations to assume that they can use the general assumptions with respect to settlement accepted in foundation design. It also indicates that foundations designed using Part 9 of the Ontario Building Code will be acceptable for the development.

Wording has been revised in the report.
There are no excessive amounts of fill.

There is not reasonable or possible scenario or reasonable expectation for a scenario in which engineered fill consisting of OPSS Granular A or OPSS Granular B Type II when placed on bedrock and compacted to 95% SPMDD would have a bearing capacity of less than 100 kPa if the fill extends out from the foundation as specified in the report.

13. General minimum depth of test pits for a subdivision is 4-5 m (glacial till, compacted sand, stiff weathered clay) or to bedrock. The pits numbered TP3, TP6, TP13, TP14 did not reach the required depth. Please provide rationale and conclusion, as to why was the minimum required depth not pursued and why the findings were determined to be sufficient to draw sound engineering conclusions and recommendations.

Section 2.5 of the Geotechnical Investigation and Reporting Guidelines for the City of Ottawa states

"The required depth of Investigation depends on many factors, including the type of structure and the associated magnitude of the loading, the subsurface conditions and their variability, the depth of planned excavation, the types of foundations to be constructed."

"the general practice is to investigate to a depth below the planned founding level equal to at least 2 to 3 times the footing width" "The investigation should extend to at least sufficient depth to investigate the most compressible portions of the deposit".

As you have stated in comments 5 and 6 above, the most of the footings will be placed above the existing ground surface.

All of the test pits were advanced to sufficient depth to confirm that there were no weaker or relatively compressible underlying layers. TH6, TH13 and TH14 were extended about 0.6, 1.0 and 1.0 metres (respectively) below the depth at which water entry into the test pits was observed. Further these three test pits were extended to at least 1.6 metres below grade where the USF is expected to be at or above the existing ground surface.

It is noted that Table 1 General Maximum Spacing Between Boreholes & Test pits permits a maximum spacing of 300 metres between test pits with no less than 1 for every 15 single family homes. The site has dimensions of about 230 metres width by 400 metres length. Based on this spacing guideline 6 test pits would be sufficient. 14 test pits were advanced at the site, 10 of which reached refusal on the bedrock. Based on the location the 10 test pits that reached refusal, these 10 test pits would be sufficient to meet the criteria provided by the City of Ottawa of the number of test pits required.

14. Section 5.3.1 (Seismic Site Classification) of the report mentions “limited information” gathered from the test pits, indicating that more information might be required to comprehensively address Seismic classification and Liquefaction potential. Please expand the discussion on these subjects, including criteria, any assumptions and rationale that was used to make the concise statements.

This section has be reworded.
The amount of information gathered is always limited. There is always the potential to obtain one more piece of information no matter how comprehensive the initial investigation is.
Since bedrock is less than 3 metres in depth at all locations where bedrock was encountered, the seismic site classification is at minimum a Site Class C. Since a higher site seismic site classification is irrelevant in terms of light residential construction, further investigation to determine if the actual site classification was higher than a Site Class C is not warranted.

15. The report states that the test pits were loosely backfilled. It appears that at least 3 test pits should be properly backfilled and compacted due to their critical location (road, driveway, septic bed). Please specify required details.

The procedure to address deleterious or disturbed material is already addressed in the report where it stated that the deleterious and loose material must be removed to expose an approved subgrade and the exposed subgrade should be inspected and approved by geotechnical personnel.
No additional discussion added to the report.

16. The report needs to clearly discuss On-site and Excess Soil Management as per latest guidelines (O.Reg.406/19) and include all three Phases of implementation due to unknown construction timelines for all individual lots.

As stated by previous comments with respect to the grade raise (comments 4, 5, 6) and excessive amounts of fill (comment 12), there will be significant amounts of fill required on the site. This fill will be required for both the lot grading and the roadway. While minor amounts of fill will be generated from the swales, generation of excess fill onsite is not going to occur.

Classification of fill brought onto the site is an environmental concern and must be addressed by the environmental consultant of the entity supplying the fill.

No additional discussion added to the report.

17. The report states that the design drawings need to be reviewed by a Geotechnical Engineer. Does that include the Civil Plans and if so, were they reviewed to assure compliance with the discovered geotechnical conditions?

Please note that the geotechnical report and the civil plans were sealed by the same engineer.

This comment implies that the engineer sealing the documents has not reviewed the documents prior to sealing them, which would be a clear violation of the use of the engineers seal.

18. No information is provided in the report, as to what laboratory testing was done on the soil samples. Please provide (refer to section 2.8 Laboratory Testing of the “Geotechnical Investigation and Reporting Guidelines for Development Applications in the City of Ottawa”).

Section 3 of the report states what Laboratory testing was completed. The laboratory testing results are included in the report.

Section 2.8 includes a list of potential laboratory tests which could be completed. It further provides reasons or purposes for completing the test. Of these tests, grain size distribution or sieve analysis *was determined as being relevant to the proposed development and accordingly testing to determine the grain size distribution was completed on select samples.*

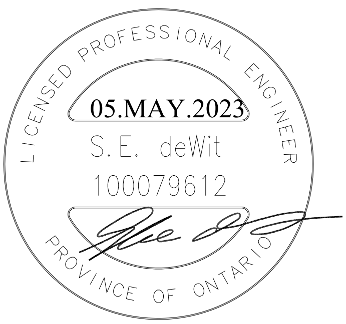
19. Basic soil chemical analysis should be provided.

20. Please provide if there are any restrictions for buried utilities (depth, material type etc.).

There are no restrictions for buried utilities.

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,



Steven deWit, P.Eng.
Kollaard Associates Inc