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

Civil • Geotechnical •
Structural • Environmental •
Industrial Health & Safety

(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

DISTRIBUTION:
DISTRIBUTION
6 copies – City of Ottawa
1 copy – Client
1 copy – Kollaard Associates
1 copy – Mississippi Conservation

Issued in support of the Subdivision Agreement

November 12, 2021

RECORD OF TEST PIT SHEETS

LIST OF FIGURES

LIST OF ATTACHMENTS

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

Civil • Geotechnical •
Structural • Environmental •
Industrial Health & Safety

(613) 860-0923

FAX: (613) 258-0475

November 12, 2021 200977

Hauderowicz, Zbigniew and Teresa
165 Constance Lake Road
Kanata, Ontario
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 single family dwellings will likely be 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.

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.

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. 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 results of the above mentioned hydrogeological investigation indicate that the total overburden thickness at the site, based on test well records provided by the well driller, ranges from about 0.30 to 4.57 metres.

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. It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring.

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. 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.

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.

Provided that any loose and disturbed soil is removed from the bearing surfaces prior to pouring concrete the total and differential footing settlements are expected to be less than 25 and 20 millimetres, respectively, using the above geotechnical reaction value.

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. In this case, the total and differential footing settlements are expected to be less than 20 and 15 millimetres, respectively, provided that the bearing surfaces are cleaned of all loose material prior to pouring concrete or placing the engineered fill.

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 area 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 and for grade raise fill thickness adjacent to the structure of up to 3.0 metres.

5.2.2 Engineered Fill

Should the complete removal of material such as topsoil and any otherwise deleterious material result in a subgrade below the proposed founding level, 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 200 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 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. As such, it is expected that bedrock may be encountered during excavating for the proposed foundations. Small amounts of bedrock removal, if required, can most likely be carried out by hoe ramming. If larger amounts of bedrock removal are required it may be more economically feasible to use drill and blasting techniques and should be carried out under the supervision of a blasting specialist engineer. Monitoring of the blasting should be carried out throughout the blasting period to ensure that the blasting meets the limiting vibration criteria established by the specialist engineer. Pre-blast condition surveys of nearby structures and existing utilities are essential.

5.2.4 Ground Water in Excavation and Construction Dewatering

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

Groundwater was encountered within the test pits put down within the 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 dated September 23, 2021, the proposed underside of footing elevation for the dwellings, to be located where groundwater was encountered, is 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.

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.

Where less than the required depth of soil cover can be provided, the foundation elements should be protected from frost by using a combination of earth cover and extruded polystyrene rigid insulation. A typical frost protection insulation detail could be provided upon request, if required.

5.2.7 Foundation Wall Backfill and Drainage

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.

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.

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. If a sump is used, the sump should be equipped with an emergency backup pump. The sump discharge should be equipped with a backup flow protector.

5.2.8 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 limited information from the test pits, for seismic design purposes, in accordance with the 2012 OBC Section 4.1.8.4, Table 4.1.8.4.A., the site classification for seismic site response is Site Class C. The subsurface conditions in the upper 30 metres below the proposed footing design levels are indicated to consist of a thin layer of overburden followed by bedrock.

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 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 of compact to dense silty sand/sand, stiff silty clay, compact to dense glacial till and bedrock.

Where silty clay soils are encountered at thr proposed dwelling 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. Large trees should be planted no closer than 1 times their height from a proposed dwelling.

Excluding the silty clay, the remainder of the subsurface soils encountered at the site are not particularly sensitive to depletion of moisture by trees. As such there are no planting restrictions associated with the sites where the proposed dwellings will not be located on silty clays.

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:

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

Steven deWit, P. Eng.

Attachments: Table I - Record of Test Pits
Figures 1 to 3

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.

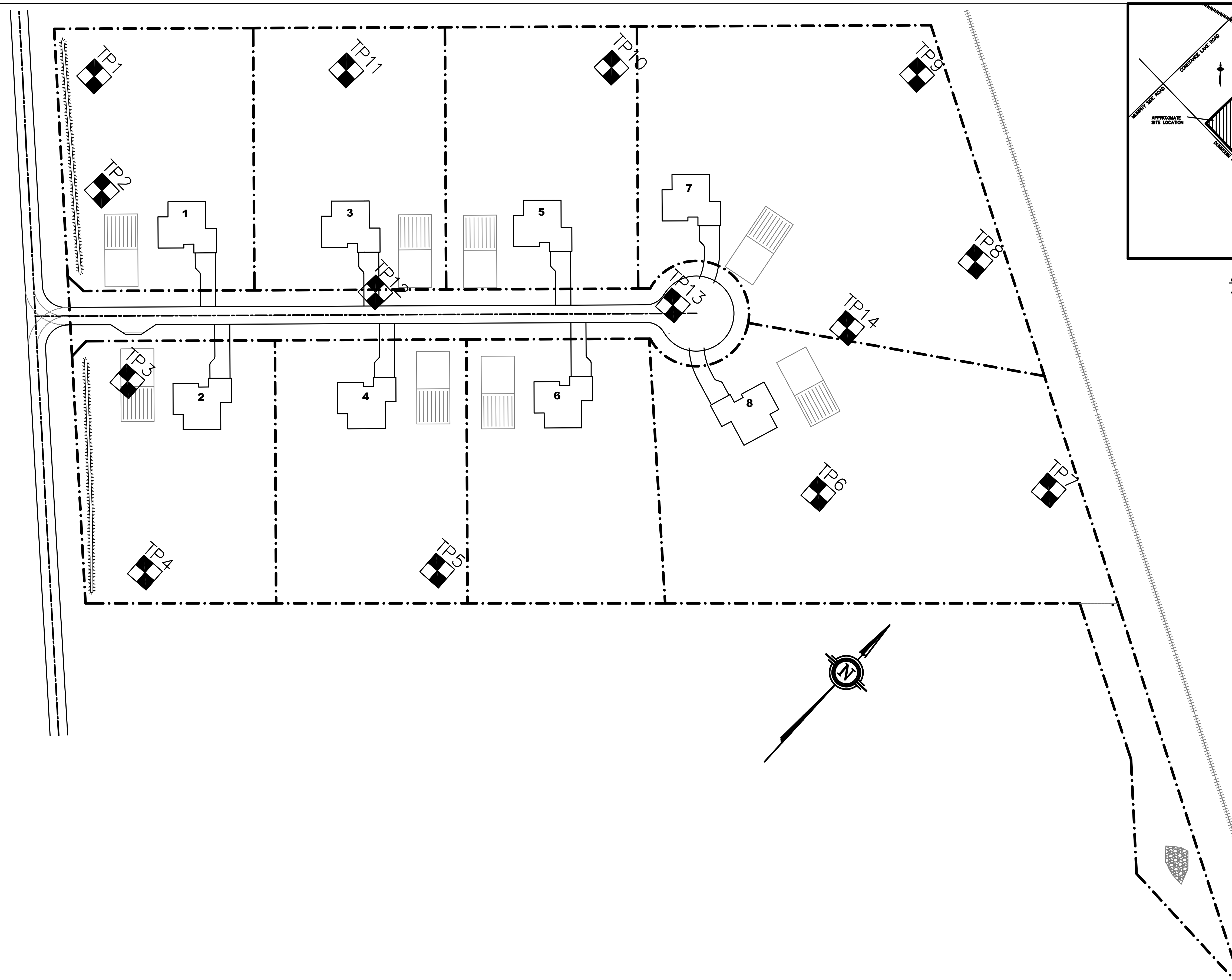
KEY PLAN	FIGURE 1

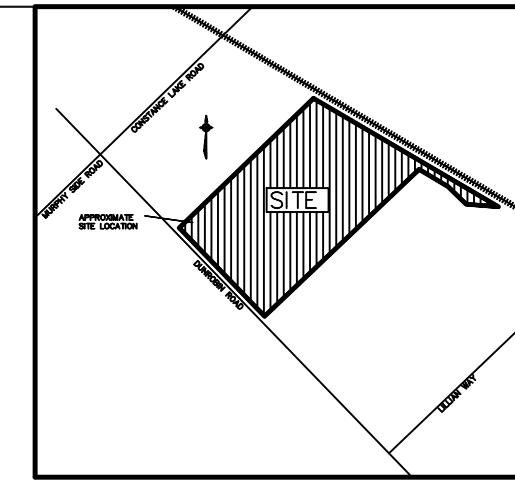
NOT TO SCALE

Kollaard Associates
Engineers

Project No. 200977

Date November 12, 2021





KEY PLAN
NOT TO SCALE

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

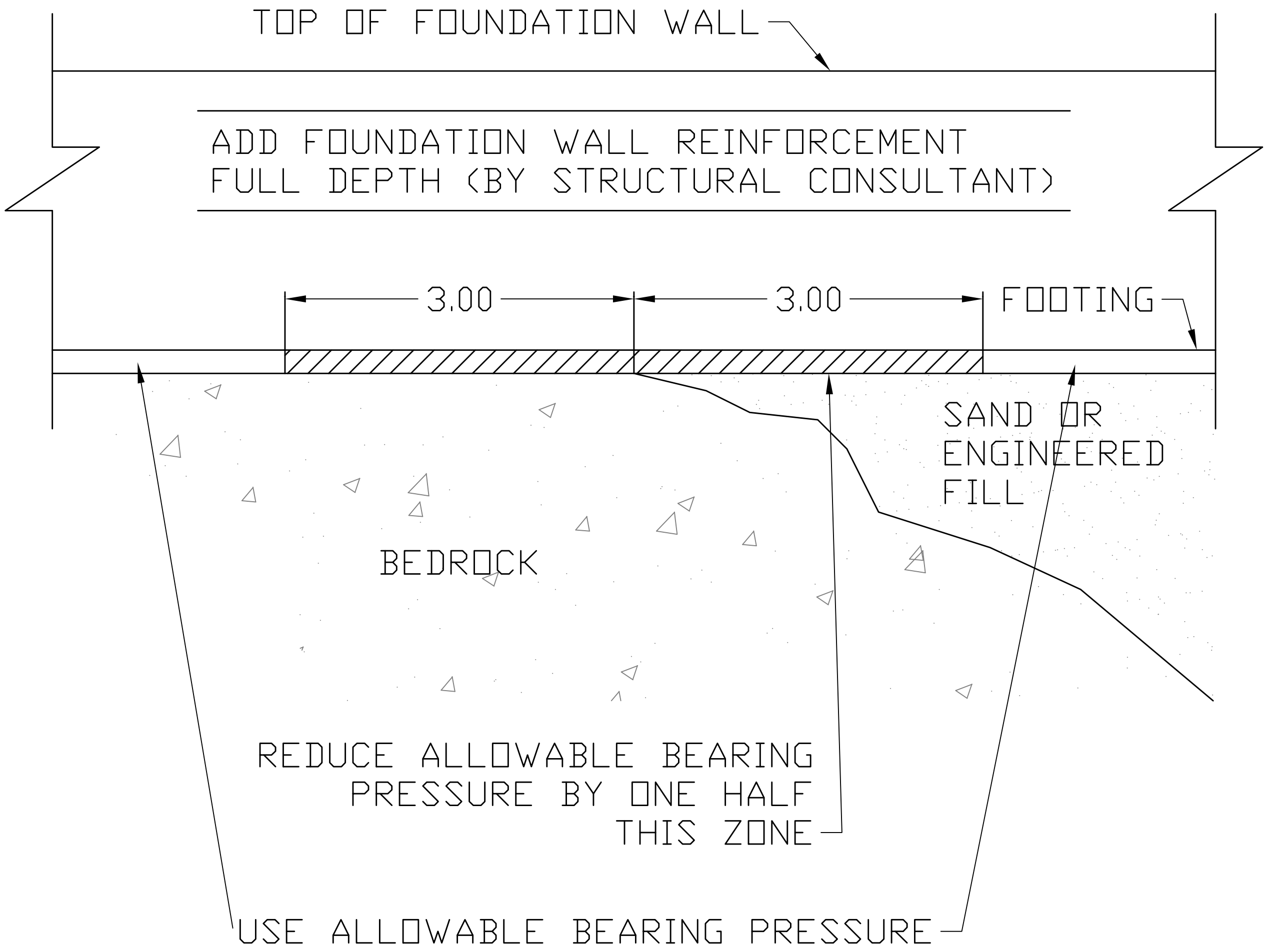
DESIGN	SD
CHECKED	SD
DRAWN	SD
CHECKED	SD
APPROVED	SD

SCALE
1: 750
0 5 10 20 30

PROJECT LOCATION	2050 DUNROBIN ROAD, CITY OF OTTAWA, ONTARIO
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

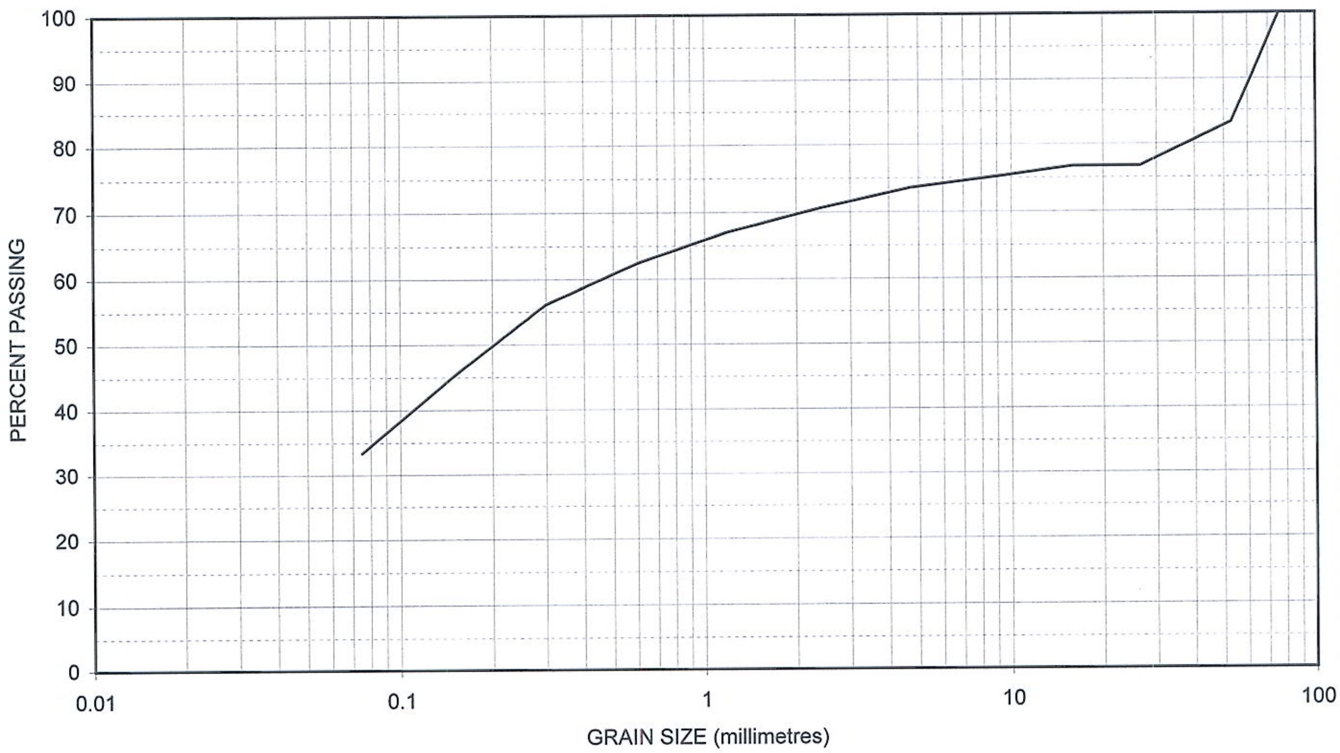
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL SUBDIVISION DEVELOPMENT 2050 DUNROBIN ROAD
PROJECT NO. 200977
DATE: NOVEMBER 2021

KOLLAARD ASSOCIATES INC.

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.

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

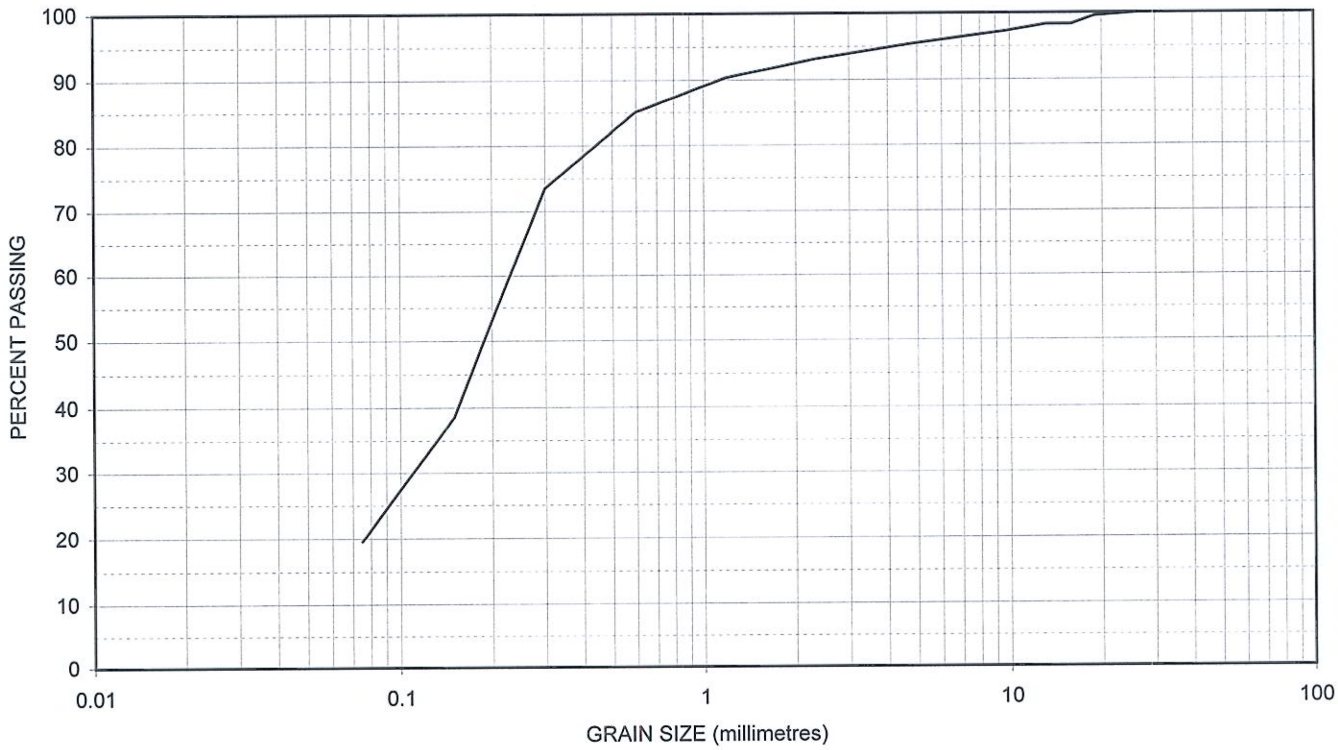
Issued by: C. R. Morey, P. Eng.

Date: October 12, 2007

Grain Size Distribution Analysis

Kollaard Associates Engineers

Test Pit 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.

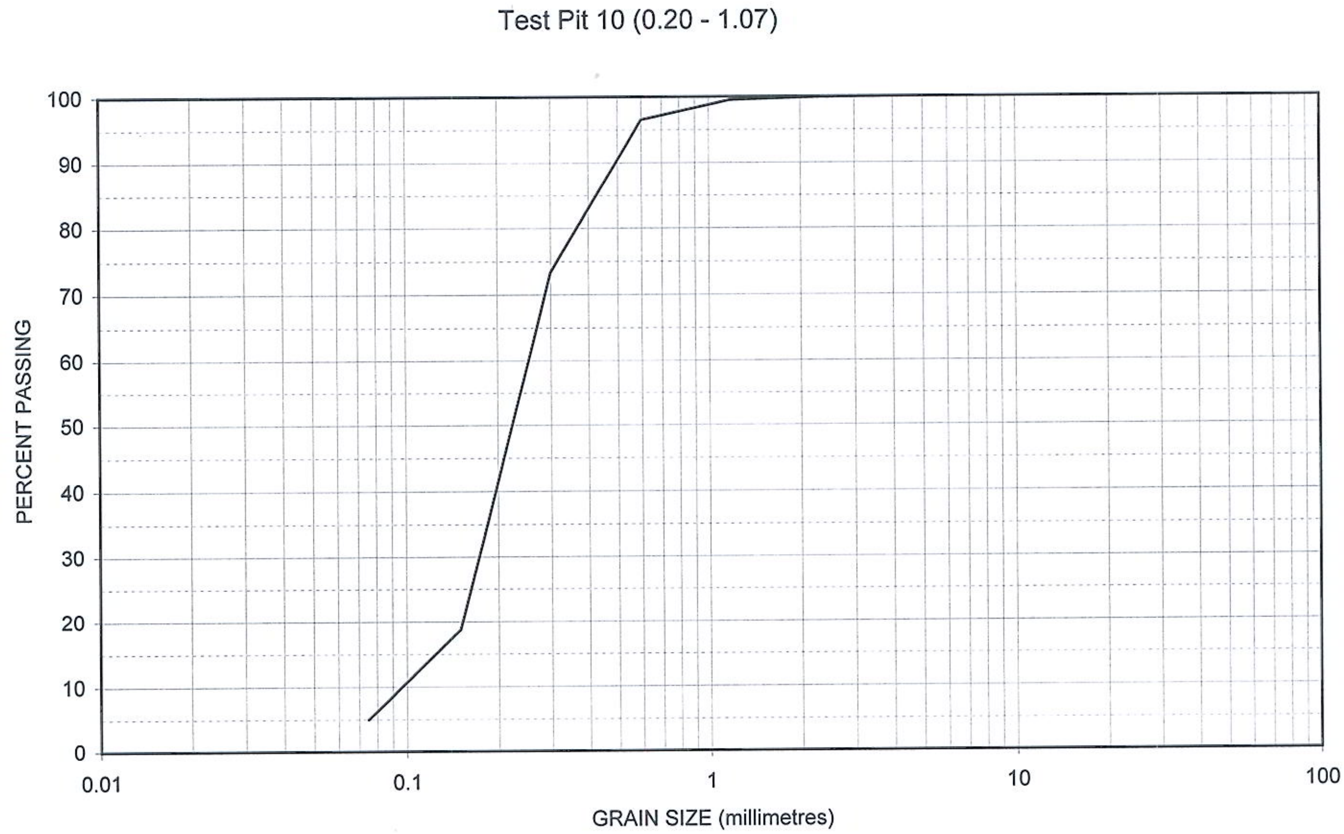
Kollaard Associates
Engineers
Box 189, 215 Sanders Street, Unit 1
Kemptville, Ontario K0G 1J0
(613) 860-0923, FAX: (613) 258-0475

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	99.9	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

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