PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL SUBDIVISION DEVELOPMENT
PARTS OF LOTS 7 & 8, CONCESSION 5
GEOGRAPHIC TOWNSHIP OF WEST HAWKESBURY
NOW TOWNSHIP OF CHAMPLAIN
VANKLEEK HILL, ONTARIO

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1 INTRODUCTION

Habitations Robert Inc. retained the services of Lascelles Engineering & Associates Ltd. (Lascelles) to conduct a preliminary geotechnical investigation for a proposed residential subdivision development located in the Village of Vankleek Hill, Ontario.

The purpose of the investigation was to identify the subsurface soil conditions within the project by means of a limited number of test pits, and based on the factual information obtained, provide preliminary guidelines on the geotechnical engineering aspects of the design of the proposed dwellings' foundation and roadways, including construction consideration which may influence the design of the subdivision development in efforts of obtaining "Draft Plan" approval.

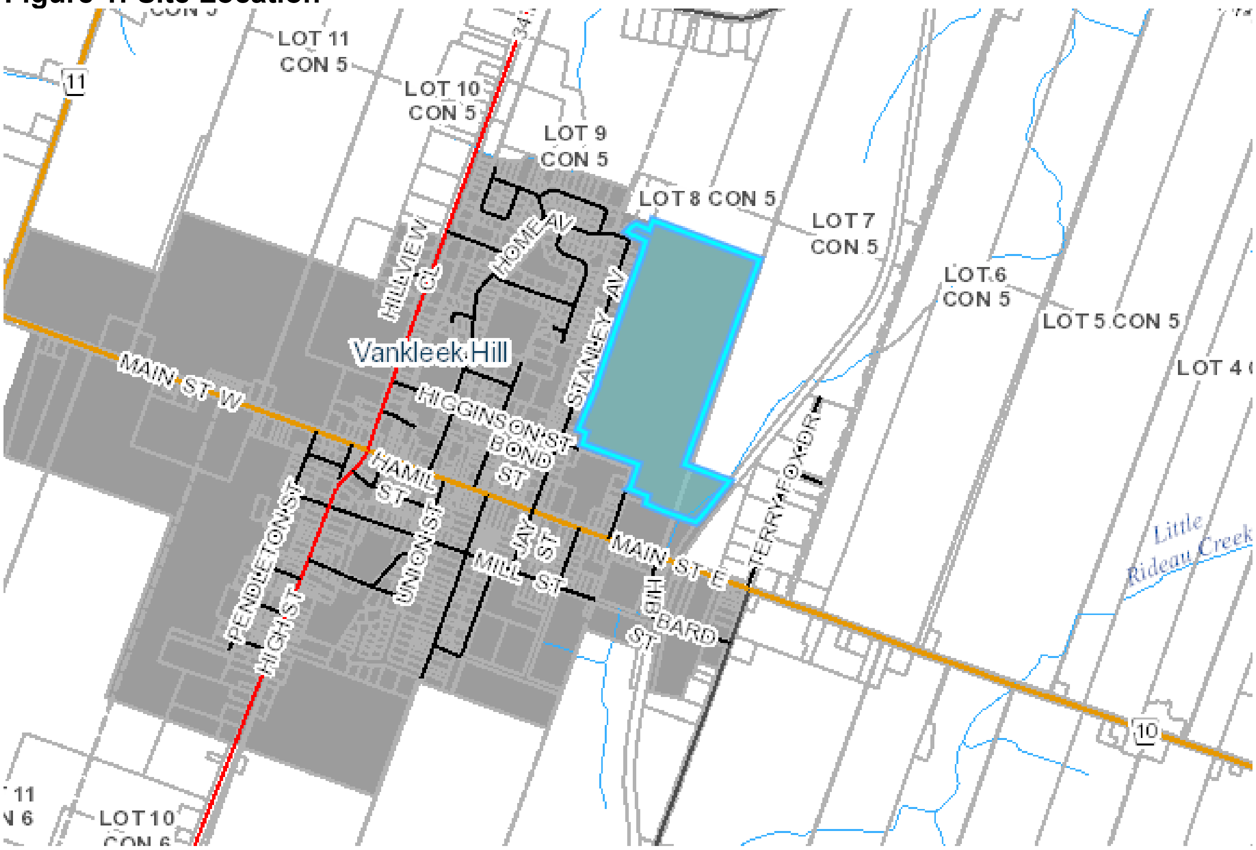
Should there be any changes in the design features, which may relate to the guidelines provided in the report, Lascelles Engineering & Associates Ltd. should be advised in order to review the report recommendations.

2 PROJECT AND SITE

The site under investigation consists of two adjacent farm lots, of which neither has a civic address. The westerly property would be set on Part of Lot 8, while the easterly lot would be mostly set on Part of Lot 7. Both parts are within Concession 5, within the Geographic Township of West Hawkesbury, now the amalgamated Township of Champlain and within the United Counties of Prescott-Russell (UCPR). The property has an irregular trapezoid shape of being approximately 662m wide (west-east) and 882m deep (north-south), for a total surface of 46.3 ha or 114.4 acres. However, the proposed subdivision would be located on Part 8, with only the proposed stormwater management facility located on Part 7. Therefore, only this (western) portion of the property designated to be part of the subdivision was investigated.

It is understood that the western portion of the property is already within the Urban designated area of the Village of Vankleek Hill, which would allow for the subdivision to be developed. The remaining (eastern) portion of the property is currently designated as Rural. Therefore, the area of the proposed subdivision will have an irregular shape of being approximately 330m wide (west-east) and 760m deep (north-south), for a total surface of 23.8 ha or 58.8 acres. Refer to **Figure 1** for location.

Figure 1: Site Location



Cyan line outlines the location of property

Ref: UCPR – GIS

It is our understanding that the project will consist of the development of an urban subdivision holding approximately 272 lots composed of single and semi-detached dwelling units. The subdivision will be serviced by municipal water and sewers. A stormwater facility in the form of a wet pond will be located in the southeast portion of the subdivision. Access to the future subdivision will be from Farmers Avenue, Higginson Street and Home Avenue. A draft plan of the subdivision prepared by Schultz Barrette Surveying is presented in **Appendix A**.

Currently, the entire property is vacant of structures, except for an old wooden equipment shed located in the central portion of the subdivision site. The property is used as farmland for the most part with its northeastern portion being forested. The subdivision property is located within a east facing sloped area with ground surface elevation ranging around Elev. 115m to Elev. 95m, which accounts for 20m of elevation change within a 300m distance. The subdivision is bordered by residential development to the west and south, by undeveloped and vacant lands to the east and north. It is noted that a railway runs along the eastern boundary of the property.

3 PROCEDURE

The fieldwork for this investigation was carried out on January 23 and 24, 2019 and consisted of digging twenty (20) test pits across the proposed subdivision property. Prior to any fieldwork, the test pit locations were cleared for the presence of any underground services and utilities. It is noted that the fieldwork was carried out in conjunction with a Phase II Environment Site Assessment (ESA), where the test pits (TP-12 and TP-16 to TP-20) located in the southeastern portion of the subdivision were carried out to confirm the presence/absence of soil and groundwater contamination. The locations of the test pits were overlaid over the draft subdivision plan prepared by Schultz Barrette Surveying, which is attached in **Appendix A**.

The test pits were completed using an excavator supplied and operated by Cliftondale Construction Co. The test pits were taken to depths ranging from 3.05m to 3.96m below ground surface (bgs). Sampling of the overburden materials encountered in the test pits was carried out by means of grab samples taken either directly from the excavation walls or from the bucket of the excavator. All soil samples collected from test pits were placed and sealed in plastic bags to prevent loss of moisture. The recovered soil samples collected from the test pits were classified based on visual and tactile examination and the results of the in-situ tests (field vane, dynamic cone penetration test, etc.). Upon completion, the test pits were backfilled with the excavated overburden materials and lightly compacted.

The fieldwork was supervised throughout by a member of our engineering staff who supervised the digging of the test pits, coordinated the testing of the materials, cared for the samples collected and logged the subsurface conditions encountered at each location. All soil samples were transported to our office for further examination by our geotechnical engineer. All samples collected during this project will be kept in storage for a period of six (6) months following the issuing of this report, where at which time, they will be disposed of, unless a written or verbal notice is received, requesting otherwise.

Six (6) 20mm diameter slotted PVC standpipes and three (3) 50mm diameter PVC monitoring wells (for the ESA) were installed in the test pits prior to backfilling them in order to measure and establish the static groundwater level.

Finally, all test pits were surveyed and located using a GPS (Global Positioning System) receiver using NAD 83 datum (North American Datum). The topographic survey was conducted using a laser level. The test pit elevations were referenced to the site benchmarks given to the center nut on top of the fire hydrant on Farmers Avenue; elevation 102.19 considered geodetic.

4 SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

4.1 General

The local geology maps were reviewed to obtain the general surficial geology and bedrock formation that would be found underlying this property. Based on our review, the surficial geology underlying this property would consist of Champlain See Sediments in the form nearshore (beaches) sediments consisting of gravel, sand and boulders, generally well sorted. The site is flanked to the north and south by glacial till deposits with Offshore Marine Deposit (silt and clay) within the eastern low-lying portion of the site. The bedrock beneath the site would belong to Lindsay Formation which is generally described as limestone with shale interbeds.

The subsurface conditions encountered in the test pits were classified based on visual and tactile examination of the materials recovered from the test holes and the results of the in-situ testing and field observations. The soil descriptions presented in this report are based on commonly accepted methods of classification and identification of soil, employed in geotechnical practice. Classification and identification of soil involves judgement and Lascelles does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The subsurface soil conditions encountered at each test pit location are given in the Test Pit Logs presented in **Appendix B**. These logs indicate the subsurface conditions encountered at specific test locations only. Boundaries between zones on the logs are often not distinct, but are rather transitional and have been interpreted.

4.2 Topsoil

A layer of topsoil (300mm to 610mm) was encountered at the surface in all test pits completed at this site. The topsoil was described as dark brown sandy loam.

The material classified as topsoil was based on colour and the presence of organic materials and is intended as identification for geotechnical purposed only. This does not constitute a statement as to the suitability of this layer for cultivation and sustaining plant growth.

4.3 Fill Material

A layer of fill (610mm) was encountered in TP-16 only, underlying the topsoil. The fill was described as medium grained sand mixed with some organics and debris (plastic bottles), brown in colour, and moist. The fill was found to extend to 0.91m bgs. TP-16 is located in the southern portion of the site, and along the south property line.

4.4 Clay

A clay deposit was encountered in TP-12 underlying the sand and gravel deposit and in TP-17 to TP-20 underlying the topsoil. The clay is located in the eastern portion of the site and at the based of the slope, where the ground surface is at its lowest and where grades fall below Elev. 90m.

The clay was described as silty, brown to greyish brown in colour, and moist. It is noted that the top of the clay layer in TP-12 and TP-20 was described as being sandy with some gravel for the first 1.83m and 0.91m respectively. This is likely an outwash of the upper till unit located at a higher elevation. The clay consistency was established to range from very stiff becoming firm with depth. All noted test pits were terminated within this soil unit at depths between 3.05m to 3.35m bgs.

A representative sample of the clay deposit was collected from TP-18 (S3) at a depth of 2.75m bgs and submitted for an Atterberg Limit analysis. The result indicate that the clay has a plastic limit of 25.2 percent, a liquid limit of 65.5 percent, for a plasticity index of 40.3. According to the Unified Soil Classification System, the clay would be classified as CH (high plasticity). The laboratory reports are presented in **Appendix C.**

4.5 Sand and Gravel

A sand and gravel deposit was encountered in TP-1 to TP-3 and TP-6, which are all located in the northern portion of the subdivision. The deposit is described as mixture of sand and gravel with traces of silt and clay, presence of seashells and some cobles with occasional boulders (Ø about 1m). It is brown in colour and within a compact to dense state of packing and moist. This deposit would likely be part of an old beach that would have washed the fines of the upper till unit located at a higher elevation. TP-1, TP-2, TP-3 and TP-6 were terminated within this deposit at depth of 3.65m bgs.

One representative sample of this soil unit was collected from TP-3 (S2 – 2.4m bgs) and submitted for a gradation analysis. The gradation analysis indicated that the sand and gravel matrix consists of 48.8 percent of gravel, 46.4 percent of sand and 4.7 percent silt and clay. According to the Unified Soil Classification System, the sand and gravel material would be GW (well-graded gravels or gravel-sand mixtures, little or no fines). The laboratory reports are presented in **Appendix C.**

4.6 Glacial Till

Glacial till deposit was encountered directly underlying the topsoil in TP-4 to TP-5, and TP-7 to TP-11 and underlying the fill in TP-16. These test pits are mostly located in the western portion of the site at a higher elevation. The till was described as silty to sandy with some gravels and trace of clay, and some cobles with occasional boulders (Ø about 1m). It is brown in colour, compact near the surface becoming very dense with depth and moist. It is noted that very large boulders were encountered. For example, TP-5 was terminated over a large boulder at 3.05m bgs. Furthermore, occasional layers or beds of either silty to coarse sand were also found within TP-4 and TP-5 at depths ranging from 0.30m to 0.91m, and from 2.13m to 3.05m bgs, respectively. TP-4, TP-5, and TP-7 to TP-11 were terminated within this glacial till unit at depths of 3.05 to 3.96m bgs.

Two representative samples of the glacial till unit were collected from TP-9 (S1) and TP-10 (S1) were submitted for a gradation analysis. The gradation analysis indicated that the till's matrix consists of 20.7 to 26.9 percent of gravel, 38.0 to 46.7 percent of sand and 26.4 to 41.3 percent silt and clay. According to the Unified Soil Classification System, the till material would be classified as GM (gravel-sand-silt mixtures). The laboratory reports are presented in **Appendix C.**

4.7 Groundwater Conditions

At the time of digging the test pits, groundwater infiltration or seepage was observed in TP-19 and TP-20 only at depths of 1.52m and 2.44m bgs, respectively. The static water level was measured within the standpipes and monitoring wells on October 21, 2019 and are provided on the test pit logs and in the table below;

Test Pit	Water Level m bgs	Test Pit	Water Level m bgs
TP-1	2.00	TP-3	2.94
TP-4	1.35	TP-5	1.33
TP-9	1.28	TP-14	1.89
TP-17	0.43	TP-18	0.29
TP-20	0.36		

Based on these levels, the inferred groundwater flow direction would be easterly to north-easterly, and would be consistent with local terrain topography. It should be noted that the groundwater table can easily fluctuate with seasonal weather conditions (i.e.: rainfall, droughts and underground service trenches or ditches at or in the vicinity of the site.

5 GEOTECHNICAL CONSIDERATIONS

5.1 General

This section of the report provides general engineering guidelines on the geotechnical design aspects of the project based on our interpretation and review of the information obtained from the test pits and the project requirements. It is our understanding that the project will consist of the development of an urban subdivision holding approximately 272 lots composed of single and semi-detached dwelling units. The subdivision will be serviced by municipal water and sewers. A stormwater facility in the form of a wet pond will be located in the southeast portion of the subdivision. Access to the future subdivision will be from Farmers Avenue, Higginson Street and Home Avenue. A draft plan of the subdivision prepared by Schultz Barrette Surveying is presented in **Appendix A**.

5.2 Foundations

The subsurface investigation revealed that the subsurface soil conditions underlying the proposed residential subdivision of the site consists sand-gravel and glacial till deposits in the upper portion of the subdivision. The clay deposit would be encountered only in the lower area of the site, outside of any proposed residential lots and only in the area of the wet pond. The approximate limits of each soil deposits are showed in the Site Plan presented in **Appendix A**. The sand-gravel and glacial till deposits are considered suitable for supporting the footings for the proposed residential dwellings once all organics and fill material are removed from the dwelling's footprint to expose undisturbed native soil.

Conventional strip and pad footings set over sand-gravel or glacial till or properly prepared and approved engineered fill, may be designed using a maximum allowable bearing pressure of **75kPa** for serviceability limit state (**SLS**) and **100kPa** for ultimate limit state (**ULS**) factored bearing resistance. The bearing capacity provided is conditional on a minimum footing width of 0.4m for strip footings and a minimum width of 0.9m for pad footings on any sides. Furthermore, it is recommended all underside of footings be located at minimum 0.3m above the groundwater table.

5.3 Settlement

Provided that any loose and/or disturbed soil is removed from the bearing surfaces prior to pouring concrete or placing of structural fill, foundations set over native soil or properly prepared and approved engineered fill designed using the recommended serviceability limit state capacity value, the total settlement will be less than 25mm. The differential settlement between adjacent column footings is anticipated to be 15mm or less.

5.4 Structural Fill

Where excavation below the underside of the footing is performed in order to reach a suitable founding stratum, considerations should be given to support the footings on structural fill. The structural fill should be placed over undisturbed native soils in layers not exceeding 200mm and compacted to 100 percent of its Standard Proctor Maximum Dry Density (SPMDD). In order to allow the spread of load beneath the footings and to prevent undermining during construction, the structural fill should extend 1m beyond the outside edges of the footings and then outward and downward at 1 horizontal to 1 vertical profile (or flatter) over a distance equal to the depth of the structural fill below the footing. The material used as structural fill to support the footings should consist of imported granular material meeting Ontario Provincial Standards Specifications (OPSS) requirements for a Granular A, or an approved equivalent material.

Prior to placing any structural fill or pouring the footings, any disturbed soils along the base within the footings' area should be removed and that the subgrade soils should be inspected and approved by the geotechnical engineer. The structural fill should be tested to ensure that the specified compaction level is achieved.

5.5 Seismic Design

Based on the results of the geotechnical investigation, the subsurface at this property can be classified as a Class “D” as per the Site Classification for Seismic Site Response in accordance with the latest version of the Ontario Building Code. It is noted that a greater seismic site response class may be obtained by carrying out seismic velocity testing using a multichannel analysis of surface waves (MASW).

5.6 Potential for Soil Liquefaction

Based on the characterisation of the subsurface soil conditions established at this site and the recommendations provided herein, the potential of soil liquefaction is not considered to be a concern.

5.7 Slab-on-Grade Construction

For predictable performance of proposed concrete slab-on-grade, it is recommended that they rest over native soil or structural fill only. Therefore, all organic, deleterious or otherwise objectionable fill material encountered shall be removed from the building's footprint. The adequacy of the existing fill found within the footprint of the building and if it can be left underneath the concrete slab-on-grade will be determined at the time of construction by a geotechnical engineer, once it is fully exposed. Regardless, the existing fill will need to be properly compacted using appropriate equipment should it be determined to be adequate.

The exposed native subgrade surface should then be inspected and approved by geotechnical personnel. Any evidently soft areas should be sub-excavated and replaced with suitable engineered fill however disturbances should be minimized as much as possible.

Any underfloor fill needed to raise the general floor grade shall consist of OPSS Granular B Type I material or an approved equivalent, compacted to 95 percent of its SPMDD. The final lift shall be compacted to 98 percent of its SPMDD. A 200mm layer of OPSS Granular A material shall be placed under the slab and compacted to at least 100 percent of the SPMDD.

In order to minimize and control cracking, the floor slab should be provided with wire or fiber mesh reinforcement and crack control joints. The crack control joints should be spaced equal distance in both directions and where possible not exceeding a spacing of 4.5 metres. The mesh reinforcement should be carried through the joints.

5.8 Frost Protection

All exterior footings, and those located in any unheated portion of the proposed building should be provided with at least 1.5m of earth cover for frost protection purposes. Exterior footings constructed in areas that are to be cleared of snow during the winter period should be provided with at least 1.7m of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided using a combination of earth cover and extruded polystyrene insulation. Lascelles should review the detailed design of frost protection with the use of equivalent insulation prior to construction.

In the event that foundations are to be constructed during winter months, foundation soils are required to be protected from freezing temperatures using suitable construction techniques. Therefore, the base of all excavations should be insulated from freezing temperature immediately upon exposure, until the time that heat can be supplied to the building interior and footings have sufficient soil cover to prevent freezing of the subgrade soils.

5.9 Foundation Drainage

It is anticipated that the dwellings will have basements and will therefore, require permanent perimeter drainage. The drainage pipe shall be embedded in a 300mm layer of 20mm diameter clear crushed stone wrapped in a geotextile and set adjacent to the perimeter footings. The drainage pipe should be connected positively to a suitable outlet such as a sump pit or storm sewer. In order to reduce the potential for ponding of water adjacent to the foundation walls, roof water should be controlled by a roof drainage system that directs water away from the building and the exterior grade should be sloped to promote water away from the foundation walls.

5.10 Foundation Wall Backfill

To prevent possible foundation frost jacking of foundation wall, the backfill material should consist of free draining, non-frost susceptible material such as sand or sand and gravel meeting OPSS Granular B Type I grading requirements.

The foundation fill should be compacted to 90 percent of its SPMDD using light compaction equipment, where no loads will be set over top. The compaction shall be increased to 95 percent under walkways, slabs or paved areas close to the foundation or retaining walls. Backfilling against foundation walls should be carried out on both sides of the wall at the same time.

5.11 Retaining Walls and Shoring

The following **Table 1** below provides the suggested soil parameters for the design of retaining wall and/or shoring systems. For excavations near existing services and structures, the coefficient of earth pressure at rest (K_o) should be used.

Table 1: Material Properties for Shoring and Permanent Retaining Wall Design (Static)

Type of Material	Bulk Density (kg/m^3)	Pressure Coefficient	
		Active (Ka)	At Rest (Ko)
Clay	18	0.45	0.80
Sand	19	0.33	0.50
Till	22	0.27	0.50
Granular B Type I	20	0.33	0.50
Granular B Type II	23.1	0.31	0.47
Granular A	23.5	0.27	0.43

The above values are for a flat surface behind the wall, a straight wall and a wall friction angle of 0 degrees. The designer should consider any difference between these coefficients, and make appropriate corrections for a sloped surface behind the wall, angled wall or wall friction as required. The bearing capacity for the design of a retaining wall are the same as provided for the building structures provided it is founded over native soil or properly prepared and approved structural fill.

Retaining walls should also be designed to resist the earth pressures produced under seismic conditions. Lascelles Recommends the use of combined coefficients of static and seismic earth pressure, referred to as K_{AE} for active conditions and K_{PE} for passive conditions for routine design purposes.

The total active and passive loads under seismic conditions can be calculated using the following two equations;

$P_{AE} = ½ K_{AE}\, γ\, H^2 (1-k_V)$
$P_{PE} = ½ K_{PE}\, γ\, H^2 (1-k_V)$

Where;
K_{AE} = Combined Static and Seismic Active Earth Pressure Coefficient
K_{PE} = Combined Static and Seismic Passive Earth Pressure Coefficient
H = Total Height of the Wall (m)
K_h = horizontal acceleration coefficient
K_v = vertical acceleration coefficient
γ = bulk density (kg/m^3)

These equations are based on a horizontal slope behind the wall and a vertical back of the retaining wall and zero wall friction. For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values.

A = Zonal acceleration ratio = 0.30
K_h = Horizontal acceleration coefficient = 0.10
K_V = Vorizontal acceleration coefficient = 0.067

The above value of K_h corresponds to ½ of the A value and the value K_V of corresponds to 0.67 of the K_h value. The angle of friction between the soil and the wall has been set at 0^o to provide a conservative estimate. The following **Table 2** provides the parameters for seismic design of retaining structures.

Table 2: Material Properties for Shoring and Permanent Retaining Wall Design (Seismic)

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II
Bulk Unit Weight, γ (kN/m^3)	20	23.3
Effective Friction Angle (degrees)	30	32
Angle of Internal Friction Between wall and Backfill (degrees)	0	0
Yielding Wall		
Active Seismic Earth Pressure Coefficient (K_{AE})	0.37	0.33
Height of the Application of P_{AE} from the base of the wall as a ration of its height (H)	0.36	0.37
Passive Seismic Earth Pressure Coefficient (K_{PE})	3.06	3.48
Height of the Application of P_{PE} from the base of the wall as a ration of its height (H)	0.30	0.30

6 Potential of Corrosive Environment

6.1 Sulphate Attack on Buried Concrete

One (1) sand-gravel and three (3) glacial till samples were sent to Paracel Laboratories Ltd., an accredited chemical analysis laboratory, for analysis of sulphate content within the overburden. The results of the analysis found a sulphate concentration ranging from less than 5 to 127 µg/g (<0.0005 to 0.0127%). The Laboratory Certificates of Analysis is presented in **Appendix D**.

Based on the CAN/CSA – A23.1 standards (Concrete Materials and Methods of Concrete Construction), a sulphate concentration of 0.1% (1000 µg/g) or less in soil falls within the negligible category for sulphate attack on buried concrete. As such, buried concrete for footings and foundation wall will not require any special additive to resist sulphate attack and the use of normal Portland cement is acceptable.

6.2 Sulphate Attack on Buried Concrete

The same samples were also submitted for analysis of pH, Resistivity and Redox Potential. The purpose of this testing was to assess the potential for corrosive environment on any buried steel. The Laboratory Certificates of Analysis are presented in **Appendix D**.

The potential for an aggressive corrosive soil environment was established in reviewing the above measured parameters and according to standard provided by the American Water Works Association (AWWA) C-105/A21.5-10. Based on the noted standard, corrosion protection for buried steel is only required where a corrosivity index of 10 or greater is encountered. Based on the results, the calculated corrosivity in index was found to be all below 5. As such, any buried steel as part of this project would not require any special or specific corrosion protection measures with respect to cast iron pipes.

7 Excavation and Groundwater Control

The detailed civil design for the subdivision is not completed at the time of writing this report. Consequently, it is anticipated that shallow excavation in overburden soil would not exceed 3.6m bgs for the proposed dwellings and municipal services. Most of the shallow excavation will be through sand-gravel or glacial till deposits.

According to the Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 213/91 and its amendments, the surficial overburden soil anticipated to be excavated into at this site can be classified as Type 3 for fully drained excavations. Therefore, shallow temporary excavation in the overburden soil classified as Type 3 can be cut at 1 horizontal to 1 vertical for a fully drained excavation starting at the base of the excavation and as per requirements of the OHSA regulations.

The listed slopes are for fully drained excavations. Gentler slopes could be required under undrained excavations or below the water table, where localised water infiltrations can occur and where the excavations are exposed for a prolonged period of time.

Any excavated material stockpiled near a trench or open excavation should be stored at a distance equal to or greater than the depth of the excavated soil within the trench or open excavation and equipment circulation should be restricted away from the top of the slope excavation.

In the event that the aforementioned slopes are not possible to achieve due to space restrictions, the excavation should be shored according to OHSA O. Reg. 213/91 and its amendments. A geotechnical engineer should design and approve the shoring and establish the shoring depth under the excavation profile. Refer to the parameters provided in Tables 1 and 2 in Section 5.11 for use in the design of any shoring structures. The excavation for the underground services could be carried out within tightly fitting, braced steel trench boxes, approved by a professional engineer.

No bedrock excavation is anticipated to be required for the installation of the underground services at this site. However, it is possible that large boulders (greater than 1m in size) may be encountered as part of the glacial till, and may need to be broken up in order to excavate.

A condition survey of any nearby structures and services should be undertaken prior to commencing any construction. In view of the potential for ground source vibration generated by heavy equipment during the construction activities, it is recommended that the excavation activities be monitored throughout the project by a vibration specialist engineer or consultant and that the vibration limits be established based on the local conditions and nearby structures to ensure that ground vibration are not exceeded.

7.1 Groundwater Control

Groundwater seepage and infiltration entering into shallow and temporary excavations performed within the overburden should be mitigated by pumping from sumps installed in the excavation. Surface water runoff into the excavation should be avoided and diverted away from the excavation.

It is anticipated that the invert of underground services may be founded below the water table. Although the sand-gravel and glacial till are compact to dense, they are nevertheless sensitive below the water table and may also be susceptible to piping and scouring from water pressure at the base of the excavation. Therefore, the base of the excavation should not be exposed for prolonged periods of time and should be backfilled as soon as possible.

It should be noted that large volume pumps may be required in the more pervious sand-gravel deposit, where the hydraulic conductivity could be as high 10^{-3} cm/s, such as in the northern portion of the site and depending on the depth and extent of the excavations. This will need do be reviewed once the final design of the municipal services is completed.

7.2 Pipe Bedding Requirements

Bedding, thickness of cover material and compaction requirements for the underground services should conform to the manufacturers design requirements and to the requirements and detailed installations outlined in the Ontario Provincial Standard Specifications (OPSS) and any applicable standards or requirements from the Town of Champlain.

It is recommended that the bedding for any underground service be placed over native material or structural fill only. Consequently, any fill or organic material should be removed from the loading influence of the proposed underground service.

Where the invert of an underground service will be founded below the groundwater table, the overburden may be sensitive to disturbances and may also be susceptible to piping and scouring from water pressure at the base of the excavation. Therefore, special precautions should be taken in these areas to stabilize and confine the base of the excavation such as using recompression (thicker bedding) and/or dewatering methods (pre-pumping). In order to properly compact the bedding, the water table should be kept at least 0.30m below the base of the excavation at all time during the installation of the underground services.

As an alternative to Granular A bedding and only where wet conditions are encountered, the use of “clear stone” bedding, such as 19mm clear stone, OPSS 1004, may be considered only in conjunction with a suitable geotextile filter. Without proper filtering, there may be entry of fines from native soils and trench backfill into the bedding, which could result in loss of support to the pipes and possible surface settlements.

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

7.3 Trench Backfill

Acceptable and compactable native materials should be used as trench backfill between the roadway subgrade level and the depth of seasonal frost penetrations (i.e. 1.8m below finished grade). In order to reduce the potential for differential frost heaving between the new excavated trench and the adjacent section of roadway, the selected trench backfill material should match, as much as possible, the existing soil exposed on the trench walls. Any boulders larger than 300mm in size should not be used as trench backfill. Where there is lack of backfill material and it would need to be imported, it should conform to OPSS Granular B Type I or approved equivalent.

Where two different frost susceptible soil types are used in the trench backfill, frost tapers should be provided. The minimum frost taper should consist of cutting back the side slope of the trench to 3 horizontal to 1 vertical profile starting at 1.2m below the finish grade.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadway, the trench should be compacted in maximum 300mm thick lifts to at least 95 percent of the SPMDD. The specified density may be reduced where the trench backfill is not located within or in close proximity to existing roadways or any other structures.

8 REUSE OF ON-SITE SOILS

The existing overburden found at this site consist of sand-gravel and glacial till as well as clay in the eastern portion of the site. The glacial till and silty clay are considered frost susceptible and are not recommended for engineered fill or backfilling against foundation wall. It is however possible to use these soils for landscaping and general backfilling purposes. Any stones, cobbles or boulders larger than 300mm in size should not be used as service trench backfill. The sand-gravel deposit was found to contain a low silt content and could therefore, we used as backfill material depending on this intendent use.

It should be noted that the adequacy of a material for reuse as backfill will depend on its water content at the time of its use and on the weather conditions prevailing prior and during that time. Therefore, all excavated materials to be reused should be stockpiled in a manner that will minimise any significant changes in its moisture content, especially during wet conditions. Any excavated materials proposed for reuse as part of this project should be stockpiled in order to allow the material to be properly inspected and approved prior to reuse by a geotechnical engineer.

9 PAVEMENT DESIGN

For predictable performance of the pavement areas, any organic, soft or deleterious materials should be removed from the proposed pavement areas to expose undisturbed subgrade soil. The exposed subgrade should be inspected and approved by geotechnical personnel and any evidently loose and unstable areas should be sub-excavated and replaced with suitable earth borrow approved by the geotechnical engineer. The subgrade should be shaped and crowned to promote drainage of the roadway. Following approval of the preparation of the subgrade, the granular subbase may be placed.

It is anticipated that the subgrade soils for the new parking and access road will consist mostly of sand-gravel and glacial till. The recommended pavement structure for the proposed municipal street should consist of:

40 millimetres of hot mix asphaltic concrete surface layer (HL3 – PG 58-34) over
50 millimetres of hot mix asphaltic concrete binder layer (HL8 – PG 58-34) over
150 millimetres of OPSS Granular A base over
300 millimetres of OPSS Granular B, Type II subbase

The base and subbase granular materials should conform to OPSS Form 1010 material specifications. Prior to importing any granular material onto the site, it should be tested and approved by a geotechnical engineer prior to delivery to the site and should be compacted to 100% SPMDD. Compaction of the granular pavement materials should be carried out in maximum 200 mm thick loose lifts to 100% of its SPMDD using suitable vibratory compaction equipment.

The Job Mix Formula (JMF) of the asphaltic concrete should be in accordance with OPSS 1150 for Material Specification for Hot Mix Asphalt. The asphaltic concrete should be placed in accordance to OPSS 310 for Construction Specification for Hot Mix Asphalt. The asphaltic concrete should compacted to a minimum of 92% of the Maximum Relative Density. The JMF and its constituents should be reviewed, tested and approved by a geotechnical engineer prior to delivery to the site.

9.1 Paved Areas and Subgrade Preparation

The proposed access lanes and parking areas should be stripped of vegetation, topsoil, fill, debris and other obvious objectionable material. Following the backfilling and satisfactory compaction of any underground service trenches up to the subgrade level, the subgrade should be shaped, with any resulting loose areas sub-excavated down to an adequate bearing layer and replaced with approved backfill. Following approval of the preparation of the subgrade, the pavement structure may be placed.

For areas of the site that require the subgrade to be raised, the material should consist of OPSS Granular B Type 1 or approved equivalent. Any materials proposed for this use should be approved by the geotechnical engineer before placement. Materials used for raising the subgrade to the proposed roadway subgrade level should be placed in maximum 300 mm thick loose lifts and be compacted to at least 95% of the SPMDD using suitable compaction equipment.

If the roadway subgrade is disturbed or wetted due to construction operations or precipitation, the granular thicknesses give above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or incorporate a non-woven geotextile separator between the roadway subgrade surface and the granular subbase material.

The preparation of subgrade should be scheduled and carried out in such a manner that a protective cover of overlying granular material is placed as quickly as possible in order to avoid unnecessary circulation by heavy equipment over the subgrade. Frost protection of the surface should be implemented (i.e. insulated tarps, etc.), if works are carried out during the winter months.

Transitions should be constructed between new and existing pavement structures where the new street will meet with the existing street. In areas where the new pavement structure will abut existing pavement structure, the depths of granular materials should be tapered 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 performance of the pavement structure is highly dependent on the subsurface groundwater conditions and maintaining the subgrade and pavement structure in a dry condition. To intercept excess subsurface water within the pavement structure granular materials, sub-drains with suitable outlets should be installed below the pavement structure subgrade, if adequate overland flow drainage is not provided (i.e. ditches). The surface of the pavement should be properly graded to direct runoff water towards suitable drainage features. It is recommended that the lateral extent of the subbase and base layers not be terminated vertically immediately behind any proposed the curb/edge of pavement line but be extended beyond the curb.

10 CONSTRUCTION CONSIDERATIONS

The current report is considered preliminary in nature and was prepared to provide general characterisation of the soil and groundwater condition across the subdivision by means of a limited number of test pits. The purpose of the investigation was to identify any potential constraints and to provide general guidelines on the geotechnical engineering aspects for the design of the subdivision in support in obtaining “Draft Plan” approval. Considering the size of the subdivision and that it will be developed in several phases over the course of several years, it is recommended that a specific geotechnical investigation be carried out on a per phase basis to supplement and confirm the findings and recommendations provided as par of this preliminary investigation.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed development do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. All footing areas and any engineered fill areas (if required) for the proposed project should be inspected by Lascelles Engineering and Associates Ltd. to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundations (if required) should be inspected to ensure that the materials used conform to the gradation and compaction specifications.

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

11 Report Conditions and Limitations

It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document is neither intended nor authorized by Lascelles Engineering & Associates Ltd. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible subsurface contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report.

The recommendations provided in this report are based on subsurface data obtained at the specific test locations only. Experience indicates that the subsurface soil and groundwater conditions can vary significantly between and beyond the test locations. For this reason, the recommendations given in this report are subject to a field verification of the subsurface soil conditions at the time of construction.

The report recommendations are applicable only to the project described in the report. Any changes to the project will require a review by Lascelles Engineering & Associates Ltd., to ensure compatibility with the recommendations contained in this report.

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

Yours truly,
Lascelles Engineering & Associates Ltd.

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100524774
November 28, 2019
PROVINCE OF ONTARIO

Reviewed by:

Mario Elie, Project Manager

Appendix A

Test Pit Location & Draft Plan of Subdivision

Appendix B

Test Pit Logs

Appendix C

Laboratory Reports

Appendix D

Laboratory "Certificates of Analysis"