GEOTECHNICAL INVESTIGATION

PROPOSED DEVELOPMENT
4980 SUNSET DRIVE, PORT STANLEY

LDS PROJECT NO. GE-00667

MARCH 31, 2022

Submitted to:

WASTELL HOMES

DISTRIBUTION (VIA EMAIL):

Wastell Homes – Julian Novick, P. Eng. julian@wastell.ca

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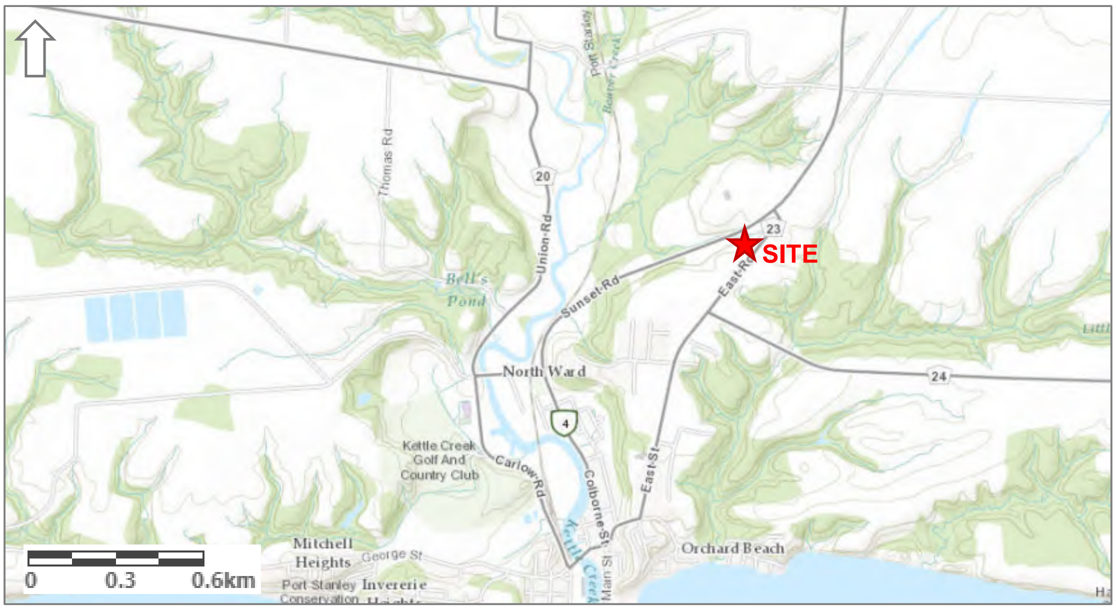
Appendices

1. INTRODUCTION

LDS Consultants Inc. (LDS) has been retained to conduct a Geotechnical Investigation and Preliminary Hydrogeological Assessment for a proposed commercial development. The subject lands are located on the southwest corner of the intersection of Sunset Drive and East Street, on the northeast end of the community of Port Stanley, Municipal Number (MN) 4980 Sunset Drive. A Key Plan showing the site location is provided on Figure 1, below.

Figure 1: Key Plan



It is understood that the proposed development will include the construction of four one-storey commercial buildings, with associated surface parking. The site is expected to be serviced with municipal services, and accessed from an internal roadway which will connect to Sunset Drive along the northern limits of the site. A Preliminary Concept Plan is provided on Drawing 1, in Appendix A.

1.1 Terms of Reference

This document has been prepared for the purposes of providing geotechnical and hydrogeological comments and recommendations for the design and construction of a proposed commercial development located at MN 4980 Sunset Drive, on the northeast end of the community of Port Stanley. The scope of work for this investigation was outlined in LDS' email proposal, dated January 25, 2022. Authorization to complete this Investigation was received from Mr. Julian Novick, of Wastell Homes, on January 25, 2022.

This report provides a summary of the borehole findings (documenting soil and groundwater conditions at the site). The report provides geotechnical comments and recommendations for the proposed commercial development, including: site preparation (including the re-use of excavated materials as engineered fill, structural fill, and guidance for engineered fill placement, and preliminary guidance on the new Excess Soil Management regulation [O.Reg. 406]), temporary excavations (including maximum slope inclinations to provide stable excavation side slopes in accordance with OHSA requirements), excavation support (shoring methods, (if required), groundwater control (including the need for a Permit to Take Water (PTTW) or Environmental Activity and Sector Registry (EASR) submission for construction dewatering, foundation design (including soil bearing capacity, subgrade preparation, and potential settlements), slab on grade construction (including lateral earth pressures, and foundation backfilling), site servicing (including the re-use of onsite soils in service trenches), pavement design (including pavement component thicknesses for local roads and reinstating service connections which extend into the municipal right-of-way) and slope stability review and analysis.

As noted, the report also provides information about the characterization of the hydrogeological setting for the site, including: characterization of the hydrologic and hydrogeological setting, a summary of MECP well records within 500 m of the site, a discussion of the potential effects on shallow groundwater at the site and on the natural features (wooded areas and valley slopes on the south side of the site), as it relates to the proposed construction, and stormwater management considerations (including factored soil infiltration rates and a discussion of limitations which result from soil and/or shallow groundwater conditions.)

This report is provided on the basis of the terms noted above, and on the assumption that the design will follow applicable codes and standards. The format and content of this report has been guided to address specific client needs. The site investigation and recommendations provided in this report follow generally accepted practice for geotechnical and hydrogeological engineering consultants in Ontario.

The format and content of this report has been guided to address specific client needs. LDS has provided engineering guidelines for the geotechnical design and construction at the site.

Laboratory testing, where applicable, follows ASTM or CSA Standards.

1.2 Qualifications of Assessor

The program which was undertaken for this project was conducted under the supervision of Rebecca Walker, P. Eng., QPESA. She has been thoroughly trained in conducting geotechnical and hydrogeological assessments. Mrs. Walker is a licensed professional engineer in the Province of Ontario. She obtained a Bachelor of Applied Science in Geological Engineering from Queen's University in 1998 and is a Qualified Person (QP) registered with MECP. She has been practicing geoscience services under the Guideline of Professional Engineers Providing Geotechnical Engineering Services under the Professional Engineers Act in Ontario.

Mrs. Walker has over 20 years of direct experience in the geotechnical and hydrogeological consulting industry. Over 3,600 projects have been completed under her supervision. Mrs. Walker is also a recognized expert in the industry and has testified as an expert witness in Local Planning Appeal Tribunal (formerly Ontario Municipal Board) hearings and Municipal Councils related to groundwater hydrogeology and geotechnical matters for land development and construction. She has been retained for many projects, both directly and indirectly (as a subconsultant) by local municipalities as a hydrogeological and geotechnical consultant.

2. SITE CHARACTERIZATION

2.1 Site Description, Topography and Surface Drainage

A review of aerial photographs dating from 2005 to current indicates that the site has contained a restaurant, which fronts on Sunset Drive in the northwest corner of the site, with associated site parking. The remaining lands have contained an open field, with a mixture of vegetation and small to medium sized mature trees. A private gravel laneway transects the site, connecting the restaurant area to the rest of the site. Sometime during 2006, some earthworks were performed at the site, including the re-routing of the private laneway, and the removal of several trees from the western half of the lot. Between 2009 and 2013, it appears that the eastern half of the site was used as a boat storage yard. The site is irregular in shape, and comprises a total area of approximately 2.7 hectares.

The grade of Sunset Drive is set slightly above the ground surface at the site, and a roadside drainage ditch is present between the roadway and the northern property limits. Some snow accumulation was observed within the drainage ditch during the site visits conducted by LDS. The site is located at the base of a vell vegetated slope, which runs along the southern limits of the site, and is connected to East Road which runs along the top of the slope. From a topographical perspective, the ground surface exhibits a relief of 15 meters from the top of the slope to the north/northwest, towards Sunset Drive. Any minor surface flows which occur at the site under existing conditions, are generally expected to follow the topography of the site. The Site is connected to the wetlands located on the north side of Sunset Drive (MN 5043) by a drain which crosses beneath Sunset Drive and terminates within the site limits.

The site is bordered by commercial lands to the west, Sunset Drive to the north, a newly constructed fire station to the east, and a residential dwelling/East Road to the South. The locations of the aforementioned site features, are highlighted on Drawing 2, in Appendix A.

2.2 KCCA Generic Regulation

In May 2006, Ontario Regulation 181/06 came into effect in the Kettle Creek Conservation Authority (KCCA) watershed, which locally implements the Generic Regulation (Development, Interference with Wetlands and Alterations to Shoreline and Watercourses). This regulation replaces the former Fill, Construction and Alteration to Waterways regulations, and is intended to ensure public safety, prevent property damage and social disruption, due to natural hazards such as flooding and erosion. Ontario Regulation 181/06 is implemented by the local Conservation Authority, by means of permit issuance for works in or near watercourses, valleys, wetlands, or shorelines, when required.

A large portion of the subject lands are identified as being within the KCCA Regulated Area. These limits are shown on Drawing 3, in Appendix A. KCCA should be consulted on any proposed developments within the subject lands, to confirm if their Generic Regulation applies, and to identify if a Section 28 permit is required for construction activities at the site.

2.3 Source Water Protection Mapping

Where proposed developments are being planned, it is important to determine the presence of Significant Groundwater Recharge Areas and High Vulnerability Aquifers in the area. These areas are protected under the Clean Water Act (2006).

LDS has reviewed the MECP Source Water Protection Information to determine whether the site is located in any identified areas of source water concern, as they relate to local groundwater quality (current to June 16, 2021). The property is located within the Kettle Creek Source Protection Area, and the following observations are noted for the site:

- The Property is not located in any of the following designated areas listed in the MECP Source Protection mapping:
    - Wellhead Protection Area, Wellhead Protection Area E (GUDI), Wellhead Protection Area Q1 or Wellhead Protection Area Q2;
    - Intake Protection Zone or Intake Protection Zone Q;
    - Highly Vulnerable Aquifer;
    - Issue Contributing Area; and,
    - Event Based Area.
- The Property is located within a Significant Groundwater Recharge Area. This is demonstrated on Drawing 4, in Appendix A.

In general, Significant Groundwater Recharge Areas are defined as areas where water seeps into an aquifer from rain and melting snow, supplying water to the underlying aquifer. Groundwater recharge is largely controlled by soil conditions, and typically occurs in upland areas. As defined in the Clean Water Act (2006), an area is a significant groundwater recharge area if,

- the area annually recharges water to the underlying aquifer at a rate that is greater than the rate of recharge across the whole of the related groundwater recharge area by a factor of 1.15 or more; or,
- the area annually recharges a volume of water to the underlying aquifer that is 55% or more of the volume determined by subtracting the annual evapotranspiration for the whole of the related groundwater recharge area from the annual precipitation for the whole of the related groundwater recharge area.

The Source Water mapping indicates that this part of the Significant Groundwater Recharge Area has a vulnerability rating of 0, which is indicative of a low groundwater vulnerability. Regardless, it is recommended that development at the site incorporate suitable measures to maintain water quality, and measures which would allow for post-development infiltration to occur.

2.4 Review of Geological Mapping

Select geological mapping and publications were reviewed for the purposes of reviewing regional characteristics for soil conditions in the area of Port Stanley, Ontario. Findings are summarized below, for reference.

Physiography & Quaternary Geology

Physiographic mapping for Southwestern Ontario *(Chapman, L.J. and Putnam, D.F. 2007. Physiography of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 228)*, identifies that Port Stanley is located within the western extent of the physiographic region known as the Norfolk Sand Plain, and is set within a broad sand plain. Natural subgrade soil conditions are expected to be comprised of predominantly sand and silty sand soils.

Quaternary geology mapping for the Port Stanley area *(Quaternary Geology, Ontario Geological Survey Map, 1985, Port Stanley Area, Scale 1:50,000)* indicates that the study area predominantly consists of glaciolacustrine sand deposits, comprised of fine to coarse grained sand, from the Late Wisconsin glaciation period. An excerpt from the aforementioned mapping is provided on Drawing 5, in Appendix A.

Bedrock Geology

Bedrock geology mapping for Southwestern Ontario *(Ontario Geological Survey. 1:250 000 scale, Bedrock Geology of Ontario. Ontario Geological Survey, Miscellaneous Release Data 126, Revised 2006)* indicates that bedrock in the general area consists of limestone, dolostone and shale from the Marcellus Formation, from the Middle Devonian Period.

Geological publications and well records in the area indicate that the bedrock surface is below 86-101 m of overburden soils in the vicinity of the site. Bedrock was not encountered during the fieldwork for this investigation.

2.5 MECP Well Record Review

A review of local well records available through the Ministry of Environment, Conservation, and Parks (MECP) for this area was carried out to review the water levels recorded in the nearby wells. Drawings C1 and C2 in Appendix C show the location of the wells (with corresponding Well Registration No.) which are in close proximity (within 500 m) of the site. The well records are summarized in Appendix C, for reference.

The majority of the water supply wells noted in the well records are set into the deep (>30 m depth) or intermediate (15 – 30 m depth) overburden aquifers, with reported static water levels ranging from 3.1 to 33.5 m, and 18.3 to 20.4 m, respectively. One water supply well noted in the records (located ~50 m northeast of the site) is set in the shallow (<15 m depth) sandy overburden aquifer, with a reported static water level of 4.3 m.

3.0 SUMMARIZED CONDITIONS

3.1 Field Program and Laboratory Testing

LDS field staff and the drilling contractor carried out a Safety Meeting prior to drilling at the site, which included a review of the underground utility locates were completed through Ontario-One-Call in preparation for the drilling program

LDS carried out a field program consisting of a series of boreholes, drilled on February 10, 2022. The boreholes were advanced at the site by a local drilling-contractor, using a track-mounted drill-rig. Four boreholes (denoted as BH1 through BH4) were advanced to a depth of 5.0 m (16.5 feet) below existing grade. The fieldwork was supervised by members of LDS' technical staff.

Ground surface elevations at the borehole locations were surveyed by LDS using a Trimble R10 GPS rover. The location of the boreholes is summarized below, and illustrated on Drawing 6, in Appendix A.

Table 1 – Borehole Locations

Location	Northing, m N	Easting, m E	Ground Surface Elevation (m asl)
BH1/MW	4725503.24	483268.71	201.39
BH2	4725522.19	483324.16	201.47
BH3/MW	4725544.22	483389.19	202.07
BH4	4725570.02	483445.51	203.39

Monitoring wells were installed in two of the boreholes (BH1 and BH3) to allow for monitoring the stabilized groundwater level at the site. Wells are comprised of a 50 mm diameter CPVC pipe, with a slotted and filtered screen. Details of monitoring well construction are provided on the attached borehole logs. The screens on each well are mill-slotted, with a slot spacing of 0.5 mm, and were backfilled with Type 2 Silica Sand. Above the screened depth, the annular space was backfilled with a bentonite slurry, up to ground surface. The wells have been equipped with lockable caps. The monitoring wells have been registered with the Ministry of Environment, Conservation, and Parks (MECP), in accordance with Ontario Regulation (O.Reg.) 903.

Table 2 (below) summarizes the well construction details.

Table 2 – Well Construction Details

Borehole	Ground Surface Elevation, m	Well Installation Depth, m	Screened Length, m	Screened Strata
BH1/MW	201.39	4.57	3.05	Fine grained sand, some silt
BH3/MW	202.07	4.57	3.05	Fine grained sand, some silt

The depth to groundwater seepage and short-term water level measurements were obtained prior to backfilling the remaining boreholes. Boreholes were backfilled with a mixture of bentonite chips and cuttings, to restore holes back to level conditions with the ground surface.

All samples recovered from the site were returned to LDS for detailed examination and selective testing. Two grain size analyses were carried out on select samples of the predominant sandy soils, where perched groundwater conditions were identified. Routine moisture content determinations were carried out on select samples and results are presented on the borehole logs provided in Appendix B.

Collected soil samples will be disposed of, following the issuance of this Report, unless prior arrangements have been made for longer term storage.

3.1.1 Soil Conditions

A series of four boreholes were advanced at the site to examine soil and shallow groundwater conditions. The borehole locations are shown on Drawing 6, appended. In general, soils observed in the boreholes consisted of topsoil/fill overlying sand. General descriptions of subsurface conditions are summarized in the following sections. Borehole logs are provided in Appendix B, for reference.

It should be noted that boundaries of soil indicated in the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries reflect transition zones for the purposes of geotechnical design and should not be interpreted as exact planes of geological change.

Topsoil

Each borehole was surfaced with a layer of topsoil. The topsoil consisted of dark brown sandy loam, and the thickness generally ranging from 75 to 200 mm across the site. The topsoil was in a moist state at the time of the fieldwork, based on visual and tactile examination.

It should be noted that topsoil quantities noted above are based on information provided at the borehole locations only, and may vary in areas with existing vegetation and tree cover, and where tilling has occurred and mixed the topsoil with the underlying soil strata. If required, a more detailed analysis (involving additional shallow test pits) is recommended to accurately quantify the amount of topsoil to be removed for construction purposes.

Fill

A layer of fill material was encountered underlying the topsoil in Borehole BH4, on the east end of the site, and extends to 2.9 m below ground surface. The fill consisted of sand, and was described as grey in colour, with a fine-grained texture, and containing trace to some silt and some organics. The fill is described as being in a variable loose to compact state, based on recorded Standard Penetration Test (SPT) N-values in the range of 6 to 14 blows per 0.3 m of split-spoon sampler penetration. Moisture content determinations conducted on recovered samples of the fill material generally range between 15.7 to 19.5 percent, generally indicative of moist to very moist soil conditions.

Sand

The predominant subgrade soil encountered in each borehole was a layer of natural sand. Each borehole terminated within this layer. The sand was generally described as being brown in colour, with a fine grained texture, and containing some silt.

Two samples of the sand layer were submitted for gradation analysis, and the following table shows the grain size distribution. The results are also shown graphically in Appendix B.

Table 3 – Gradation Summary, Sand

Sample ID	Unified Soil Classification			
	% Silt	% Sand	% Gravel	% Cobbles
BH1, Sample 5 – 4.6 m depth	17.7%	82.3%	0.0%	0.0%
BH3, Sample 5 – 4.6 m depth	23.7%	76.3%	0.0%	0.0%

The sand is in a loose to compact state, based on SPT N-values in the range of 5 to 23 blows per 0.3 m of split-spoon sampler penetration. In Borehole BH2, very loose soil conditions (SPT N < 4 blows) were encountered within the sand layer between 2.1 and 2.9 m below ground surface.

Moisture content determinations conducted on recovered samples of the sand range between 7.4 and 27.4 percent above the stabilized groundwater elevation, and on the order of 14.7 to 32.8 percent below the stabilized groundwater level.

3.1.2 Soil Permeability

The hydraulic conductivity of a soil depends on a number of factors, including particle size distribution, degree of saturation, compactness, adsorbed water (which depends on clay content). The heterogeneous nature of glacial deposits can also contribute to variations in soil permeability where the soil composition may include localized areas with increased fine material or sandy material which can influence soil permeability at different points within the soil strata.

Based on the gradation results presented in Section 3.1.1, the following values for saturated hydraulic conductivity have been determined for the predominantly sandy soils encountered across the site. Hazen's method was used to correlate the grain size analysis to the hydraulic conductivity of the sand soils. This correlation is based on the following relationship:

$$k \text{ (cm/s)} = C(d_{10})^2$$

where, d_{10} is the diameter (size measured in mm) at which 10% of the sample passes; and,
C is an empirical coefficient (average value of 1.0).

Table 4 – Hydraulic Conductivity and Factored Infiltration Rates from Grain Size Analyses

Sample ID	Sample Composition			Parameter		
	% Silt	% Sand	% Gravel	D_{10} (mm)	Saturated Hydraulic Conductivity (m/sec)	Factored Infiltration Rate (mm/hr)
BH1, Sample 5 – 4.6 m depth	17.7	82.3	0.0	0.062	3.84 $\times10^{-5}$	49
BH3, Sample 5 – 4.6 m depth	23.7	76.3	0.0	0.065	4.23 $\times10^{-5}$	50

The natural water-bearing sand soils have a saturated hydraulic conductivity in the range of 4.0 $\times10^{-5}$ m/s, based on the above results.

The above infiltration rates have been calculated using correlation from TRCA/CVC Low Impact Development Stormwater Management Planning and Design Guide protocol which references Ontario Ministry of Municipal Affairs and Housing (OMMAH). 1997. Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions. Toronto, Ontario. A Factor of Safety of 2.5 has been applied, in accordance with TRCA/CVC Low Impact Development Stormwater Management Planning and Design Guide protocol.

A number of factors can influence the actual soil permeability and infiltration rate onsite during the site grading activities, including cut-fill activities, and the use of onsite or imported materials to achieve design grades. It is recommended that geotechnical inspection of materials which are used onsite and field testing during the construction phase of the project be carried out to confirm that infiltration rates which have been used for design purposes are appropriate to the actual site conditions.

3.1.3 Shallow Groundwater Conditions

Short term water level observations were recorded from the open boreholes at the time of installation. Groundwater observations in the open boreholes and a review of soil moisture contents are indicative of the shallow groundwater generally being contained within the sandy soils near surface. Short term water levels are summarized in the following table.

Table 5 - Short Term Groundwater Observations

Borehole	Ground Surface Elevation, m asl	Groundwater Observations, m bgs	Groundwater Elevation, m asl
BH2	201.47	4.27	197.20
BH4	203.39	Dry	--

Stabilized water level measurements were recorded in the monitoring wells installed across the site on February 25 and March 11, 2022, and are summarized in the following table.

Table 6 – Stabilized Groundwater Observations

Monitoring Well	Ground Surface Elev. (m, asl)	Depth to Groundwater (m, bgs) *Groundwater Elevation (m, asl)*	
		February 25, 2022	**March 11, 2022**
BH1/MW	201.39	4.54 / 196.85	4.36 / 197.03
BH3/MW	202.07	4.47 / 197.60	4.38 / 197.69

Shallow groundwater is present within the near-surface sandy soils, below Elevation 197.7 m. Shallow groundwater will vary in response to climatic or seasonal conditions, and, as such, may differ at the time of construction, with higher levels possible during mild weather conditions which create melting conditions, and during wet periods.

The potentiometric surface and water table mapping provided in the Little Creek Subwatershed Study identifies that groundwater contained within the shallow to intermediate depth aquifers flow laterally to local water courses, where discharge occurs. For the purposes of this study, that would suggest that the groundwater flow will be in a north-westerly direction, towards the wetlands located on the north side of Sunset Drive. The water level measurements taken at the site are indicative of a westerly groundwater flow direction. This may change under changing seasonal conditions, and as such additional groundwater level monitoring under seasonal conditions is recommended.

Wells have been registered with MECP and may be used for further stabilized water level measurements and/or water quality sampling, as needed.

4.0 GEOTECHNICAL COMMENTS AND RECOMMENDATIONS

It is understood that the proposed development will include the construction of four one-storey commercial buildings, with associated surface parking. The site is expected to be serviced with municipal services, and accessed from an internal roadway which will connect to Sunset Drive along the northern limits of the site. A Preliminary Concept Plan is appended; however, it is important to note that the final site design may be changed. If this occurs, the geotechnical comments and recommendations provided in this report should be reviewed to confirm that they remain applicable and appropriate for the proposed site development.

The boreholes generally revealed a layer of surficial topsoil/fill which is underlain by natural sand soils. Shallow groundwater was encountered in the near surface sandy soils, located approximately 4.4 to 4.5 m below existing ground surface (below Elevation 197.7 m.)

The following sections of this report provide geotechnical comments and recommendations for the proposed commercial development, including: site preparation (including the re-use of excavated materials as engineered fill, structural fill, and guidance for engineered fill placement, and preliminary guidance on the new Excess Soil Management regulation [O.Reg. 406]), temporary excavations (including maximum slope inclinations to provide stable excavation side slopes in accordance with OHSA requirements), excavation support (shoring methods, (if required), groundwater control (including the need for a Permit to Take Water (PTTW) or Environmental Activity and Sector Registry (EASR) submission for construction dewatering, foundation design (including soil bearing capacity, subgrade preparation, and potential settlements), slab on grade construction (including lateral earth pressures, and foundation backfilling), site servicing (including the re-use of onsite soils in service trenches), pavement design (including pavement component thicknesses for local roads and reinstating service connections which extend into the municipal right-of-way) and slope stability review and analysis.

4.1 Site Preparation

4.1.1 Building Demolition and Removal of Existing Structures

The subject site currently contains an old restaurant, which fronts on Sunset Drive. It is anticipated that this building will be removed prior to the site grading activities.

Based on the age of the existing building, it is possible that hazardous buildings materials and/or designated substances may be present. Owner requirements are set forth by the Ontario Ministry of Labour (MOL) under Ontario Regulation (O.Reg.) 490/09: Designated Substances; Section 30(1) of the Ontario Occupational Health and Safety Act (OHSA); and Section 10 of O.Reg. 278/05: Designated Substance – Asbestos on Construction Projects and in Buildings and Repair Operations, as amended. It is recommended to complete a Designated Substances Survey / Hazardous Building Materials Survey (DSS/HMS) for the Site building prior to demolition to identify any hazardous substances within the premises and to provide guidance on how to handle/maintain any such products during the demolition.

During the demolition of the buildings, it is important that any existing concrete floor slab and building foundations be removed from areas which will house the proposed structures at the site. In the event that old and/or abandoned septic tanks or septic distribution systems are encountered, they should be removed in their entirety. Septic tanks should be pumped out by licensed contractors, prior to being removed from the site. In the event that any old or abandoned wells are revealed, they should be properly decommissioned under the supervision of a licensed well technician, in accordance with O.Reg. 903.

4.1.2 Site Grading Activities

Based on existing site conditions, it is expected that some site grading activities will be required. Vegetation removal and topsoil stripping is anticipated throughout the area to be developed. In general, this is expected to require the removal of about 30 to 200 mm of surficial topsoil. Thicker topsoil areas may also be present between the borehole locations, in proximity to existing wooded areas, and where local depressions are present at the site.

The boreholes were located away from the existing building and site services. If existing services are encountered during the site preparation work, they may need to be removed or rerouted, as appropriate, particularly if they are located within future building footprint areas. Fill material associated with trench backfill may require site review by the geotechnical consultant to determine its suitability to remain in place, depending on the final site design.

Surficial topsoil may be stockpiled on site for possible re-use as landscaping fill. In the event that material is disposed of offsite, testing of the material for transport should conform to MECP Guidelines and requirements.

The existing fill material is generally described as containing deleterious inclusions (organics), as well as being in a loose state. This material is not considered suitable to support the new building, or to support site pavements without risk of settlement.

Prior to placement of engineered fill or new building foundations, existing fill and topsoil, vegetation and otherwise deleterious materials should be removed. Once complete, the exposed subgrade should be thoroughly proof-rolled and inspected by geotechnical field staff from LDS. Any loose or soft zones noted during the inspection should be over excavated and replaced with approved fill.

In areas which engineered fill is to be placed to raise grades, the exposed subgrade soils should be approved by the geotechnical consultant following topsoil stripping. In accordance with the Ontario Building Code (Section 4.2.4.15), foundations may be set on fill material provided that it can be demonstrated that the fill is capable of safely supporting the building and that detrimental movement of the building will not occur. In this regard, it is recommended that any fill material placed in future building footprints be engineered and verified through an inspection and testing program. Engineered fill should consist of suitable, compactable, inorganic soils, which are free of topsoil, organics and miscellaneous debris. For best compaction results, the fill material should have a moisture content within about 3 percent of optimum, as determined by Standard Proctor testing.

The existing fill material encountered in Borehole BH4 is <u>not</u> considered suitable for re-use as engineered fill. The placement of the engineered fill should be monitored by the geotechnical consultant to verify that suitable materials are used, and to confirm that suitable levels of compaction are achieved. The engineered fill material should be placed in maximum 300 mm (12 inch) thick lifts and uniformly compacted to 100 percent Standard Proctor Maximum Dry Density (SPMDD). Additional notes regarding engineered fill placement are provided on Drawing 7, in Appendix A.

The existing natural subgrade soils, comprised of sand, that are not mixed with obviously unsuitable material may be suitable for re-use as engineered fill. The possible re-use of onsite soils should be subject to review and approval by the geotechnical consultants.

Fill material containing building debris and / or topsoil and organic inclusions is generally not expected to be suitable for re-use onsite, except where landscaping (non-structural) fill may be needed. Offsite disposal of these soils will require analytical testing, in accordance with MECP Guidelines and classification requirements for transport and disposal. The testing requirements for disposal will depend on the requirements outlined by the receiver.

4.1.3 Excess Soils Management Considerations

In December of 2019, the Ministry of Environment, Conservation, and Parks (MECP) released a new regulation under the Environmental Protection Act, titled *On-Site and Excess Soil Management* to support improved management of excess construction soil. Due to Covid-19, the implementation of this regulation was delayed, however, as of January 1, 2021, the new Excess Soil Regulation (O. Reg. 406/19) is being phased in across Ontario.

Excess soil is defined as material that was generated during construction activities at a Site but will not be needed onsite for grading, fill, or other purposes and therefore needs to be excavated and removed from the Site. The regulation requires a project leader (which in this case, would be the owner of the property) to comply with specific requirements before their contractor can remove excess soil from a project area. Generally, these requirements include:

- Preparation of an Assessment of Past Uses Report which is similar to a Phase One Environmental Site Assessment for the source site, to evaluate the presence of potentially contaminating activities which may have resulted in the potential for impacted soil or groundwater conditions to be present at the source site;
- Preparation and implementation of a Sampling and Analysis Plan which outlines the suggested sample locations and sampling intervals, analytical sample testing parameters, and sampling frequency;
- Preparation of a Soil Characterization Report, following the soil sampling and analytical testing;
- Preparation of an Excess Soil Destination Assessment Report which identifies where excess soils can be disposed offsite, including a review of Beneficial Reuse Sites, if the developer and/or their contractor have a potential re-use site being considered; and,
- Development and implementation of a tracking system.

Soil testing should reflect the highest concentration of contaminants of potential concern (as determined by the QP) on site. In order to adequately characterize the excess soil, the regulation prescribes a minimum number of samples to be collected, depending on soil volume excavated, as well as a minimum list of parameters to be analyzed for. The new requirements on number of samples and minimum sample parameters are summarized in the following tables.

Table 7 – Minimum Number of Samples

<table>
<tr><th rowspan="2">Volume Threshold</th><th colspan="2">Minimum number of samples for Bulk Soil Analysis</th><th rowspan="2">Minimum number of samples of Leachate Analysis</th></tr>
<tr><th>Small Volume Projects</th><th>Volume Independent Projects</th></tr>
<tr><td>≤350 m3</td><td>≥ 3 samples</td><td>- -</td><td>- -</td></tr>
<tr><td>≤350 m3 to <600 m3</td><td rowspan="4">- -</td><td>≥ 3 samples</td><td>≥ 3 samples</td></tr>
<tr><td>>600 m3 to <10,000 m3</td><td>≥1 sample for each additional 200 m^3 within threshold limits</td><td rowspan="3">3 samples + 10% of Bulk Soil samples collected</td></tr>
<tr><td>>10,000 m3 to <40,000 m3</td><td>≥1 sample for each additional 450 m^3 within threshold limits</td></tr>
<tr><td>>40,000 m3</td><td>≥1 sample for each additional 2,000 m^3 beyond threshold limit</td></tr>
</table>

Table 8 – Minimum Analytical Requirements

Minimum Parameters to be Analyzed	Surface and Subsurface Soils
Metals (including Hydrides)	✓
Benzene, Toluene, Ethylbenzene, and Xylenes (BTEX)	✓
Petroleum Hydrocarbons (PHCs) F1 – F4	✓
pH, EC, SAR	✓
Leachate Analysis	See Note 1
Notes 1. Leachate analysis is conditional on contaminant of potential concern being identified by the QP, the volume of excess soil exceeding 350m^3 and applicable standards	

It should also be pointed out that for Volume Independent Projects (<350 m^3) additional Excess Soils Standards (which somewhat differ from the currently used O. Reg. 153/04 SCSs) were developed and need to be considered when moving materials from one Site to another. The above notes the minimum sampling requirements; based on past site uses the QP may require additional sample parameters to be added to the above listed. Furthermore, O. Reg. 406/19 may have other implications on proposed soil management activities (such as guidelines of receiving site and temporary soil storage sites) that are not noted above.

The onus is on the Excess Soil Source Site to carry out environmental soil quality testing for the removal and transport of their excess soils. The Source Site is required to have a Qualified Person (QP) complete a Soil Characterization Report (SCR) summarizing the soil testing results, which can be provided to the Beneficial Re-Use (receiver) Site for review to confirm the quality of materials which is being proposed to be imported to the site. There are significant efforts and costs associated with analytical testing of soils and preparation of the required documents, for which the Source Site may look to Beneficial Re-use Sites to share some of the cost.

For sites which require imported fill (identified as Beneficial Re-Use Sites), a Qualified Person (QP) will need to be retained to prepare an Excess Soil Destination Assessment Report (ESDAR), which outlines the geotechnical requirements for beneficial reuse of imported materials onsite along with the environmental soil quality criteria (including the applicable O.Reg. 153/04 Site Condition Standards) for material which is appropriate to be accepted at the Site. In this case, material meeting the O.Reg. 153/04 Table 1 Site Condition Standards, Industrial/Commercial/Community Land Use (or better) would be suitable for acceptance. This is recommended as a result of the woodlands/wetlands located within close proximity of the site.

4.2 Methane Abatement

As presented in MECP Guideline D-4-1, the LEL (lower explosive level) of methane is generally considered to be 5% methane by volume. That means the mixture is too lean to burn if there is less than 5% methane present. But at 5%, it can burn or explode if there is an ignition source. The total combustible vapours are presented as an equivalent % LEL value in the above table.

A threshold limit of 500 ppm is used for monitoring purposes, to identify if a potential hazard exists (equivalent to 0.05% methane). For additional reference, the National Institute for Occupational Safety and Health's (NIOSH) maximum recommended safe methane concentration during an 8-hour period is 1,000 ppm.

No discernable methane concentrations were recorded in the open boreholes. As noted in Section 9.13.4.2 (b) of the Ontario Building Code, where detected soil gas levels remain below the threshold limit identified above, no special methane abatement measures are required.

4.3 Excavations and Groundwater Control

Excavations for the proposed buildings and site services are generally expected to extend into the natural sand soils, or possible engineered fill material, depending on final site grades. Site servicing depths are generally expected to be in the range of 4 m maximum depth.

All work associated with design and construction relative to excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). The following soil classifications are provided in accordance with Section 226 of Ontario Regulation 213/91:

- The natural sand and sandy fill soils encountered in the boreholes are generally classified as Type 3 soils above the stabilized water table, or where soils have been suitably dewatered. For excavations which extend through or terminate in Type 3 soil, temporary excavation side slopes must be cut back at a maximum inclination of 1H:1V from the base of the excavation.

It should be noted that, if wet seams or zones are encountered, some sloughing to flatter slopes may be expected. If the construction excavation side slopes recommended above cannot be maintained due to lack of space or close proximity of other structures, an engineered excavation support system must be used. Minimum support system requirements for steeper excavations are stipulated in Sections 234 through 242 of the Occupational Health and Safety Act. The engineered shoring system, if required, must be in place prior to commencement of the installation operations.

In the event that construction occurs in seasonally wet conditions or when frozen soil conditions are present, care will be required to maintain safe excavation side slopes, and suitable excavation bases. The contractor should use a reasonable effort to direct surface run-off away from open excavations.

4.3.1 Excavation Support

If space restrictions at the site do not allow for conventional open cut without risk of undermining, or where excavation sizes are to be limited, the use of adequate bracing or shoring may be required. In the natural subgrade soils, bracing will not normally be required if the structures are behind a 45-degree line drawn up from the near edge of the excavation.

If the construction excavation side slopes recommended above cannot be maintained due to lack of space or close proximity of other structures, an engineered excavation support system must be used. Minimum support system requirements for steeper excavations are stipulated in Sections 234 through 242 of the Act and Regulations. The shoring system must be designed to be internally (overturning, and sliding) and externally stable (slope stability/base heave).

A prefabricated trench box may be used provided that it is designed (by a professional engineer) to withstand the soil and hydrostatic loading (if applicable).

Based on the field and laboratory testing during the present geotechnical investigation and our experience with similar soils, the following soil parameters are recommended for the design of the engineered shoring system.

Table 9 – Soil Parameters for Excavation Support

Soil	φ	γ (kN/m^3)	K_a	K_o	K_p
Compact Sand and Silty Sand	30	19.5	0.33	0.50	3.15
Compact Granular 'B' (OPSS 1010)	32	22.0	0.31	0.47	3.25

In the event that imported fill material is present near the excavation which vary materially from the above soils, the geotechnical consultant should review the soil conditions to confirm the design parameters.

4.3.2 Groundwater Control

Based on the results of the investigation, shallow groundwater is located approximately 4.4 to 4.5 m below existing ground surface, contained within the near surface sandy soils.

Conventional groundwater control methods are expected to be suitable for shallow excavations which remain above the groundwater table at the site, to address surface water infiltration and minor shallow groundwater seepage for excavations which do not extend below the stabilized groundwater table.

Where excavations extend below the stabilized groundwater table, or where groundwater levels are elevated, positive groundwater control methods may need to be utilized for construction dewatering. Soil permeability values in the natural sandy subgrade soils are expected to be in the range of 10^{-5} m/s, based on laboratory testing (presented in Section 3.2). This information is provided to assist with determining appropriate construction dewatering methods.

Groundwater control measures at the site should be sufficient to maintain stable excavated slopes; and provide a dry and stable base for excavations and construction operations. The contractor should use a reasonable effort to direct surface run-off away from open excavations.

Although not anticipated based on the current soil and groundwater information for the site, it should be noted that for projects requiring positive groundwater control with a removal rate in excess of 50,000 litres per day, a submission to the Environmental Activity and Sector Registry (EASR) will be required, and a Permit to Take Water (PTTW) will be required for volumes in excess of 400,000 litres per day. PTTW applications are submitted to and approved by MECP according to Sections 34 and 98 of the Ontario Water Resources Act R.S.O. 1990 and Water Taking and Transfer Regulation O. Reg. 387/04. The need for an EASR submission or PTTW should be reviewed when design depths for the building foundations and site servicing have been verified.

4.4 Building Components

4.4.1 Foundation Design

For design of conventional strip and pad footings on the natural subgrade soils below 1.2 m below existing grades or supported on engineered fill, the following allowable bearing pressures (net stress increase) can be used for design of footings:

- Serviceability Limit States (SLS) 150 kPa (~3000 psf)
- Ultimate Limit States (ULS) 225 kPa (~4700 psf)

All footings exposed to seasonal freezing conditions should be protected from frost action by at least 1.2 m (4 ft.) of soil cover or equivalent insulation.

The natural sandy subgrade may be susceptible to disturbance by construction activities, especially during adverse weather conditions or when water seepage from excavation sidewalls are present. Consequently, after the founding surfaces have been exposed, the soils should be thoroughly recompacted to provide a uniform base, suitable to provide the bearing capacity noted above. Consideration should be given to placing concrete foundations as soon as possible following excavation and subgrade inspection.

Excessive differential settlements can occur where the subgrade support material types differ below the underside of continuous strip footings, (i.e., natural glacial till to engineered fill). As such, where strip footings transition from one material to another the transition between the materials should be suitably sloped or benched to mitigate differential settlements. It is recommended that the following transition precautions to mitigate/accommodate potential differential settlements be considered, and incorporated into the design, subject to review by the structural engineer:

- For strip footings, the transition zones should be adequately reinforced with additional reinforced steel lap lengths or widened footings;

- Steel reinforced poured concrete foundation walls; and
- Control joints throughout the transition zone(s).

Individual spread footings should generally be spaced a minimum distance of 1.5 times the largest footing width apart from each other to avoid stress bulb interaction between footings. This assumes the footings are at the same elevation.

Footings at different elevations should be located such that the higher footings are set below a line drawn up at 10 horizontal to 7 vertical from the closest edge of the lower footing. It is important that servicing excavations which encroach on the building foundations are checked to ensure that they do not undermine the building foundations.

Verification of the footing base conditions should be undertaken by the geotechnical engineer at the time of excavation. Provided that the stability of the soils exposed at the founding level is not compromised as a result of construction activity, precipitation, cold weather conditions, etc., and the design bearing pressures are not exceeded, the total and differential settlements of footings are expected to be less than 25 mm and 19 mm, respectively.

It should be noted that the recommended bearing capacities have been calculated by based on the observations of the soil and groundwater conditions within the borehole program at the site. Where variations occur between the borehole locations, and during construction of the new buildings, site verification by the LDS' geotechnical engineer is recommended to confirm soil conditions and verify soil bearing capacity.

4.4.2 Slab on Grade Construction

Concrete floors for the new buildings may be constructed using conventional concrete poured slab techniques, following the review and approval of the subgrade soils

In preparation for the construction of the floor slab, any unstable (loose) fill material should be removed and recompacted (as noted previously) where founding soils will support the floor slab. In the event that the exposed subgrade soils are wet they will exhibit a greater sensitivity to disturbance.

Care should be taken to protect the subgrade below the floor slab during construction, by limiting construction traffic on the prepared subgrade soils. In addition, if the exposed subgrade soils are exposed to inclement weather conditions (i.e. rain, snow, freezing conditions), some remedial works may be required to remove wet, soft, or disturbed soils prior to stone and concrete placement.

A moisture barrier, consisting of a minimum 200 mm thick of uniformly compacted 19 mm clear stone should be placed over the approved subgrade. For design purposes, the modulus of subgrade reaction (k) can be taken as 45 MPa/m, for the compacted stone over approved subgrade soils. An alternate configuration of compacted granular material such as OPSS 1010 Granular A may also be considered for the moisture barrier. If alternative materials are proposed for use onsite, the minimum level of compaction and overall design thickness of the moisture barrier layer should be reviewed by the geotechnical consultant.

The water-to-cement ratio of the concrete utilized in the floor slab should be strictly controlled to minimize shrinkage of the slab. Adequate joints and / or the use of fibre reinforcement may be considered by the designer to help control cracking. The sawcut depth for control joints should be ¼ of the slab thickness. The use of super plasticizers should be considered to reduce shrinkage and increase workability of the concrete.

During construction, concrete sampling and testing is recommended to ensure that concrete mix design requirements are satisfied.

4.4.3 Concrete Recommendations

CSA A.23-1.04 provides minimum requirements for concrete, including Exposure Class, maximum water to cement ratios, allowable air entrainment, slump, temperature requirements, etc. The design of the building foundations should have regard to the above referenced standard, and should be reviewed by the designer for conformance to CSA standards.

It is recommended that the water-cement ratio and slump of concrete used for floor slabs be controlled to minimize shrinkage of the slabs. Adequate joints and/or the use of fibre reinforcement may be considered by the designer to help control cracking. During construction, concrete sampling and testing is recommended to ensure that concrete mix design requirements are satisfied.

Concrete sampling and testing for foundations (in accordance with CSA A23.1 and project specifications) is recommended. During cold weather, freshly placed concrete should be covered with insulating blankets to protect against freezing.

4.4.4 Seismic Design Considerations

Subsoil and groundwater information at this site have been examined in relation to Section 4.1.8.4 of the Ontario Building Code (OBC) 2012. The subsoils expected below the buildings will generally consist of natural sand. It is anticipated that the proposed development will be founded on these deposits, below any loose or soft zones.

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2012 indicated that to determine the site classification, the average properties in the top 30 m are to be used. The boreholes at the site were advanced to a maximum depth of 5.0 m below existing ground surface. The Site Classification recommendation is based on the available information as well as our interpretation of conditions at and below the boreholes, and based on a review of geological mapping and MECP well records, and our knowledge of the soil conditions in the area.

Based on the above assumptions, interpretations in combination with the known local geological conditions, the Site Class for the proposed development is “D” as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012. In the event that a higher Site Classification is being sought by the structural design engineer, additional boreholes and / or multichannel analysis of surface waves (MASW) testing would be required to determine if the soil conditions below the current depth of exploration can support a higher Site Classification.

4.5 Site Services

Subgrade soils beneath new services are generally expected to consist of natural sand soils. Minor groundwater seepage from the near surface sandy soils should be anticipated. Although no bearing problems are anticipated for flexible or rigid pipes founded on natural deposits, localized base improvement along the trench bottom may be required for excavations which terminate in wet subgrade soils. The extent of base improvement or stabilization is best determined in the field during construction, with consultation from LDS' geotechnical engineer.

For services supported on native deposits, the bedding should conform to Municipal and OPS Standards. Bedding aggregate should be compacted to a minimum 95 percent SPMDD. Water and sewer lines installed outside of heated areas should be provided with a minimum 1.2 m of soil cover for frost protection.

Requirements for backfill in service trenches, etc. should also conform to Municipal and OPS Standards. A program of in situ density testing should be set up to ensure that satisfactory levels of compaction are achieved. Based on the results of this investigation, excavated material for trenches will generally consist of silt. Select portions of this inorganic material may be used for construction backfill provided that reasonable care is exercised in handling the material. In this regard, material should be within 3 percent of the optimum moisture as determined by the Standard Proctor density test. The underlying sand and gravel soils encountered below 5.5 m depth are expected to be wet, and may require drying or blending with drier material prior to re-use.

Stockpiling of material for prolonged periods of time should be avoided. This is particularly important if construction is carried out in wet, adverse weather.

Soils excavated from below the stabilized groundwater table may be too wet for re-use as backfill, unless adequate time is allowed for drying, or if material is blended with approved dry fill; otherwise, it may be stockpiled onsite for re-use as landscape fill, or disposed of off-site, testing of the material for transport should conform to MECP Guidelines and requirements.

Backfill above bedding aggregate can consist of excavated (inorganic) soils, compacted in maximum 300 mm thick lifts to a minimum of 95 percent SPMDD. A program of in situ density testing should be set up to ensure that satisfactory levels of compaction are achieved.

4.6 Pavement Design

The development will be accessed with an internal road network, accessing Sunset Drive to the northwest. The exposed subgrade soils within the roadways are expected to be comprised of re-compacted soils comprised of sand. The road subgrade should be thoroughly proof-rolled and reviewed by the geotechnical consultant. In the event that unstable fill, loose or soft areas are noted, additional work may be required to sub-excavate and replace unstable soils with suitable compactable material. This work should be completed under the supervision of the geotechnical consultant. In general terms, compacted soils supporting site pavements (including the upper 1 m of service trench backfill) should be compacted to a minimum level of 98 percent SPMDD.

The recommended pavement structure provided in this report is based on the natural subgrade soils encountered in the boreholes or suitably re-compacted soils, as described previously. Provided that the preceding recommendations are followed, the pavement thickness design requirements given in the following table are recommended for the anticipated subgrade conditions and traffic loading on the internal network of local roads.

Table 10 – Pavement Design Recommendations

Pavement Component	Minimum Design Thicknesses			Compaction Requirements
	Light Duty Car Parking	Internal Roads & Truck Parking	Tie-ins and Restoration of Sunset Drive	
Asphaltic Concrete	35 mm HL 3 45 mm HL 8	40 mm HL 3 50 mm HL 8	50 mm HL 3 60 mm HL 8	97% BRD
Granular A Base	150 mm	150 mm	150 mm	100% SPMDD
Granular B Subbase	250 mm	300 mm	450 mm	100% SPMDD

The recommended pavement structure provided in this report is based on natural subgrade soil properties determined from visual examination and textural classification of the soil samples. Where new pavements intersect with Sunset Drive, the subgrade beneath new pavement should be tapered to match existing road subgrade to minimize differential frost heaving for the pavement structure. Site review by the geotechnical engineer is recommended to verify this at the time of construction.

It is recommended that a program of inspection and materials testing (including laboratory analyses and compaction testing) be carried out during construction to confirm that geotechnical requirements are satisfied.

- Samples of both the Granular 'A' and Granular 'B' aggregates should be checked for conformance to OPSS 1010 prior to use on site, and during construction.
- The asphaltic concrete paving materials should conform to the requirements of OPSS 1150. The asphalt should be placed in accordance with OPSS 310.
- Specified compaction levels are identified in the table, above. Alternatively, to the specified compaction range noted in the above table for asphalt compaction, a compaction level of 92.0 to 96.5 percent of the Marshall relative density (MRD) is also an appropriate measure for asphalt compaction.

Good drainage provisions will optimize pavement performance. The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum grade of two percent) to provide effective surface drainage. The sand subgrade soils have good natural drainage and therefore pavement subdrains are not anticipated.

4.7 Curbs and Sidewalks

Concrete for any new exterior curbs and sidewalks should be proportioned, mixed placed and cured in accordance with the requirements of OPSS 353 and OPSS 1350, which are summarized (in part) below:

- minimum 28-day compressive strength = 30 MPa;
- coarse aggregate = 19.0 mm nominal max. size;

- maximum slump = 60 mm; and,
- air entrainment = 7.0 ± 1.5%.

During cold weather (when the air temperature is at or is likely to fall below 5°C within 96 hours of concrete placement) the freshly placed concrete must be covered with insulating blankets to protect against freezing, as per OPSS 904. Ice and snow must be removed from the area where concrete is to be placed and the concrete must not be placed against frozen ground. All cold weather protection material shall be on site prior to each concrete placement.

Subgrade for sidewalks should consist of undisturbed natural soil or well compacted fill. A minimum 100 mm thick layer of compacted (minimum 100 percent SPMDD) Granular 'A' should be placed below sidewalk slabs. It is recommended that Granular 'A' material extend at least 150 mm beyond the edges of the proposed sidewalk. The subgrade and granular base should be prepared in accordance with the requirements of OPSS 315.

The concrete for the sidewalk shall be according to OPSS 1350 and the following:

- class of concrete = Nominal 28 day compressive strength 30 MPa
- coarse aggregate = 19 mm nominal maximum size
- air content = 7.0% ± 1.5%, measured prior to placement
- slump = 70 ± 20 mm

Field sampling and testing of concrete shall be according to OPSS 904.

4.8 Retaining Walls

If consideration is being given to incorporating retaining wall structures into the final site design, the proposed retaining wall structures should be founded on natural undisturbed subgrade soils, or approved structural fill material. Based on the soil conditions observed onsite, a design pressure of 150 kPa is considered appropriate for the sandy subgrade soils which are in a compact state. Site inspection by a geotechnical inspector is recommended during construction to verify the suitability of the subgrade soils. LDS can assist in this regard, upon request.

The retaining walls should be provided with a subdrain system equipped with a positive outlet to provide an outlet for any infiltrated surface water which accumulates behind the wall, throughout the retaining wall system. Refer to Table 9, for design soil parameters.

4.9 Slope Stability

This portion of the Geotechnical Investigation – more specifically, the Slope Stability Assessment has been prepared to satisfy the requirements of the Central Elgin Official Plan, namely those requirements which are spelled out in Section 3.2, Natural Hazards. Some relevant excerpts are provided below, for reference.

- Section 3.2.1 (d) - *Where development and/or site alteration is proposed on lands adjacent to the Natural Hazard designation, the site specific limits of the natural hazard(s) shall be determined through relevant* *studies prepared by a qualified professional with recognized expertise in the appropriate principles using accepted methodologies to the satisfaction of the Municipality and the conservation authority having jurisdiction in the area. Those limits shall be interpreted as the correct limits of the Natural Hazard designation and such interpretation shall not require amendment to this Plan.*

- Section 3.2.2 (b)- *The Natural Hazard designation shown on the land use schedules includes those areas in which there may be potential for risk to life and property as a result of erosion hazards. The Erosion Hazard Limit is determined using the 100 year erosion rate (the annual rate of recession extended over a hundred year time span), an allowance for slope stability and an erosion access allowance to be no less than 6 metres.*
    - *b) Where new development and/or site alteration is proposed within 30 metres of a Natural Hazard designation shown on the land use schedules: 1. The proponent shall complete a geotechnical analysis to determine the Erosion Hazard Limit. The analysis is to be prepared by a qualified professional having recognized expertise in the appropriate principles using accepted methodologies and approved by the Municipality and the Conservation Authority. 2. The Erosion Hazard Limit shall be interpreted as the correct limits of the Natural Hazard designation and such interpretation shall not require amendment to this Plan.*

4.9.1 Site Reconnaissance & Slope Stability Rating

LDS carried out a geotechnical review of the condition of the vegetated slope which borders the site. This slope stability assessment has been conducted to support the proposed commercial development located proximal to the toe of the slope. A site review was carried out by LDS on February 25th, 2022. At the time of the site reconnaissance visit, the slopes were observed to be well vegetated with mature trees.

During the site reconnaissance, sufficient site details were collected to assess the slope condition using the Ministry of Natural Resources and Forestry (MNRF) Slope Stability Rating Chart. The Rating Chart summarizes site observations and empirically scores various elements which contribute to slope stability, to assess the potential for slope instabilities at the site. Three locations were selected for review, each one proximal to Boreholes BH1, BH2 and BH3, denoted as A-A', B-B' and, C-C' respectively. A Slope Stability Rating Chart has been completed for each profile, and are included in Appendix C for reference.

The Slope Instability Rating is in the range of 30 to 34, indicating a low potential for instability. The slope rating suggests that the following scope of work is appropriate to assess the slope's stability:

- Site inspection;
- Survey; and
- Desktop Study and Report.

In addition to the above, LDS advanced four boreholes at the site to assess the soil and groundwater conditions in proximity of the slope.

Select site photographs are presented below for reference. The slopes were generally found to be in a stable condition, with no active zones of slope failure, and no obvious areas of seepage on the slope face. Tree cover throughout the slope is not indicative of movement or slumping. The site observations are consistent with the slope rating charts, which identify a low risk of instability.

	Slope Cross Section 101 Photo taken February 25, 2022
	Slope Cross Section 102 Photo taken February 25, 2022

Slope Cross Section 103

Photo taken February 25, 2022

4.9.2 Slope Setbacks

The Erosion Hazard Limit generally defines the development setback required for hazard lands. The Erosion Hazard Limit is typically comprised of three components; a toe erosion allowance (where the base of the slope is in proximity to a watercourse); the stable slope configuration (based on a minimum factor of safety of 1.4); and the emergency access allowance (providing access to the slope for remedial work, and allowing sufficient space for ingress/egress in the event of an emergency).

The toe erosion allowance is not applicable to the slope which borders the site.

For the determination of the stable slope geometry, LDS has carried out some preliminary analysis using Slope W software, and assuming that soil conditions observed in the boreholes and indicated in the geological mapping are representative soil conditions in the slope which borders the site. A minimum factor of safety of 1.4 is recommended as the threshold for an acceptable slope stability, as indicated in the report “Geotechnical Principles for Stable Slopes” prepared for the Ministry of Natural Resources. A number of potential failure types were assessed, including shallow slumping/sliding failures, medium depth rotation failures near the crest of the slope, and deep rotational failures through the entire height of the slope.

Slope stability calculations indicate the following range of factors of safety:

Cross Section	Shallow Sliding Failure	Medium Depth Failure	Deep Rotational Failure
Slope Profile A	> 1.67	> 1.88	> 1.86

The slope stability analysis yields a minimum factor of safety of 1.67. and is considered acceptable from a geotechnical standpoint. Therefore, the existing slope is considered to be in a stable condition.

The final component is the emergency access allowance. It is understood that this access allowance is required to ensure that there is a large enough safety zone for people and vehicles to enter and exit an area during an emergency, such as slope failure.

In accordance with the PPS, 6 to 15 m setback is required. Since the subsurface conditions within the study area are generally considered to be geologically stable, we recommend that at a minimum, a planning setback of 6 m be applied to the base of the slope. This is also consistent with the minimum Central Elgin planning requirements.

4.9.3 Geotechnical Comments and Recommendations

The proposed construction at the site can be carried out without detrimental impact to the long-term slope stability, provided that some care is taken by the contractors doing the work, and by adhering to the geotechnical comments provided below.

The following comments are provided with regards to site grading and earthworks activities which may be planned in proximity to the existing slopes and natural areas.

- In the event that construction occurs in seasonally wet conditions or when frozen soil conditions are present, care will be required to maintain safe excavation side slopes, and suitable excavation bases. The contractor should use a reasonable effort to direct surface run-off away from open excavations.
- Vegetation on the slope should be maintained. A program of plantation where appropriate, including deciduous trees and deep-rooted vegetation is recommended.
- Excavations should not undermine the toe of the slope.
- In the event that existing drains are exposed during the excavation and site grading works, the drains should be re-routed to ensure continued controlled flows into an appropriate discharge location away from the toe of the slope.
- Final design drawings including the lot layout and services etc. should be reviewed by this office to ensure that the comments and recommendations provided in this report have been properly interpreted.

4.10 Geotechnical Inspection and Testing

An effective inspection and testing program is an essential part of construction monitoring. The Inspection and Testing Program may include the following items:

- Inspection and materials testing during engineered fill placement (full-time monitoring is recommended) and site servicing works, including soil sampling, laboratory testing, and compaction testing;
- Footing base confirmations for any foundations constructed on engineered fill;
- Inspection and testing during construction of site pavements including compaction testing and laboratory testing; and,
- Concrete sampling and testing for curbs and sidewalks.

The Municipality may require inspection and testing records for servicing tie-ins to verify that project specifications have been satisfied for site servicing connections and road repairs, if required.

5. HYDROGEOLOGICAL DISCUSSION

5.1 Hydrogeologic Setting

The Little Creek Subwatershed Study (May 2000), identifies two major types of aquifers in the broader area of the study - those being shallow to intermediate unconfined overburden aquifers, and deeper overburden aquifers. Each are summarized below, as they relate to the proposed residential development of the site.

Shallow & Intermediate Overburden (Sand) Aquifer (0-15 and 15-30 m depth)

Shallow overburden aquifers in the broader area are generally contained within sandy subgrade soils or weathered silty soils in which an unconfined aquifer is present, and perched above less permeable silt/clay subgrade soils, which act as an aquitard. This type of aquifer can be interconnected with surface water features, and is generally fed by infiltrated surface water. Shallow overburden aquifers tend to be heavily influenced by site topography.

As discussed in Section 3.1.3, stabilized water levels were recorded during two visits to the site, prior to issuance of this report. The results indicate that the shallow groundwater contained within the near surface sandy soils. The groundwater measurements (as reported in Section 3.1.3) indicate that the shallow flow generally follows a north-westerly direction, towards the wetlands area located on the north side of Sunset Drive.

It is important to note that shallow groundwater will vary in response to climatic or seasonal conditions, and, as such, may differ depending on the seasonal conditions. Shallow groundwater in unconfined aquifers can be significantly influenced by exceptional and/or sustained rainfall events.

Deep Overburden Aquifers (30+ m depth)

Throughout the study area, deep overburden aquifers, consisting of stratified deposits of varying composition, underlie the less permeable aquitard layer. These soils are described as containing layers, ranging in thickness between 3 and 10 metres, of sand, clay and till, and are generally found to be discontinuous in nature, due to erosional and depositional conditions associated with glacial advancement and retreat. Sand and gravel deposits are present within glacial tills, as a function of the heterogeneous nature of glacial deposits. These aquifers can be consistent over a few hundred meters, but are not often delineated on a regional basis.

A large quantity of the water supply wells for the area, as summarized in the MECP well records, are reportedly sourced from deep overburden aquifers. Excavation depths for building foundations and site servicing for the site are not expected to penetrate down to the deep overburden aquifers.

The regional information provided in the Little Creek Subwatershed Study indicates that the deep overburden aquifer flow directions are difficult to determine (due to the limited information available), but are generally expected to flow towards the south, towards Lake Erie.

Given that the depth to the deep overburden aquifer is some 50 to 70+ m below existing ground surface, the proposed development is expected to have little to no impact on the deep overburden aquifer.

As shown on Drawing 5 in Appendix A, bedrock is estimated at more than 86 to 101 m below ground surface in the vicinity of the site. As such, the potential impact to the bedrock aquifer from the proposed residential development at the site is not anticipated to be significant, and no further discussion is provided regarding the bedrock aquifer.

5.2 Water Level and Groundwater Quantity Considerations

5.2.1 Construction Dewatering

Conventional groundwater control methods are generally expected to be suitable for shallow excavations at the site, to address surface water infiltration and minor shallow groundwater seepage for excavations which do not extend below the stabilized groundwater table.

Where excavations extend below the stabilized groundwater table, or where groundwater levels are elevated, positive groundwater control methods may need to be utilized for construction dewatering. Groundwater control measures at the site should be sufficient to maintain stable excavated slopes; and provide a dry and stable base for excavations and construction operations. The contractor should use a reasonable effort to direct surface run-off away from open excavations.

In the event that groundwater control requires water takings in excess of 50,000 litres per day, an EASR submission or PTTW will be required. The extent of dewatering, estimates for water-taking volumes and zone of influence calculations can be carried out when servicing depths and design grades are available. Under both the EASR and PTTW approval process, a dewatering and discharge plan would need to be prepared, with consideration for potential impacts to nearby water supply wells, and natural features.

Once design depths for site servicing are available for review, LDS can provide additional comments to confirm if an EASR or PTTW is required for construction dewatering efforts at the site.

5.2.2 Local Water Supply Wells

Typical site servicing depths and excavations for building foundations are expected to be well above the intermediate and deep overburden aquifers. From a quantitative standpoint, temporary construction dewatering will not result in the alterations in the water level within those aquifers.

However, it has been recognized that one water supply well noted in the MECP well records in the vicinity of the site is set in the shallow overburden aquifer. A well survey of the nearby and neighbouring properties is recommended to confirm the presence of any additional shallow wells which may be present in the area.

In the unlikely event that long-term or permanent water supply interference occurs to a shallow well located in the area, which can be attributed to the development activities at the site, the developer should have a contingency plan which includes providing an alternate water source, which may include a suitable replacement well, either by deepening the existing well, or installation of a new well. Alternatively, coordination of a connection to the municipal water service may also be considered.

Any wells which are deemed to no longer be suitable for use, should be decommissioned in accordance with the requirements of Ontario Regulation 903.

5.3 Hydrogeological Considerations

5.3.1 Impacts to Existing Surface Water Features

The Site is connected to the wetlands located on the north side of Sunset Drive (MN 5043) by a drain which crosses beneath Sunset Drive and terminates within the site limits (proximal to Borehole BH3.) Under the proposed post-development conditions, much of the surface is expected be covered with hard surfaces, comprised of buildings and paved parking areas, resulting in the existing drain being removed (or rerouted) as part of the site grading work.

For the realignment or rerouting of the existing drain, the need for maintaining stable embankment slopes and having adequate erosion control protection measures (such as erosion control blankets or addition of bonded fibre matrix on bare soils in proximity to realigned watercourse) should be anticipated.

Where possible, clean stormwater runoff (from roof-tops and landscaped areas) may be able to be directed towards the naturalized areas. When site grading work at the site is complete, and if shallow sandy soils are present and can be utilized to provide secondary infiltration opportunities, consideration for strategically located low impact development (LID) features is also recommended for consideration. This is discussed further in the following section.

5.3.2 Low Impact Development Considerations

Consideration should be given to utilize stormwater management options which promote opportunities for secondary infiltration or reduced run-off under post-development conditions. Measures such as drywells, open bottom catchbasins and infiltration galleries are generally considered well suited to the site, where clean stormwater runoff (from roof-tops and rear yards) can be directed to these types of features which would allow infiltration of the stormwater run-off into the natural sandy subgrade soils which underlie the near-surface silt till soils. There is a significant separation distance available between the bottom of these types of features and the stabilized groundwater table, therefore groundwater is not expected to cause any impairment to the operation of properly design LID features.

These types of LID (Low Impact Development) features could be used to help to mitigate the impacts of increased runoff and stormwater pollution by managing runoff as close to its source as possible. Additional strategies which provide opportunities for stormwater detention and evapotranspiration, such as lot grading controls may also be utilized to help attenuate run-off volumes.

As general guidance, LID features should be located no closer than four (4) metres from building foundations to reduce the likelihood of water damage, in accordance with the Zoning By-law and the Ontario Building Code. The design of LID features should also provide adequate separation from service trenches and other underground utilities.

The actual infiltration capacity of a soil depends on a number of factors, including particle size distribution, degree of saturation, compactness, adsorbed water (which depends on clay content). The heterogeneous nature of glacial deposits can also contribute to variations in soil permeability where the soil composition may include localized areas with increased fine material or sandy material which can influence soil permeability at different points within the soil strata. Based on the analyses conducted on collected samples from the site, the sandy soils encountered in the boreholes have a saturated hydraulic conductivity in the range of 10^{-5} m/s, with factored infiltration rates (FS = 2.5) in the range of 50 mm/hr.

It is important to note that the presence and effective depth of sandy soils may be altered by site grading activities at the site. The stormwater management strategy at the site will need to consider site grading activities at the site, which may alter the near-surface soil conditions, as a result of cut-fill activities to accommodate design grades.

Field confirmation of soil permeability with percolation testing and confirmation of the effective infiltration rates in the natural or reconstructed subgrade soils will need to be undertaken to confirm soil suitability for the enhanced infiltration-based LID strategies which are considered at the site.

5.3.3 Turbidity Monitoring

While active construction dewatering occurs at the site, a program which includes turbidity monitoring is recommended, to confirm that the quality of discharge water will not have adverse impacts to sensitive receptors. In the event that water discharged from the site is considered to have an elevated turbidity level, associated construction activities should be halted until remedial measures can be implemented. Such measures may include enhanced or more robust sediment and erosion control measures, incorporating pooling areas and measures that will reduce suspended solids, temporary storage measures to prevent off-site discharge.

5.3.4 Well Decommissioning

Monitoring wells associated with the preparation of this report have been installed at the site, to document stabilized groundwater conditions. These wells have been registered with MECP, and may be utilized for ongoing monitoring to document seasonal variations in the stabilized groundwater levels at the site.

A site plan showing all wells to be maintained and protected at the site will be provided to the contractors working at the site, to ensure that monitoring wells are not inadvertently damaged during the construction activities onsite.

When the monitoring wells are determined to be no longer required, the wells should be properly decommissioned in accordance with Ontario Regulation 903. This regulation identifies that only certified and qualified well drilling technicians are permitted to direct the decommissioning work for existing wells.

Decommissioning a well which is no longer in use helps to ensure the safety of those in the vicinity of the well, prevents surface water infiltration into an aquifer via the well, prevents the vertical movement of water within a well, conserves aquifer yield and hydraulic head and can potentially remove a physical hazard.

5.4 Water Quality Considerations

Baseline groundwater conditions (including general chemistry parameters) have not been established under the current scope of work for this investigation. Prior to construction, consideration may be given to carrying out baseline water quality sampling to establish the general chemistry and characteristics of the shallow groundwater, if encountered.

LDS is not aware of any contaminant plumes or existing environmental contamination in the vicinity of the site.

Construction activities at the site are generally not expected to impact the chemistry or bacteriological properties of the intermediate depth aquifer. However, the possibility exists that a spill or uncontrolled release of fuel or associated material could occur during construction, which could have a direct impact to the unconfined shallow to intermediate groundwater aquifer, or that sediment discharge could impact the effectiveness of stormwater infrastructure in the area. Additional comments are provided below, in this regard.

Given the naturally low permeability of the silt/clay soils which underlie the site (as described in the Little Creek Subwatershed Study), the deep overburden aquifers are not considered to be vulnerable to contamination from surface sources. However, shallow groundwater contained within sandy soils (such as those noted within the well records) may be more susceptible to water quality impacts as a result of surface activities during construction, since it does not have the benefit of a low-permeability protective soil layer above it.

5.4.1 Potential Impact from Construction Equipment

The possibility exists that a spill or uncontrolled release of fuel or associated material could occur during construction, which could have a direct impact to surface water and shallow groundwater conditions.

A Best Management Practice (BMP) and spill contingency plan (including a spill action response plan) should be in place for fuel handling, storage and onsite equipment maintenance activities. It is recommended that there be a designated equipment fuelling areas, and implementing a spill contingency plan (including a spill action response plan) for fuel handling, storage and onsite equipment maintenance activities to minimize the risk of contaminant releases as a result of the proposed construction activities.

It is important to note that if a spill (possible incident) is related to the contractor's activities, the contractor is responsible to report the incident to the Spills Action Centre, and/or notify the local MECP office. Depending on the type of incident, water sampling and quality testing may be warranted to document the extent of the impact. Scoping for the required testing will depend on the incident report.

5.4.2 Potential Impact from Uncontrolled Erosion / Sediment Discharge

Surface water quality can be detrimentally impacted by uncontrolled erosion and sediment discharge from the site. As such, it is imperative that an adequate Sediment and Erosion Control Strategy be established for the site. In addition to implementing sediment and erosion controls during construction, regular inspection and maintenance will also be necessary to ensure that sensitive receptors are not negatively impacted during construction.

Sediment and erosion control measures will be required to limit sediment discharge towards the natural features. It is important to ensure that the sediment control measures are installed properly, and in accordance with the design drawings. If deficiencies are identified in its performance through regular inspection, enhancements beyond the recommended design may be required.

The following table summarizes general mitigation measures are suggested as best management practices to limit foreseeable events where contamination or negative impacts to hydrologic features at the site may be possible.

Table 11 – Construction BMPs

Practice / Task	During Site Grading	During Site Servicing	During Home Construction & Partial Pavements	Following Construction
Delineate work areas to limit construction activities encroaching into the natural heritage features and setback areas, to prevent unnecessary vegetation removal.	✓	✓	✓	
Installing perimeter ESC measures such as silt fence and/or silt sock around temporary soil stockpiles, with dedicated points of access clearly marked onsite.	✓	✓		
Use of mud-mats at construction entrance/exit points to help control the amount of loose soil being carried offsite from construction vehicles	✓	✓		
Dedicated fuel storage and equipment fuelling areas located away from natural or otherwise sensitive features. Contractors should have an emergency spills management plan.	✓	✓		
Incorporate trench plugs/clay collars in servicing trenches to minimize groundwater migration through granular pipe bedding and disturbed backfill material.		✓		
Re-establishing vegetative cover in disturbed areas. In areas which are susceptible to erosion, additional measures may include the use of sod, hydroseeding, or mulch to protect the exposed subgrade soils.	✓	✓	✓	✓
Maintain perimeter silt fence (and other perimeter ESC measures) in place until disturbed areas and lots are sodded/seeded, and vegetative cover has become established.			✓	✓
Build-up boulevard areas to help limit sediment-laden stormwater run-off (from open or partially constructed lots) from discharging into catchbasins and stormwater infrastructure, and regular inspection and maintenance of silt bags/geotextile filters installed in catchbasins.			✓	✓

As construction work progresses at the site, regular maintenance and additional sedimentation measures may be required to limit the effect of siltation of run-off water in localized areas.

6.0 CLOSING

The geotechnical recommendations provided in this report are applicable to the project described in the text. LDS would be pleased to provide a review of design drawings and specifications to ensure that the geotechnical comments and recommendations provided in this report have been accurately and appropriately interpreted.

It is important to note that the geotechnical investigation involves a limited sampling of the subsurface conditions at specific borehole locations. The conclusions and recommendations presented in this report reflect site conditions existing at the time of the investigation and a review of available information which has been presented in the report. Should subsurface conditions be encountered which vary materially from those observed in the boreholes, we recommend that LDS be consulted to review the additional information and verify if there are any changes to the geotechnical recommendations.

The comments given in this report are intended to provide guidance for design engineers. Contractors making use of this report are responsible for their construction methods and practices, and should seek confirmation or additional information if required, to ensure that they understand how subsurface soil and groundwater conditions may affect their work.

No portion of this report may be used as a separate entity. It is intended to be read in its entirety.

We trust this satisfies your present requirements. If you have any questions or require anything further, please feel free to contact our office.

Respectfully Submitted,

LDS CONSULTANTS INC.

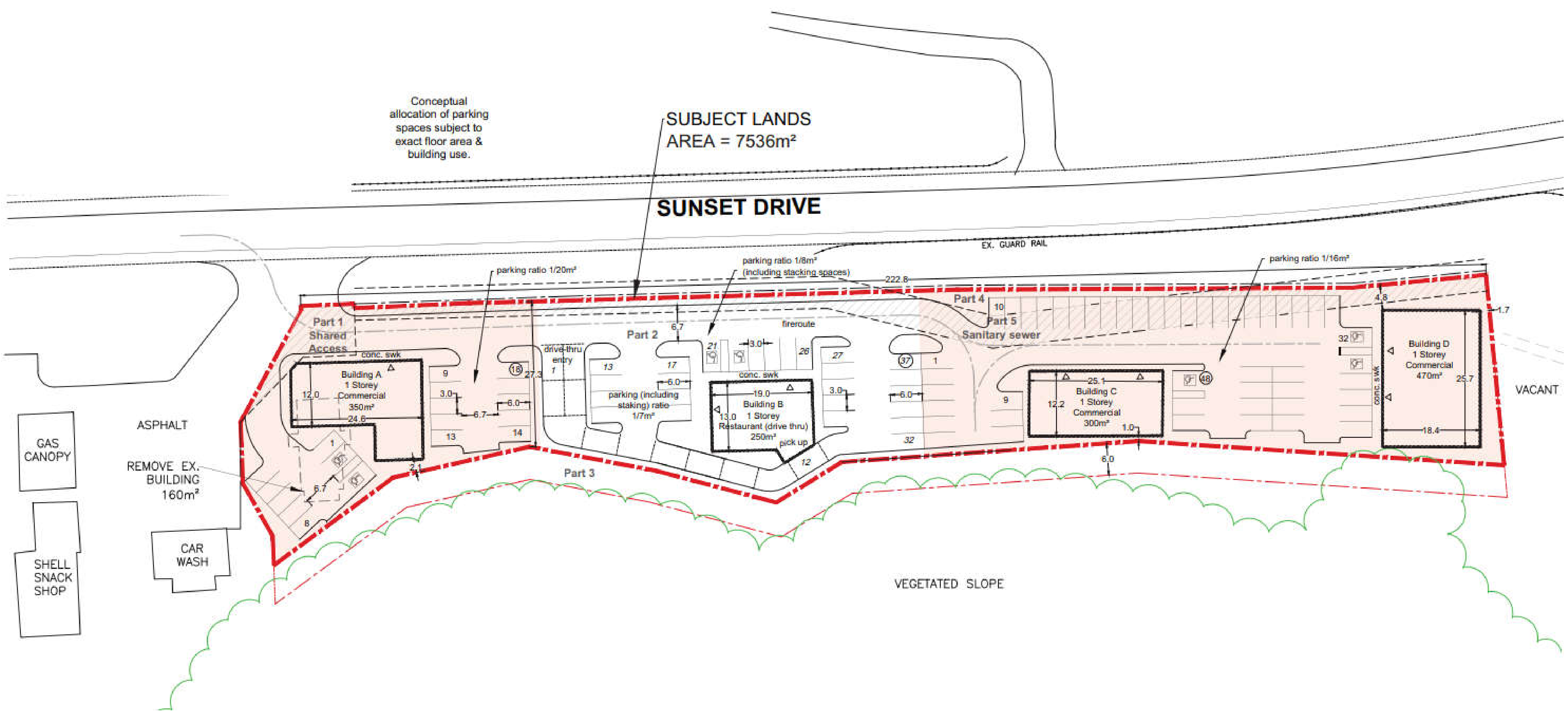
LICENSED PROFESSIONAL ENGINEER
Apr 4, 2022
R.A. WALKER
PROVINCE OF ONTARIO

Shaun M. Hadden, EIT.
Geotechnical Services
Office: 226-289-2952
Cell: 519-537-0039
shaun.hadden@LDSconsultants.ca

Rebecca A. Walker, P. Eng., QP$_{ESA}$
Principal, Geotechnical Services
Office: 226-289-2952
Cell: 519-200-3742
rebecca.walker@LDSconsultants.ca

APPENDIX A

DRAWINGS AND NOTES



SOURCE:
Produced from Preliminary Conceptual Development Plan, prepared by Monteith Brown Planning Consultants, London, Ontario, November 29, 2021

LDS

PROJECT NAME
Proposed Commercial Development

PROJECT LOCATION
4980 Sunset Drive,
Port Stanley, Ontario

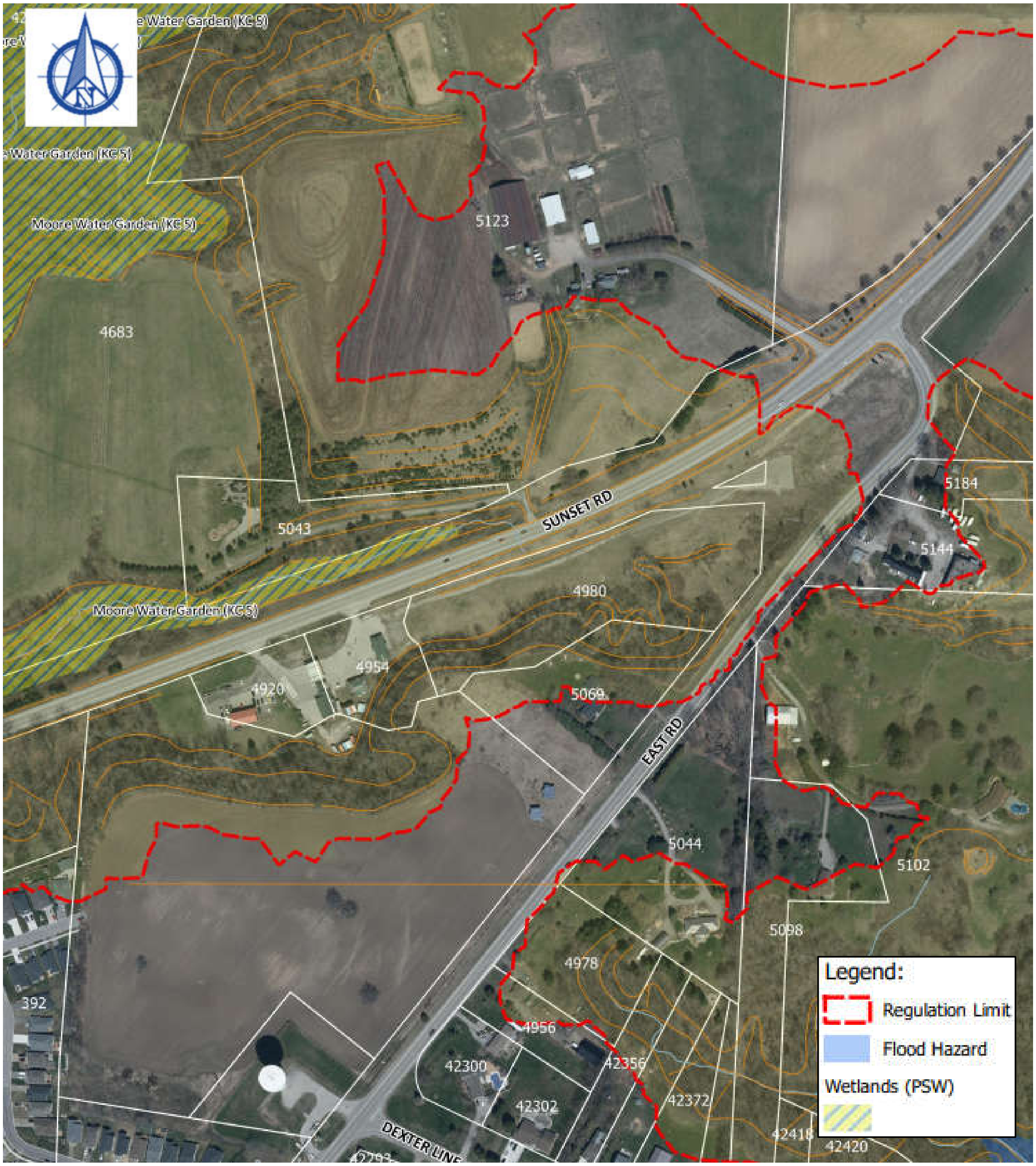
DRAWING NAME
Preliminary Concept Plan

SCALE	PROJECT NO.
1:300	GE-00677
DATE	**DRAWING NO.**
February 2022	1

SOURCE:
Google Earth Pro, Version 7.3.2.5776,
Coordinates 17T, 483870 m E, 4725295 m N,
Imagery date 7/2/2018

LDS

PROJECT NAME	Proposed Commercial Development
PROJECT LOCATION	4980 Sunset Drive, Port Stanley, Ontario
DRAWING NAME	Site Features
SCALE As Shown	**PROJECT NO.** GE-00667
DATE February 2022	**DRAWING NO.** 2



SOURCE: Kettle Creek Conservation Authority Online Interactive Mapping, February 2022

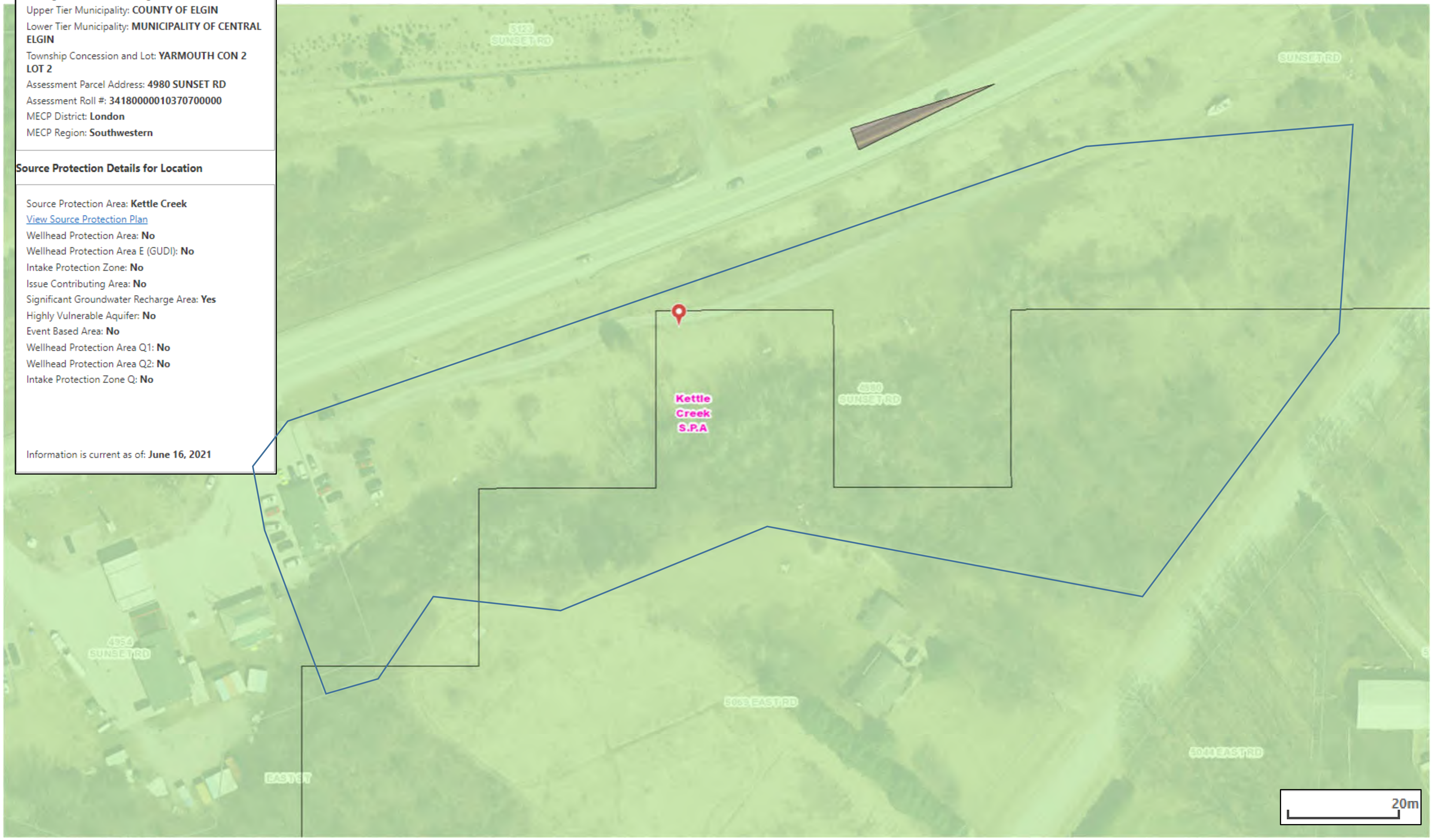
PROJECT NAME	**PROJECT LOCATION**	**SCALE**	**PROJECT NO.**
Proposed Commercial Development	4980 Sunset Drive, Port Stanley, Ontario	As Shown	GE-00667
LDS	**DRAWING NAME**	**DATE**	**DRAWING NO.**
	KCCA Regulated Lands	February 2022	3

Latitude: **42.68231** Longitude: **-81.20257**
UTM Zone: **17**
Easting: **483403.66** Northing: **4725556.73**
Upper Tier Municipality: **COUNTY OF ELGIN**
Lower Tier Municipality: **MUNICIPALITY OF CENTRAL ELGIN**
Township Concession and Lot: **YARMOUTH CON 2 LOT 2**
Assessment Parcel Address: **4980 SUNSET RD**
Assessment Roll #: **341800000103707000000**
MECP District: **London**
MECP Region: **Southwestern**

Source Protection Details for Location

Source Protection Area: **Kettle Creek**
View Source Protection Plan
Wellhead Protection Area: **No**
Wellhead Protection Area E (GUDI): **No**
Intake Protection Zone: **No**
Issue Contributing Area: **No**
Significant Groundwater Recharge Area: **Yes**
Highly Vulnerable Aquifer: **No**
Event Based Area: **No**
Wellhead Protection Area Q1: **No**
Wellhead Protection Area Q2: **No**
Intake Protection Zone Q: **No**

Information is current as of: **June 16, 2021**



LEGEND

Significant Groundwater Recharge Area
- 0
- 2
- 4
- 6

SOURCE:

Source Protection Information Atlas, Ministry of Environment, Conservation and Parks. Current to June 16, 2021.

LDS

PROJECT NAME

Proposed Commercial Development

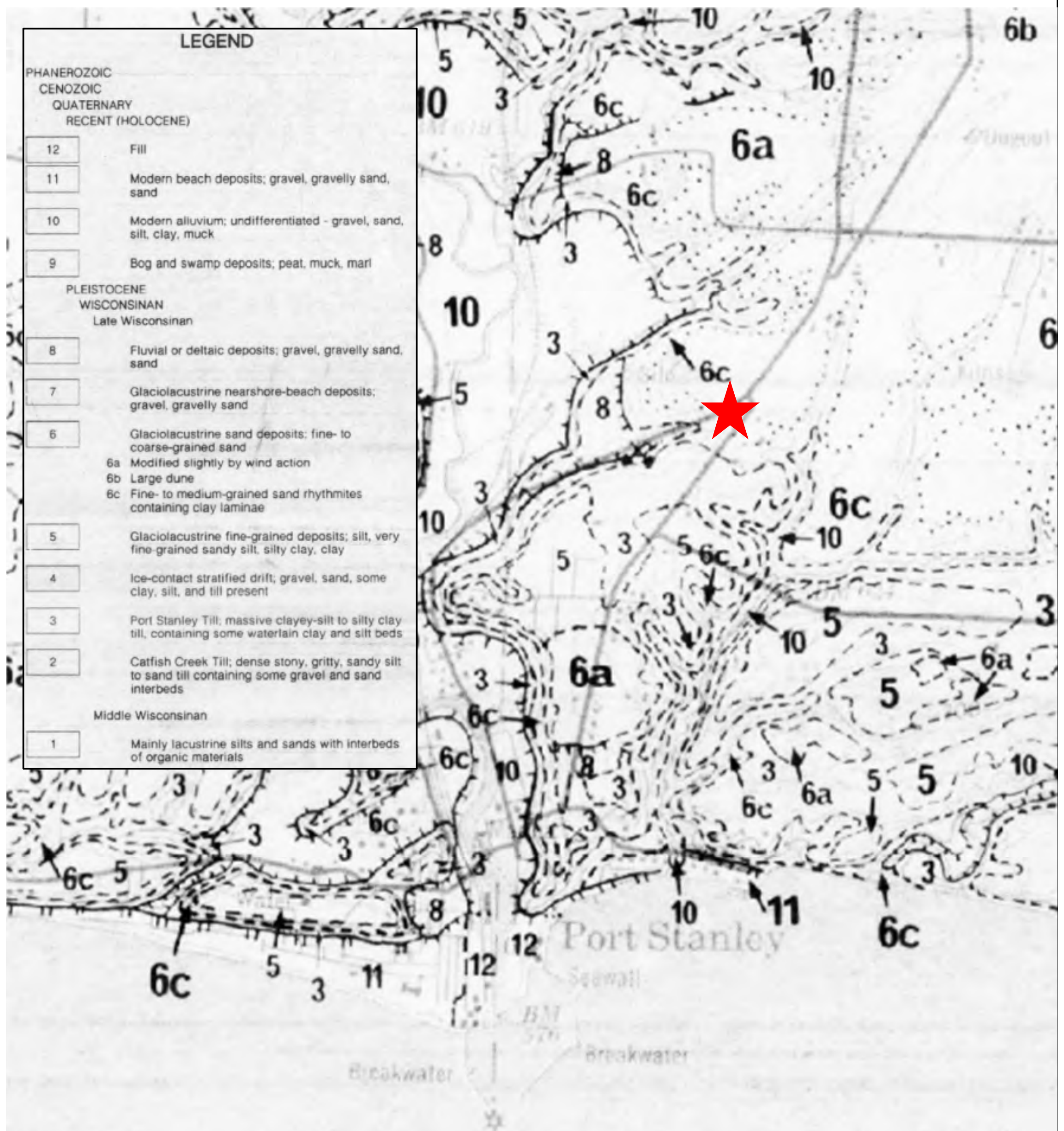
PROJECT LOCATION

4980 Sunset Drive,
Port Stanley, Ontario

DRAWING NAME

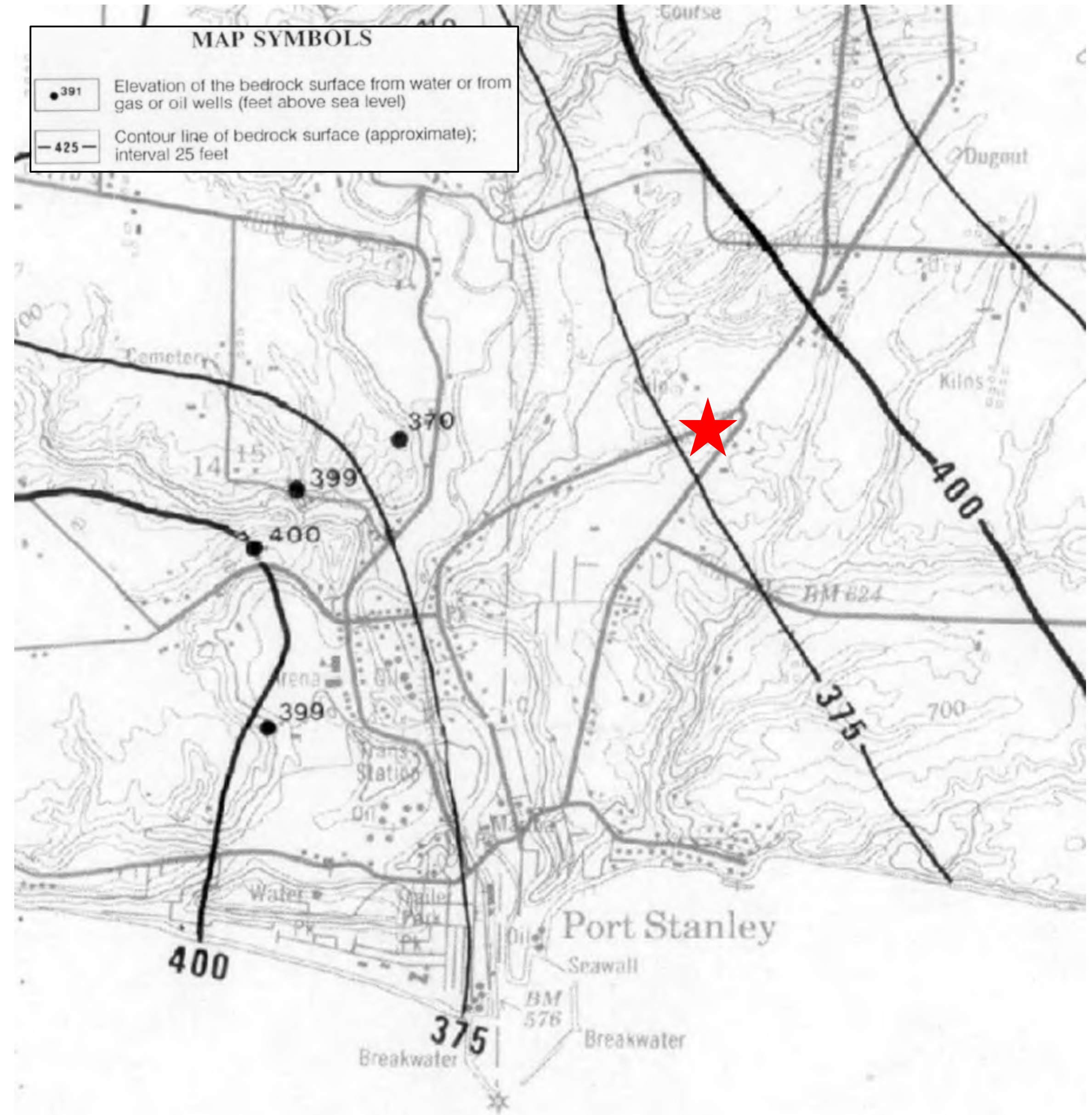
Kettle Creek Source Water Protection Mapping-MECP Information Atlas

SCALE	PROJECT NO.
As Shown	GE-00667
DATE February 2022	**DRAWING NO.** 4



SOURCE

Quaternary Geology, Port Stanley Area, Ontario Geological Survey Map P2827, Scale 1:50,00, © 1985



SOURCE

Bedrock Topography, Port Stanley Area, Ontario Geological Survey Map P3158, Scale 1:50,000, © 1990

PROJECT NAME	Proposed Commercial Development
PROJECT LOCATION	4980 Sunset Drive, Port Stanley, Ontario
DRAWING NAME	Geological Mapping
SCALE 1:50,000	**PROJECT NO.** GE-00667
DATE February 2022	**DRAWING NO.** 5

Location	Northing, m N	Easting, m E	Ground Surface Elevation (m asl)
BH1/MW	4725503.24	483268.71	201.39
BH2	4725522.19	483324.16	201.47
BH3/MW	4725544.22	483389.19	202.07
BH4	4725570.02	483445.51	203.39

Note: BH Locations surveyed by LDS

SOURCE:

Google Earth Pro, Version 7.3.2.5776,
Coordinates 17T, 483870 m E, 4725295 m N,
Imagery date 7/2/2018

LDS

PROJECT NAME

Proposed Commercial Development

PROJECT LOCATION

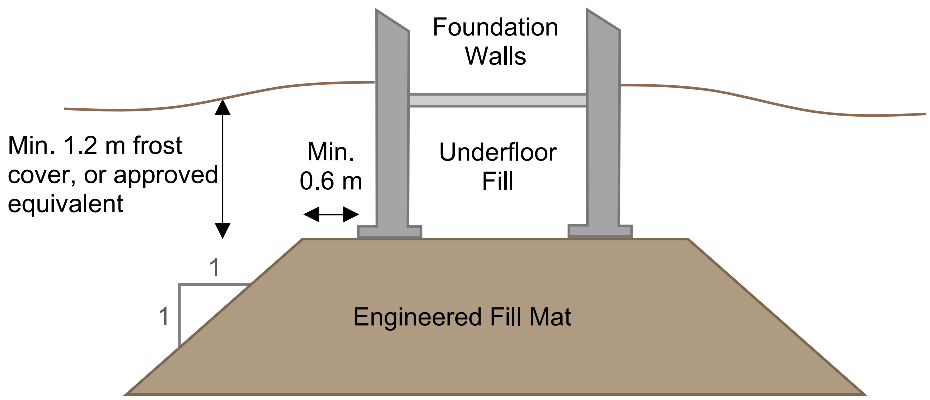
4980 Sunset Drive,
Port Stanley, Ontario

DRAWING NAME

Borehole Location Plan

SCALE	PROJECT NO.
As Shown	GE-00667
DATE	**DRAWING NO.**
February 2022	6

ENGINEERED FILL PLACEMENT
SCHEMATIC DIAGRAM



NOTES:

1. The area must be stripped of all topsoil contaminated fill material, and other unsuitable soils, and proof rolled. Soft spots must be dug out. The stripped natural subgrade must be examined and approved by the geotechnical consultant.
2. In areas where engineered fill is placed on a slope, the fill should be benched into the approved subgrade soils.
3. Material used for engineered fill must be free of topsoil, organics, frost and frozen material, and otherwise unsuitable or compressible soils, as determined by a Geotechnical Engineer. Any material proposed for use as engineered fill must be examined and approved prior to use onsite.
4. Engineered fill should be placed in maximum 300 mm thick lifts, and uniformly compacted to 100% Standard Proctor dry density. For best compaction results, engineered fill should be within 3 percent of its optimum moisture content, as determined by the Standard Proctor density test.
5. Full time geotechnical monitoring, inspection and in-situ density (compaction) is required during placement of the engineered fill.
6. Site grades should be maintained during area grading activities to promote drainage, and to minimize ponding of surface water on the engineered fill mat. Rutting by construction equipment should be kept to a minimum, where possible. Additional work to ensure suitability of engineered fill may be required if fill is placed in inclement weather conditions.
7. The fill must be placed such that the specified geometry is achieved. Refer to schematic diagram for minimum requirements. Environmental protection may be required, such as frost protection during construction, and after the completion of the engineered fill mat.
8. An allowable bearing pressure of 145 kPa (3000 psf) may be used provided that all conditions outlined above, and in the Geotechnical Report are adhered to.
9. These guidelines are to be read in conjunction with the attached Geotechnical Report prepared by LDS Consultants Inc.
10. For foundations set on engineered fill, footing enhancement and/or concrete reinforcing steel placement may be recommended. The footing geometry and extent of concrete reinforcing steel will depend on site specific conditions. In general, consideration may be given to having a minimum strip footing width of 500 mm (20 inches), containing nominal steel reinforcement.

LDS	PROJECT NAME	PROJECT NO.
	Proposed Commercial Development	GE-00667
	PROJECT LOCATION	**DRAWING NO.**
	4980 Sunset Drive, Port Stanley, Ontario	7

APPENDIX B

BOREHOLE LOGS & LABORATORY TEST RESULTS

NOTES ON SAMPLE DESCRIPTIONS

1. All descriptions included in this report follow the Canadian Foundation Engineering Manual soil classification system, based on visual and tactile examination which are consistent with the field identification procedures. Soil descriptions and classifications are based on the Unified Soil Classification System (USCS), based on visual and tactile observations. Where grain size analyses have been specified, mechanical grain size distribution has been used to confirm the soil classification.

Soil Classification (based on particle diameter)
Clay: < 0.002 mm
Silt: 0.002 – 0.075 mm
Sand: 0.075 – 4.75 mm
Gravel: 4.75 mm – 75 mm
Cobbles: 75 – 200 mm
Boulders: > 200 mm

Terminology & Proportion
Trace: < 10%
Some: 10-20%
Adjective, sandy, gravelly, etc.: 20-35%
And, and gravel, and silt, etc.: > 35%
Noun, Sand, Gravel, Silt, etc.: > 35% and main fraction

2. The compactness condition of cohesionless soils is based on excavator / drilling resistance, and Standard Penetration Test (SPT) N-values where available. The Canadian Foundation Engineering Manual provides the following summary for reference.

Compactness of Cohesionless Soils	**SPT N-Value** (# blows per 0.3 m penetration of split-spoon sampler)
Very Loose	0 – 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	50+

3. Topsoil Thickness - It should be noted that topsoil quantities should not be established from information provided at the test hole locations only. If required, a more detailed analysis with additional test holes may be recommended to accurately quantify the amount of topsoil to be removed for construction purposes.

4. Fill material is heterogeneous in nature, and may vary significantly in composition, density and overall condition. Where uncontrolled fill is contacted, it is possible that large obstructions or pockets of otherwise unsuitable or unstable soils may be present beyond the test hole locations.

5. Where glacial till is referenced, this is indicative of material which originates from a geological process associated with glaciation. Because of this geological process, till must be considered heterogeneous in composition and as such, may contain pockets and / or seams of material such as sand, gravel, silt or clay. Till often contains cobbles or boulders and therefore, contractors may encounter them during excavation, even if they are not indicated on the test hole logs. Where soil samples have been collected using borehole sampling equipment, it should be understood that normal sampling equipment can not differentiate the size or type of obstruction. Because of horizontal and vertical variability of till, the sample description may be applicable to a very limited area; therefore, caution is essential when dealing with excavations in till material.

6. Consistency of cohesive soils is based on tactile examination and undrained shear strength where available. The Canadian Foundation Engineering Manual provides the following summary for field identification methods and classification by corresponding undrained shear strength.

Consistency of Cohesive Soils	**Field Identification**	**Undrained Shear Strength** (kPa)
Very Soft	Easily penetrated several cm by the fist	0 – 12
Soft	Easily penetrated several cm by the thumb	12 – 25
Firm	Can be penetrated several cm by the thumb with moderate effort	25 – 50
Stiff	Readily indented by the thumb, but penetrated only with great effort	50 – 100
Very Stiff	Readily indented by the thumb nail	100 – 200
Hard	Indented with difficulty by the thumbnail	200+

LDS	Project	Proposed Commercial Development	Borehole ID
	Project Location	4980 Sunset Drive, Port Stanley	1/MW
	Project Number	GE-00667	Sheet 1 of 1

Date Drilled	**February 10, 2022**	Ground Surface Elevation	**201.39 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**4.2 m bgs**
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, EIT**

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
						TOPSOIL - dark brown, sandy loam, moist, 203 mm	
0.5						**SAND** - brown, fine grained, some silt, moist, loose	
1.0	SPT	1	70	9			*MC - 7.4%*
1.5						- becoming very moist and compact below 1.4 m depth	
2.0	SPT	2	70	19			*MC - 18.0%*
2.5	SPT	3	80	21			*MC - 18.3%*
3.0						- becoming saturated below 2.9 m depth	
3.5	SPT	4	80	10			*MC - 20.2%*
4.0							
4.5					*4.54 m (25-Feb-22)*	*Gradation: 0% Gravel, 82% Sand, 18% Fines (Silt/Clay)*	
5.0	SPT	5	80	18	***5.03 m***		*MC - 32.8%*
						BH Terminated at 5.03 m MW installed at 4.57 m - refer to details below	
5.5							
6.0							
6.5							
7.0							
7.5							
8.0							

Legend

- SPT Sample
- Bulk Sample
- Shelby Tube
- Stabilized Groundwater
- Inferred Groundwater

Well Construction Details

Pipe Diameter	50 mm CPVC Pipe
Installation Depth	4.57 m
Screen Length	3.05 m w/ No. 2 filter sand
Depth of Bentonite Seal	1.22 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

Water Levels:
February 25, 2022 - 4.54 mbgs
March 11, 2022 - 4.36 mbgs

LDS

Project	**Proposed Commercial Development**	Borehole ID
Project Location	**4980 Sunset Drive, Port Stanley**	**2**
Project Number	**GE-00667**	*Sheet 1 of 1*

Date Drilled	**February 10, 2022**	Ground Surface Elevation	**201.47 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**4.27 m bgs**
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, EIT**

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
						TOPSOIL - dark brown, sandy loam, moist, 203 mm	
0.5						**SAND** - brown, fine grained, trace silt, trace topsoil inclusions, very moist, compact	
1.0	SPT	1	70	14			*MC - 23.1%*
1.5						- becoming loose with no topsoil inclusions observed below	
	SPT	2	60	7			*MC - 20.8%*
2.0						- becoming very loose below 2.1 m depth	
2.5	SPT	3	70	3			*MC - 27.4%*
3.0						- becoming moist and loose below 2.9 m depth	
	SPT	4	70	5			*MC - 16.9%*
3.5							
4.0						- becoming saturated below 4.0 m depth	
4.5							
	SPT	5	70	15	*5.03 m*		*MC - 26.9%*
5.0						BH Terminated at 5.03 m Borehole observed open to 4.27 m depth at completion Water measured at 4.27 m depth at completion	
5.5							
6.0							
6.5							
7.0							
7.5							
8.0							

Legend

- SPT Sample
- Bulk Sample
- Shelby Tube
- Stabilized Groundwater
- Inferred Groundwater

Well Construction Details

Pipe Diameter	**no well installed**
Installation Depth	
Screen Length	
Depth of Bentonite Seal	

Additional Notes

MC - denotes moisture content

Project	Proposed Commercial Development	Borehole ID
Project Location	4980 Sunset Drive, Port Stanley	3/MW
Project Number	GE-00667	Sheet 1 of 1

Date Drilled	**February 10, 2022**	Ground Surface Elevation	**202.07 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**4.3 m bgs**
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, EIT**

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
						TOPSOIL - dark brown, sandy loam, moist, 203 mm	
0.5						**SAND** - brown, fine grained, trace silt, moist, loose	
1.0	SPT	1	70	7			*MC - 14.0%*
1.5						- becoming compact below 1.4 m depth	
2.0	SPT	2	70	15			*MC - 15.6%*
						- becoming saturated below 2.1 m depth	
2.5	SPT	3	80	12			*MC - 27.1%*
3.0						- becoming very moist below 2.9 m depth	
3.5	SPT	4	80	16			*MC - 17.5%*
4.0							
4.5					*4.47 m (25-Feb-22)*	*Gradation: 0% Gravel, 76% Sand, 24% Fines (Silt/Clay)*	
5.0	SPT	5	70	23	***5.03 m***		*MC - 14.7%*
5.5						BH Terminated at 5.03 m MW installed at 4.57 m - refer to details below	
6.0							
6.5							
7.0							
7.5							
8.0							

Legend

- SPT Sample
- Bulk Sample
- Shelby Tube
- Stabilized Groundwater
- Inferred Groundwater

Well Construction Details

Pipe Diameter	50 mm CPVC Pipe
Installation Depth	4.57 m
Screen Length	3.05 m w/ No. 2 filter sand
Depth of Bentonite Seal	1.22 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

Water Levels:
February 25, 2022 - 4.47 mbgs
March 11, 2022 - 4.38 mbgs

LDS	Project	**Proposed Commercial Development**	Borehole ID
	Project Location	**4980 Sunset Drive, Port Stanley**	**4**
	Project Number	**GE-00667**	*Sheet 1 of 1*

Date Drilled	**February 10, 2022**	Ground Surface Elevation	**203.39 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**Dry at completion**
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, EIT**

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
						TOPSOIL - dark brown, sandy loam, moist, 76 mm	
0.5						**FILL** - grey, silty sand, some organics, trace gravel, moist,	
1.0	SPT	1	70	14			*MC - 15.7%*
1.5						- becoming loose with trace silt observed below 1.4 m	
2.0	SPT	2	70	8		- becoming very moist below 2.1 m depth	*MC - 15.8%*
2.5	SPT	3	70	6	*2.90 m*		*MC - 19.5%*
3.0	SPT	4	70	11		**SAND** - brown, fine grained, trace silt, very moist, compact	*MC - 21.7%*
3.5							
4.0							
4.5	SPT	5	70	10	*5.03 m*		*MC - 11.3%*
5.0						BH Terminated at 5.03 m Borehole observed open and dry at completion	
5.5							
6.0							
6.5							
7.0							
7.5							
8.0							

Legend

- SPT Sample
- Bulk Sample
- Shelby Tube
- Stabilized Groundwater
- Inferred Groundwater

Well Construction Details

Pipe Diameter	**no well installed**
Installation Depth	
Screen Length	
Depth of Bentonite Seal	

Additional Notes

MC - denotes moisture content

Particle Size Distribution
Results of Sieve Analysis

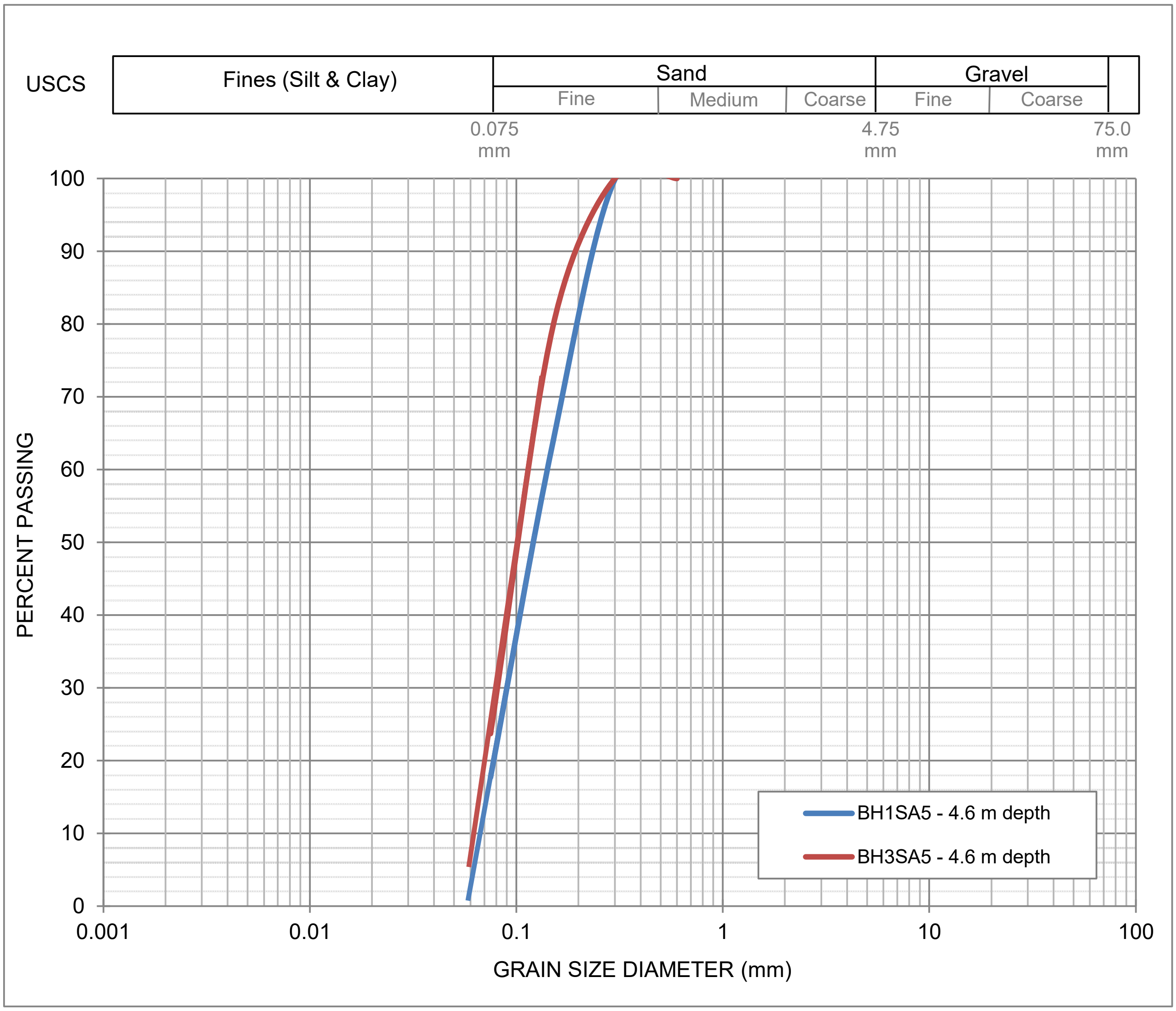
Project Name: Proposed Development

Date: 24-Feb-22

Project Location: 4980 Sunset Drive, Port Stanley

Project No.: GE-00667

Sample ID	Unified Soil Classification				Moisture Content (%)
	Fines (Silt & Clay)	% Sand	% Gravel	% Cobbles	
BH1SA5 - 4.6 m depth	17.7%	82.3%	0.0%	0.0%	32.8
BH3SA5 - 4.6 m depth	23.7%	76.3%	0.0%	0.0%	14.7



APPENDIX C

MECP WELL RECORD SUMMARY

MECP Water Supply Wells

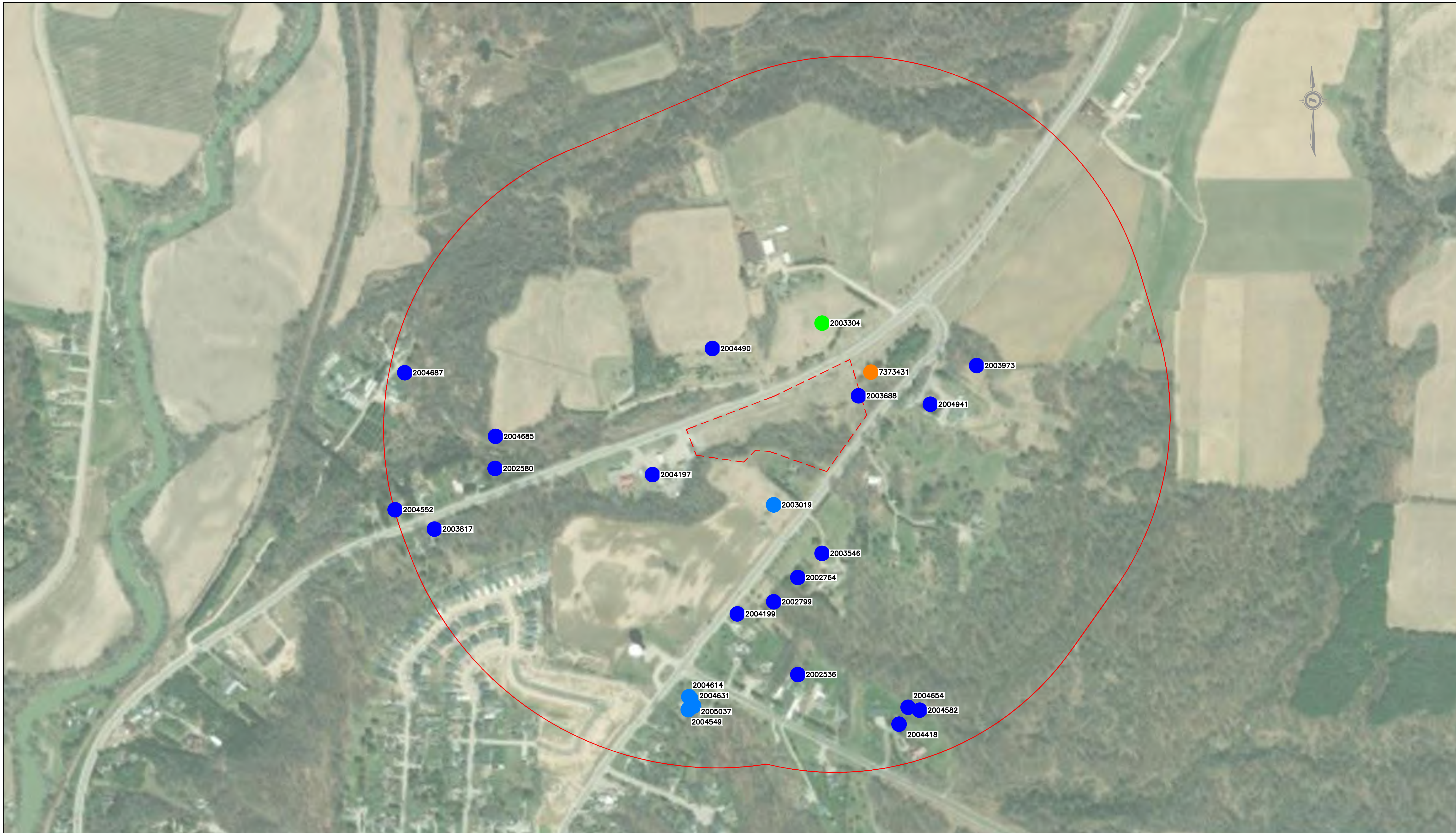
MECP Well ID	Registration Year	Well Type	Depth of Well (m)	Depth Water Found (m)	Static Water Level (m)	Pump Rate (L/min)
2004197	08/09/1989	Commercial	54.25	53.04	12.19	37.85
2003304	07/03/1979	Domestic / Livestock	8.53	7.01	4.27	26.5
2004685	07/08/1991	Domestic	51.21	50.29	15.24	7.57
2002580	06/30/1976	Domestic	50.6	49.38	3.05	22.71
2004552	08/01/1990	Domestic	49.68	47.24	9.75	18.93
2003817	03/26/1985	Domestic	74.98	74.68	12.19	3.79
2004687	07/02/1991	Domestic	40.84	39.93	*NR*	15.14
2003688	03/14/1983	Domestic	61.26	60.35	21.34	18.93
2003973	01/28/1987	Domestic	60.66	59.13	19.81	22.71
2004941	04/23/1993	Domestic	70.71	70.1	18.29	7.57
2003019	11/30/1977	Domestic	23.16	21.34	18.29	11.36
2003546	02/02/1981	Domestic	70.10	66.75	33.53	7.57
2002764	07/04/1977	Domestic	64.01	63.7	23.16	56.78
2002799	08/12/1977	Domestic	63.09	63.09	22.86	56.78
2004199	08/09/1988	Domestic	63.70	62.48	28.35	11.36
2002536	03/31/1976	Domestic	59.44	57.61	24.38	45.42
2004614	11/21/1990	Domestic	26.82	26.21	20.12	15.14
2004631	01/10/1991	Domestic	26.82	26.21	20.42	15.14
2005037	02/01/1994	Domestic	26.52	22.86	19.51	11.36
2004549	07/12/1990	Domestic	27.43	26.82	20.42	18.93
2004418	09/08/1989	Domestic	54.25	53.34	30.48	18.93
2004654	04/03/1991	Domestic	52.73	51.82	25.6	11.36
2004582	10/01/1990	Domestic	52.73	51.82	25.6	11.36

NR: Not recorded

MECP Test Holes and Abandonment Records

Well	Registration Year	Well Use	Depth of Well, m	Depth Water Found, m	Static Water Level, m	Pump Rate, lpm
2004490	02/07/1990	Test Hole	81.69	59.44	28.96	15.14
7373431	23-Nov-20	Test Hole	*NR*	*NR*	*NR*	*NR*

NR: Not Recorded



EXISTING SERVICES	DRAWING #, SOURCE	DATE	CONSTRUCTED SERVICES	COMPLETION	DETAILS	No.	REVISIONS	DATE	CONSULTANT
					DATE 2022-02-11				

GE-00667 – MECP Well Locations.dwg

CONSULTANT OR DIVISION

LDS

ENGINEER'S STAMP

Legend
- DEEP WELL (>30m)
- INTERMEDIATE WELL (15–30m)
- SHALLOW WELL (<15m)
- NON RECORDED DEPTH WELL
- PROPERTY OUTLINE
- 500m RADIUS FROM PROPERTY OUTLINE

SCALE
NTS

TITLE
MECP WELL LOCATIONS

4980 SUNSET DRIVE
PORT STANLEY, ONTARIO

PROJECT No.
GE–00667

SHEET No.
C1

PLAN FILE No.

Z:\GEOTECHNICAL SERVICES\PROJECTS\GE-00667 4980 SUNSET DR, PORT STANLEY\03 – PROJECT DRAWINGS\GE-00667 – MECP WELL LOCATIONS.DWG
2022-02-11 10:08:31 AM by: MATHEW.RUSSELL

EXISTING SERVICES	DRAWING #, SOURCE	DATE	CONSTRUCTED SERVICES	COMPLETION	DETAILS	No.	REVISIONS	DATE	CONSULTANT
					DATE 2022-02-11				

GE-00667 - MECP Water Supply Well Locations.dwg

CONSULTANT OR DIVISION

LDS

ENGINEER'S STAMP

Legend
- WATER SUPPLY WELL
- PROPERTY OUTLINE
- 500m RADIUS FROM PROPERTY OUTLINE

SCALE
NTS

TITLE
MECP WATER SUPPLY WELL LOCATIONS

4980 SUNSET DRIVE
PORT STANLEY, ONTARIO

PROJECT No.
GE-00667

SHEET No.
C2

PLAN FILE No.

APPENDIX D

SLOPE STABILITY RATING CHARTS

Slope Stability Rating Chart, A – A’

Geotechnical Principles for Stable Slopes
Ontario Ministry of Natural Resources

Site Location: 4980 Sunset Drive	**Project No.: GE-00667**
Town/City: Port Stanley, Ontario	**Inspection Date: February 25, 2022**
Inspected by: Rob Walker	**Weather: Overcast -5 °C**

	Rating Value	Slope Rating
Slope Inclination		
18 degrees or less (3H:1V or flatter)	0	
18 to 28 degrees (2H:1V to 3H:1V)	6	
28 degrees or more (steeper than 2H:1V)	16	**16**
Soil Stratigraphy		
shale / limestone	0	
sand, gravel	6	**6**
till	9	
clay, silt	12	
fill	18	
leda clay	24	
Seepage from Slope Face		
none, or near bottom only	0	**0**
near mid-slope only	6	
near crest only, or from several levels	12	
Slope Height		
2 m or less	0	
2.1 to 5 m	2	
5.1 to 10 m	4	
more than 10 m	8	**8**
Vegetation Cover on Slope Face		
well vegetated: heavy shrubs or forested with mature trees	0	**0**
light vegetation: grass, weeds, occasional trees, shrubs	4	
no vegetation: bare	8	
Table Land Drainage		
table land flat, no apparent drainage over slope	0	
minor drainage over slope, no active erosion	2	
drainage over slope, active erosion, gullies	4	**4**
Proximity of Watercourse to Slope Toe		
15 m or more from slope toe	0	**0**
Less than 15 m from slope toe	6	
Previous Landslide Activity		
No	0	**0**
Yes	6	
Slope Instability Rating		**34**

Low Potential	< 24	Site Inspection only, confirmation, report letter
Slight Potential	25-35	Site Inspection and surveying, preliminary study, detailed report
Moderate Potential	> 35	BH Investigation, piezometers, lab tests, surveying, detailed report

Notes:
Is there is a water body (stream, creek, river, pond, bay, lake) at the toe of slope?
If YES - the potential for toe erosion and undercutting should be evaluated in detail.

Slope Stability Rating Chart, B – B’

Geotechnical Principles for Stable Slopes
Ontario Ministry of Natural Resources

Site Location: 4980 Sunset Drive	**Project No.: GE-00667**
Town/City: Port Stanley, Ontario	**Inspection Date: February 25, 2022**
Inspected by: Rob Walker	**Weather: Overcast -5 °C**

	Rating Value	Slope Rating
Slope Inclination		
18 degrees or less (3H:1V or flatter)	0	
18 to 28 degrees (2H:1V to 3H:1V)	6	
28 degrees or more (steeper than 2H:1V)	16	**16**
Soil Stratigraphy		
shale / limestone	0	
sand, gravel	6	**6**
till	9	
clay, silt	12	
fill	18	
leda clay	24	
Seepage from Slope Face		
none, or near bottom only	0	**0**
near mid-slope only	6	
near crest only, or from several levels	12	
Slope Height		
2 m or less	0	
2.1 to 5 m	2	
5.1 to 10 m	4	
more than 10 m	8	**8**
Vegetation Cover on Slope Face		
well vegetated: heavy shrubs or forested with mature trees	0	**0**
light vegetation: grass, weeds, occasional trees, shrubs	4	
no vegetation: bare	8	
Table Land Drainage		
table land flat, no apparent drainage over slope	0	
minor drainage over slope, no active erosion	2	
drainage over slope, active erosion, gullies	4	**4**
Proximity of Watercourse to Slope Toe		
15 m or more from slope toe	0	**0**
Less than 15 m from slope toe	6	
Previous Landslide Activity		
No	0	**0**
Yes	6	
Slope Instability Rating		**34**

Low Potential	< 24	Site Inspection only, confirmation, report letter
Slight Potential	25-35	Site Inspection and surveying, preliminary study, detailed report
Moderate Potential	> 35	BH Investigation, piezometers, lab tests, surveying, detailed report

Notes:
Is there is a water body (stream, creek, river, pond, bay, lake) at the toe of slope?
If YES - the potential for toe erosion and undercutting should be evaluated in detail.

Slope Stability Rating Chart, C – C'

Geotechnical Principles for Stable Slopes
Ontario Ministry of Natural Resources

Site Location: 4980 Sunset Drive	**Project No.: GE-00667**
Town/City: Port Stanley, Ontario	**Inspection Date: February 25, 2022**
Inspected by: Rob Walker	**Weather: Overcast -5 °C**

	Rating Value	Slope Rating
Slope Inclination		
18 degrees or less (3H:1V or flatter)	0	
18 to 28 degrees (2H:1V to 3H:1V)	6	
28 degrees or more (steeper than 2H:1V)	16	**16**
Soil Stratigraphy		
shale / limestone	0	
sand, gravel	6	**6**
till	9	
clay, silt	12	
fill	18	
leda clay	24	
Seepage from Slope Face		
none, or near bottom only	0	**0**
near mid-slope only	6	
near crest only, or from several levels	12	
Slope Height		
2 m or less	0	
2.1 to 5 m	2	
5.1 to 10 m	4	
more than 10 m	8	**8**
Vegetation Cover on Slope Face		
well vegetated: heavy shrubs or forested with mature trees	0	**0**
light vegetation: grass, weeds, occasional trees, shrubs	4	
no vegetation: bare	8	
Table Land Drainage		
table land flat, no apparent drainage over slope	0	
minor drainage over slope, no active erosion	2	
drainage over slope, active erosion, gullies	4	**4**
Proximity of Watercourse to Slope Toe		
15 m or more from slope toe	0	**0**
Less than 15 m from slope toe	6	
Previous Landslide Activity		
No	0	**0**
Yes	6	
Slope Instability Rating		**30**

Low Potential	< 24	Site Inspection only, confirmation, report letter
Slight Potential	25-35	Site Inspection and surveying, preliminary study, detailed report
Moderate Potential	> 35	BH Investigation, piezometers, lab tests, surveying, detailed report

Notes:
Is there is a water body (stream, creek, river, pond, bay, lake) at the toe of slope?
If YES - the potential for toe erosion and undercutting should be evaluated in detail.

LDS CONSULTANTS INC.

2323 Trafalgar Street
London, Ontario N5V 0E1

www.ldsconsultants.ca