STONECAIRN CONSULTING
85 Midpark Road – Unit 3, London, Ontario N6N 1B2

GEOTECHNICAL INVESTIGATION & HYDROGEOLOGICAL ASSESSMENT

PROPOSED RESIDENTIAL & RECREATIONAL DEVELOPMENT
320 CARLOW ROAD
PORT STANLEY, ONTARIO

PROJECT NO. SC-02117

FEBRUARY 12, 2025

Submitted to:

G-LOVER HOLDINGS INC.
6297 OLDE DRIVE
APPIN, ONTARIO N0L 1A0

Distribution (via email):

James Glover – jamesgluv@gmail.com
Patrick Matkowski – pmatkowski@mbpc.ca

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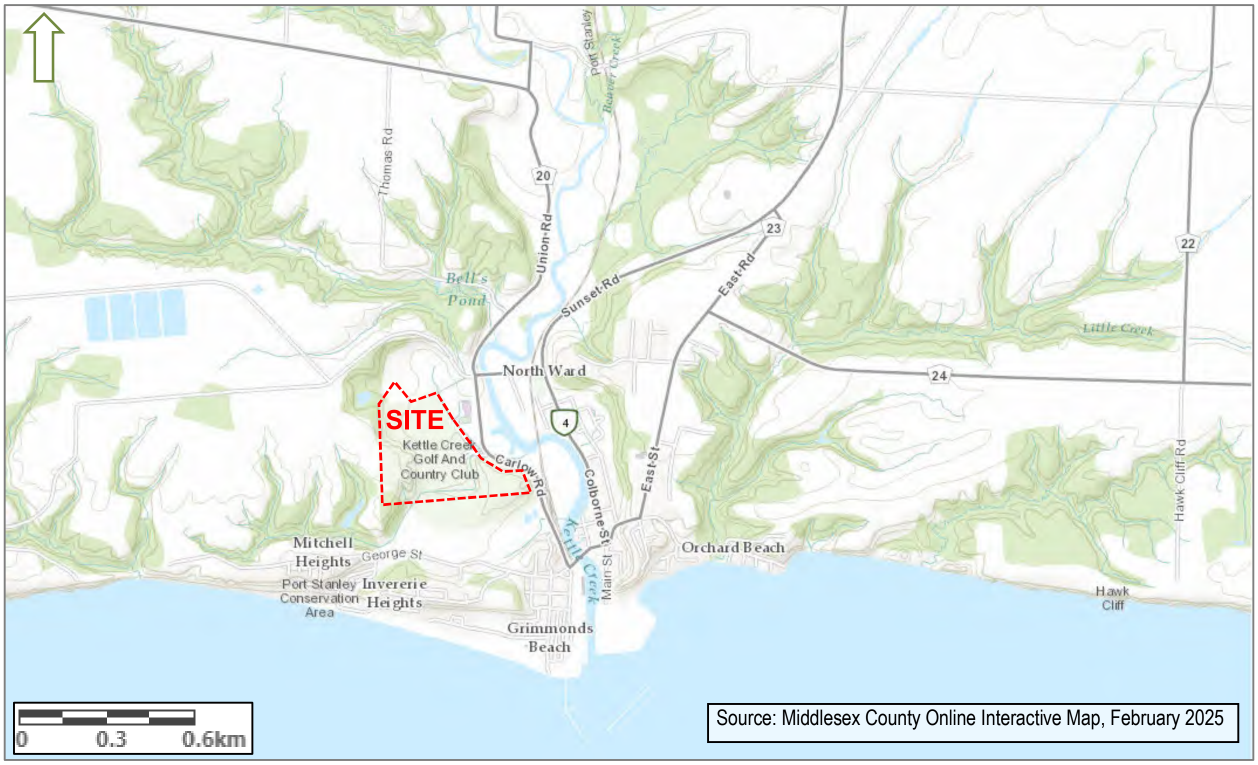
1. INTRODUCTION

Stonecairn Consulting Inc. [Stonecairn] has been retained by G-Lover Holdings Inc. to conduct a Geotechnical Assessment for a proposed residential and recreational development. Preparation of this report relies partially on information collected by LDS Consultants Inc., which ceased operations in September 2024. This report was prepared by the supervising engineer who oversaw the original geotechnical site assessment work, and provides updated information to satisfy current requirements.

The subject property is located northwest of the intersection of Carlow Road and Bridge Steet, in the community of Port Stanley, Municipal Number (MN) 320 Carlow Road. A Key Plan showing the general site location is provided on Figure 1, below.

Figure 1: Key Plan



Source: Middlesex County Online Interactive Map, February 2025

It is understood that consideration is being given to re-develop the lands as a mixed-density residential plan of subdivision. It is understood that the development will be accessed via local roadways, and serviced with municipal sewers and water supply. A stormwater management facility is expected to be located in the easterly extents of the site. A Draft Plan of Subdivision is provided on Drawing 1, appended.

Authorization to complete this Investigation was received from Mr. James Glover, on behalf of G-Lover Holdings Inc.

1.1 Terms of Reference

This report has been prepared for the purposes of providing geotechnical comments and recommendations for the design and construction of a proposed residential & recreational development located at MN 320 Carlow Road, in Port Stanley.

In preparing this report, Stonecairn was provided with the following documents:

- Draft Plan of Subdivision, prepared Monteith Brown Planning Consultants, February 2025; and,
- Summary of Meeting Notes from Pre-Application Consultation Meeting, dated July 20, 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 residential & recreational development, including:

- Site preparation, including guidance for cut and fill operations, the re-use of excavated materials as engineered fill / structural fill and guidance for engineered fill placement;
- Temporary excavations, including maximum slope inclinations to provide stable excavation side slopes in accordance with OHSA requirements, excavation support (shoring methods, if required), and lateral earth pressures;
- Groundwater Control, including the need for a Permit to Take Water (PTTW) or Environmental Activity Sector Registry (EASR) submission for construction dewatering, if required;
- Foundation design, including soil bearing capacity, subgrade preparation and allowable settlements;
- Concrete slab and basement construction (including lateral earth pressures and provisions for shallow groundwater conditions);
- Seismic design considerations based on borehole data and published information for soil conditions below the depth of exploration;
- Site servicing, including recommendations for pipe bedding and trench backfill;
- Pavement design recommendations for local roadways, construction access routes, and restoration of existing site pavements where servicing tie-ins may be expected to occur,
- Excess soil management discussion to assist contractors in understanding the characteristics of excess soils which may be generated from onsite excavations, and which may require disposal offsite.

This report also includes the slope stability analysis which was carried out for the existing slopes along the southern property limits. The analysis confirms the Erosion Hazard Limit, in accordance with MNR guidance documents and KCCA policies, and has been used to determine the development setback from the existing site slopes.

The report also provides information about the characterization of the hydrogeological setting for the site, including:

- Construction dewatering discussion, including estimated construction dewatering volumes, potential zones of influence, and confirming the requirements for permitting required to carry out site servicing which may encounter shallow groundwater conditions;
- Stormwater management considerations, and factored soil infiltration rates for at-source infiltration features, including a discussion on limitations which result from soil and/or shallow groundwater conditions; and providing recommendations for best management practices during construction and the inclusion of at-source infiltration and/or LID measures (where site conditions permit) to increase post-development infiltration volumes.
- Mitigation measures will also be discussed to address concerns with contamination which could result from typical construction activities and the installation of site services to conventional depths for this type of development.

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 site investigation and recommendations provided in this report follow generally accepted practice for geotechnical consultants in Ontario. The format and content of this report has been guided to address specific client needs.

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 25 years of direct experience in the geotechnical and hydrogeological consulting industry. Over 5,200 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

Based on a review of aerial photographs dated from 2006 to current, the subject lands have been occupied by the Kettle Creek Golf & Country Club. The site is bordered by future residential developments to the north and south, agricultural lands to the west, and a Public School, Community Centre and Carlow Road to the east. The site is irregular in shape, and comprises an area of approximately 74.1 acres.

The site includes a vell vegetated slope, which runs along the southwestern limits of the site, with the upper tablelands currently being accessed via an existing cart path. From a topographical perspective, the ground surface exhibits a relief of 32 meters from the top of the slope to the southeast and northeast. Any minor surface flows which occur at the site under existing conditions, are generally expected to follow the topography of the site.

A surface water pond (associated with the golf course) is located within the central limits of the site. Furthermore, an open channel of the Marr Drain corridor runs along the eastern and southern limits of the site, and conveys flow to the west/southwest, towards Lake Erie.

Site features are identified on the aerial photograph provided on Drawing 2, in Appendix A.

2.2 KCCA Generic Regulation

In April 2024, Ontario Regulation 41/24 came into effect as part of the significant amendments which were introduced to the Conservation Authorities Act, which included amendments to the jurisdiction of conservation authorities in Ontario, and replaced the thirty-six various separate regulation which governed each of the conservation authorities. This regulation is intended to ensure public safety, prevent property damage and social disruption, due to natural hazards such as flooding and erosion.

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.

Discussion presented in Section 4.9 of this report provides the Slope Stability analysis conducted for the valleyland slopes which border the site, and which have defined the Erosion Hazard Limit, as defined from a geotechnical standpoint. It is noted that additional ecological setbacks may apply in determining the ultimate development setback from the valleylands.

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). 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. A highly vulnerable aquifer occurs where the subsurface material offers limited protection from contamination resulting from surface activities.

MECP Source Water Protection Information has been reviewed to determine whether the site is in any identified areas of source water concern, as they relate to local groundwater quality (current to December 12, 2024). The subject properties are located within the Kettle Creek Source Protection Area, and the following observations are noted for the site:

- The subject 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.

Portions of the site fall within Significant Groundwater Recharge Area. This is denoted in yellow shading on Drawing 4, in Appendix A.

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.

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 subject soils within the southwestern limits of the site are predominantly comprised of Glaciolacustrine sand deposits, comprised of fine to medium-grained sand containing clay laminae, transitioning to Modern Alluvium deposits, comprised of gravel, sand, silt and clay, within the central and northern limits of the site, transitioning again to fluvial/deltaic deposits, comprised of gravel, gravelly sand and sand, along the south-eastern limits of the site. An excerpt from the mapping is provided on Drawing 5, in Appendix A.

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 60-92 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 shows the location of the wells (with corresponding Well Registration No.) which are in proximity (within 500 m) of the site. The supply wells are summarized in Appendix C, for reference.

The water supply wells noted in the records are set in the shallow (<15 m depth), intermediate (15-30 m depth) and deep (> 30 m depth) overburden and limestone bedrock aquifers, with reported static water levels ranging between 0.3 m and 4.3 m, 1.5 m and 16.5 m, and 0.3 and 21.0 m, respectively. Several observation wells (located on the parcels of land directly south and southeast of the site) are set in the shallow overburden aquifer, with subgrade soils described as topsoil overlying natural silt, sand and silt/clay till soils, with static water levels ranging between 1.5 m and 3.0 m. Some water supply wells in the vicinity of the site have been abandoned, following access to municipal water supply/serving which is now available in the area.

3. SUMMARIZED CONDITIONS

3.1 Field Program and Laboratory Testing

Geotechnical 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, as well as a private locator, in preparation for the drilling program.

The field program consisted of a series of boreholes, drilled on March 27 & 29, and May 10, 2023. Two additional boreholes were added in January 2025 to confirm the soil conditions in the area of the Port Stanley Arena parking lot, where site servicing for the subdivision is expected to run for connection to municipal services.

The boreholes were advanced at the site by a local drilling-contractor, using a track-mounted drill-rig. The boreholes were advanced to depths ranging from 5.0 m (16.5 feet) to 18.7 m (61.5 feet) below existing grade. The fieldwork was supervised by geotechnical field staff.

Ground surface elevations at the borehole locations were surveyed using a Trimble R12 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	4724097.56	481368.24	211.66
BH2 *(not drilled)*	-	-	-
BH3	4724494.56	481370.14	182.51
BH4/MW	4724644.82	481366.22	183.85
BH5/MW	4724625.11	481594.29	178.86
BH6	4724507.75	481576.17	180.10
BH7/MW	4724303.45	481565.77	180.17
BH8	4724171.94	481514.25	185.13
BH9	4724065.58	481694.46	181.73
BH10/MW	4724192.74	481825.00	181.38
BH11	4724432.59	481718.19	179.61
BH101	4724451.24	481797.59	178.41
BH102	4724446.92	481751.73	178.50

Due to drilling productivity and mobilization restrictions (to minimize damage to the existing grassed areas) Borehole BH2 was removed from the investigation program at the time of drilling.

Monitoring wells were installed in five of the boreholes (BH1, BH4, BH5, BH7, and BH10) 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	211.66	18.29	3.05	Sand with trace silt, and Silt
BH4/MW	183.85	6.10	3.05	Sandy Silt
BH5/MW	178.86	4.27	1.52	Sand, and Silt
BH7/MW	180.17	4.27	1.52	Sand and Gravel, and Silt
BH10/MW	181.38	9.14	1.52	Silty Sand, and Sand and Gravel

The depth to groundwater seepage and short-term water level measurements were obtained prior to backfilling the 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 the laboratory for detailed examination and selective testing. Four (4) 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 the Geotechnical Report, unless prior arrangements have been made for longer term storage.

3.1.1 Soil Conditions

A series of ten 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 overlying interlayered deposits of sand, silt, and sand and gravel, overlying silt till. 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.

Pavement Structure

Boreholes BH101 and BH102 were drilled in the existing parking lot of the Port Stanley Arena. The boreholes confirmed that the existing pavement structure is comprised of approximately 50 mm of asphalt overlying 305 to 485 mm of granular material.

Topsoil

Each borehole in the site was surfaced with a layer of topsoil. The topsoil consisted of dark brown silty loam, and the thickness generally ranging from 125 to 200 mm across the site. The topsoil was in a damp to 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 2.5 m thick layer of silt fill (with topsoil inclusions) was contacted below the site pavements within borehole BH1. The fill was generally found to be in a loose condition, with SPT N-values in the range of 3 to 9 blows per 0.30 m penetration of the split-spoon sampler. Moisture contents in the fill were reported in the range of 27 to 30 percent, indicating very moist conditions.

Sand / Silty Sand

A natural sand layer was encountered underlying the topsoil in each borehole, with the exception of Boreholes BH3 and BH4. Borehole BH8 terminated within this layer. The sand was generally described as being brown to grey in colour, stratified, with a fine to medium grained texture, and containing some silt to silty, with a noted decrease in silt content with depth. A sample of the sand layer was submitted for gradation analyses, 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
BH5, Sample 2 – 1.5 m depth	13.0	77.6	9.4	0.0

Note: Particle size determination in accordance with the Unified Soil Classification System

The sand was in a variable loose to dense state, based on Standard Penetration Test (SPT) N-values in the range of 5 to 43 blows per 0.3 m of split-spoon sampler penetration. Very loose (SPT N-value < 4 blows) soil conditions were encountered within the sand layer in boreholes BH7, BH9 and BH10 within the upper 4.0 m below ground surface. Moisture content determinations conducted on recovered samples of the sand generally range between 5.2 to 31.4 percent, generally indicative of damp to saturated soil conditions.

Marl

A layer of marl was contacted within the sand layer in boreholes BH7, BH9 and BH10, and in the Port Stanley Arena parking lot in boreholes BH101 and BH102. The marl ranges in thickness between 0.2 and 2.0 m. The marl is described as being fine grained, with a sandy texture, and containing organic inclusions (wood fragments and small shells.). The marl was generally found to be in a very loose state, based on Standard Penetration Test (SPT) N-values with typically less than 5 blows per 0.30 m penetration of the split-spoon sampler. The marl was generally found to be in a wet to saturated state, with in-situ moisture contents in the range of 35.4 to 126.1 percent.

Sand and Gravel

A layer of sand and gravel was encountered at the base of the sand layer in Boreholes BH7, BH9 and BH10. Borehole BH9 terminated within this layer. Sand and gravel was also noted in the Parking Lot area at 3.4 to 4.0 m depth in Boreholes BH101 and BH102. The sand and gravel soils were described as grey in colour, with a medium to coarse grained texture, and containing trace silt. A sample of the sand and gravel layer was submitted for gradation analysis, and the following table shows the grain size distribution. The results are also shown graphically in Appendix B.

Table 4 – Gradation Summary, Sand and Gravel

Sample ID	Unified Soil Classification			
	% Silt	% Sand	% Gravel	% Cobbles
BH10, Sample 7 – 7.6 m depth	6.3	58.9	34.8	0.0

Note: Particle size determination in accordance with the Unified Soil Classification System

The sand and gravel is in a loose to compact state, based on SPT N-values in the range of 5 to 14 blows per 0.3 m of split-spoon sampler penetration. Moisture content determinations conducted on recovered samples of the silt generally range between 9.2 to 32.6 percent, generally indicative of moist to saturated soil conditions.

Silt/Sandy Silt

A layer of silt was encountered underlying the topsoil in Boreholes BH3, BH4 and BH6, and interlayered with the near surface sandy soils in Boreholes BH1 and BH7. Boreholes BH1, BH3, BH4, BH5 and BH7 terminated within this layer. The silt encountered near surface was described as mottled brown/grey and weathered, becoming brown to grey with depth. The silt is described as containing some sand, with a noted increase in silt content with depth. Two samples of the silt layer were submitted for gradation analyses, and the following table shows the grain size distribution. The results are also shown graphically in Appendix B.

Table 5 – Gradation Summary, Silt

Sample ID	Unified Soil Classification			
	% Silt	% Sand	% Gravel	% Cobbles
BH1, Sample 11 – 18.3 m depth	70.9	29.1	0.0	0.0
BH4, Sample 6 – 6.1 m depth	64.3	33.3	2.4	0.0

Note: Particle size determination in accordance with the Unified Soil Classification System

The silt is generally in a variable very loose to compact state, based on Standard Penetration Test (SPT) N-values in the range of 2 to 24 blows per 0.3 m of split-spoon sampler penetration. Dense (SPT N-value > 30 blows) soil conditions were encountered within the silt layer in borehole BH5 at a depth of 4.0 m below ground surface. Moisture content determinations conducted on recovered samples of the silt generally range between 22.4 to 33.8 percent within the near surface weathered zone, and on the order of 16.2 to 25.5 percent below the weathered soils.

Silt Till

A layer of glacial silt till was encountered underlying the sand/sand and gravel soils in boreholes BH6, BH10, and BH11, and each of these boreholes terminated within this layer. The silt till was described as grey in colour, and containing trace to some sand, and trace fine gravel. The till is generally in a compact to dense state, based on SPT N-values in the range of 16 to 40 blows per 0.3 m of split-spoon sampler penetration. Moisture content determinations conducted on recovered samples of the till generally range between 14.4 to 20.2 percent, generally indicative of moist to very moist soil conditions.

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.

The soil permeability of select sand samples was assessed by two methods. The first method is correlation of hydraulic conductivity and factored infiltration rates based on the results of gradation analyses on collected samples. The second method is with field measurements during single well response testing.

Grain Size Analyses

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, and silt and sand soils.

This correlation is based on the following relationship:

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

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 6 – Hydraulic Conductivity and Factored Infiltration Rates from Grain Size Analyses

Sample ID	Sample Composition			Parameter		
	% Fines (Si & Cl)	% Sand	% Gravel	D_{10} (mm)	Saturated Hydraulic Conductivity (m/sec)	Factored Infiltration Rate (mm/hr)
Silt/Sandy Silt						
BH1, Sample 11 – 18.3 m depth	70.9	29.1	0.0	0.019	3.61×10^{-6}	26
BH4, Sample 6 – 6.1 m depth	64.3	33.3	2.4	0.004	1.23×10^{-7}	11
Sand						
BH5, Sample 2 – 1.5 m depth	13.0	77.6	9.4	0.045	2.03×10^{-5}	41
Sand and Gravel						
BH10, Sample 7 – 7.6 m depth	6.3	58.9	34.8	0.175	3.06×10^{-4}	84

The natural water-bearing sand, silt and sand and gravel soils have a saturated hydraulic conductivity in the range of 10^{-4} to 10^{-7} m/s, based on the results presented above.

The infiltration rates have been calculated using correlation from TRCA/CVC Low Impact Development Stormwater Management Planning and Design Guide protocol which references Ministry of Municipal Affairs and Housing (MMAH) Supplementary Guidelines to the Ontario Building Code 1997, SG-6 Percolation Time and Soil Descriptions. 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.

Single Well Response Test (SWRT)

A Single Well Response Test (rising head test) was conducted in two of the water-bearing monitoring wells on June 2, 2023 to estimate the hydraulic conductivity of the sand and gravel and sandy silt/silt soils which are prevalent across the site. The SWRT provides an estimate of the hydraulic conductivity value of the geological formation within the immediate area around the well screens, which are generally set within water-bearing seams/layers.

Static groundwater measurements were taken prior to the start of the test. A submersible pressure transducer with a water level logger was inserted into the monitoring well to measure the change in water level for the duration of the test. Use of the data logger allows for high frequency data collection and increased accuracy, compared to manual measurements during the testing.

The Hydraulic conductivity values were estimated from field SWRT data as per the Hvorslev's method (refer to worksheets provided in Appendix D). A summary of the hydraulic conductivity values estimated from the field SWRT is provided in the table below.

Table 7 - Hydraulic Conductivity and Factored Infiltration Rates from Single Well Response Tests

Well ID	Well Depth, m bgs	Screen Length, m	Formation Screened	Estimated Hydraulic Conductivity, m/s	Factored Infiltration Rate mm/hr
BH4/MW	6.1	3.05	Sandy Silt	$6.03x10^{-7}$	16
BH7/MW	4.27	1.52	Sand and Gravel, and Silt	$4.26x10^{-8}$	8

The test data results yield hydraulic conductivity values which are in the range of 10-7 to 10-8 m/s for the water-bearing sandy soils.

Onsite Verification During Construction

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 completion of drilling. In general, boreholes located within the eastern/south-eastern limits site were found to be open and dry through the full depth of the borehole excavation. Short term water levels are summarized in the following table.

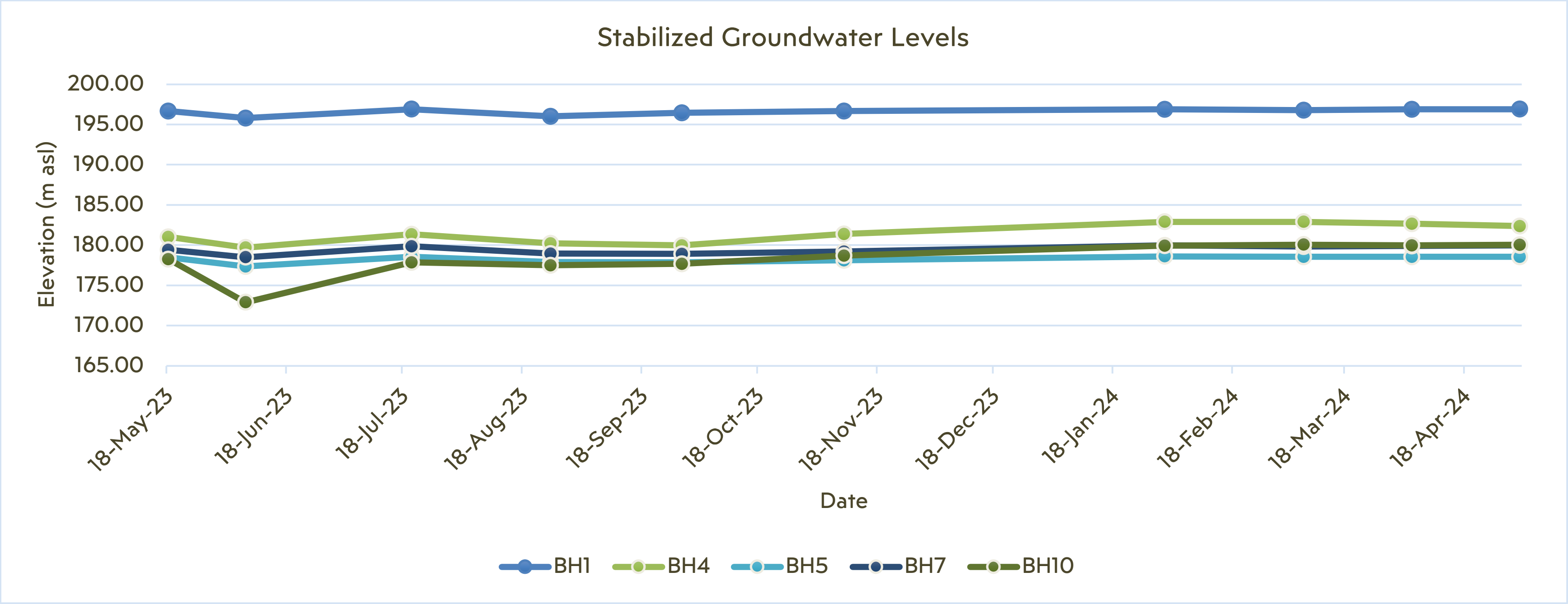
Table 8 - Short Term Groundwater Observations

Borehole	Ground Surface Elevation, m asl	Groundwater Observations, m bgs	Groundwater Elevation, m asl
BH3	182.51	2.74	179.77
BH6	180.10	1.83	178..27
BH8	185.13	2.74	182.39
BH9	181.73	3.96	177.77
BH11	179.61	4.27	225.54

Stabilized water level measurements were recorded in the monitoring wells installed across the site, as summarized in the following table. The monitoring wells have been left in place to allow for additional seasonal monitoring.

Table 9 - Stabilized Groundwater Observations

Monitoring Well	Ground Surface Elev. (m, asl)	Depth to Groundwater (m, bgs) *Groundwater Elevation (m, asl)*									
		May 18, 2023	Jun 7, 2023	Jul 20, 2023	Aug 25, 2023	Sept 28, 2023	Nov 9, 2023	Jan 31, 2024	Mar 7, 2024	April 4, 2025	May 2, 2024
BH1/MW	211.66	14.99 *196.67*	15.87 *195.79*	14.75 *196.91*	15.66 *196.00*	15.21 *196.45*	15.02 *196.64*	14.79 *196.87*	14.88 *196.78*	14.79 *196.87*	14.75 *196.91*
BH4/MW	183.85	2.84 *181.01*	4.14 *179.71*	2.51 *181.34*	3.64 *180.21*	3.88 *179.97*	2.46 *181.39*	0.95 *182.90*	0.95 *182.90*	1.20 *182.65*	1.47 *182.38*
BH5/MW	178.86	0.37 *183.48*	1.51 *182.34*	0.32 *178.54*	0.98 *177.86*	1.01 *177.85*	0.75 *178.11*	0.25 *178.61*	0.30 *178..56*	0.30 *179.56*	0.32 *178.54*
BH7/MW	180.17	0.74 *179.43*	1.67 *179.43*	0.33 *179.84*	1.22 *178.95*	1.25 *178.92*	0.95 *179.22*	0.22 *179.95*	0.34 *179.83*	0.28 *179.89*	0.20 *179.97*
BH10/MW	181.38	3.14 *178.24*	8.48 *172.90*	3.54 *177.84*	3.89 *177.49*	3.72 *177.66*	2.67 *178.71*	1.46 *179.92*	1.32 *180.06*	1.40 *179.98*	1.34 *180.04*



Shallow groundwater is present within the near-surface sandy soils, and/or intermittent sand layers at variable depths throughout the soil strata. 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 manual groundwater measurements recorded in the monitoring wells confirm a local groundwater flow direction in a north-easterly direction, towards Kettle Creek. This is demonstrated on the Groundwater Contour Plan provided on Drawing 7, in Appendix A.

3.1.4 Excess Soils Characterization

Discreet soil samples were collected from the boreholes to provide a preliminary assessment of the soil quality for the natural soils encountered onsite from an environmental standpoint. Three samples were collected from the boreholes. The following table summarizes the soil sample locations, depths, and analytical parameters included in the testing.

Table 10 - Environmental (Analytical) Samples

Sample ID	Depth	Laboratory Analytical Parameters
BH4, Sample 1	0.7 – 1.2 m	BTEX, PHCs, Metals, Inorganics, PAHs, EC, SAR, pH
BH5, Sample 1	0.7 – 1.2 m	BTEX, PHCs, Metals, Inorganics, PAHs, EC, SAR, pH
BH7, Sample 1	0.7 – 1.2 m	BTEX, PHCs, Metals, Inorganics, PAHs, EC, SAR, pH

Results are presented within Appendix E. However, the following table has been prepared which identifies sample parameters which were found to exceed the following site condition standards (SCS)as prescribed by O.Reg. 153/04:

- Table 1 SCS for residential / parkland / commercial / industrial / community property use for fine grained soils; and,
- Table 2 SCS for residential/parkland/institutional property use.

Table 11 - Chemical Exceedances

Sample ID	Table 1 SCS Exceedances	Table 2 SCS Exceedances
BH4, Sample 1	*No exceedances*	*No exceedances*
BH5, Sample 1	*No exceedances*	*No exceedances*
BH7, Sample 1	*No exceedances*	*No exceedances*

If excess soils are planned to be disposed of offsite, the fill should be sent to a receiver that can accept the material, based on the soil characterization work which has been done on the collected samples. Contractors carrying out the site works should also be aware of the Excess Soils Management Regulation (O.Reg. 406/19), which may require additional analytical sampling and testing, depending on estimated volumes of excess soils which may be generated at the site. This is discussed further in Section 4.1.2

4. GEOTECHNICAL COMMENTS AND DISCUSSION

It is understood that consideration is being given to re-develop the lands with a mixed-density residential plan of subdivision. It is understood that the development will be accessed via local roadways, and serviced with municipal sewers and water supply. A stormwater management facility is expected to be located in the easterly extents of the site. A Preliminary Draft Plan is provided on Drawing 1, appended.

The boreholes drilled at the site generally revealed a layer of surficial topsoil which is underlain by silt, sand, sand and gravel, and silt till soils. Shallow groundwater is present within the near-surface sandy soils, and/or intermittent sand layers at variable depths throughout the soil strata. Depending on the timing of construction, it is anticipated that seasonal conditions may cause variations in the stabilized groundwater level.

The following sections of this report provide geotechnical comments and recommendations to assist with design and construction of the proposed residential & recreational development, including:

- Site preparation, including guidance for cut and fill operations, the re-use of excavated materials as engineered fill / structural fill and guidance for engineered fill placement;
- Temporary excavations, including maximum slope inclinations to provide stable excavation side slopes in accordance with OHSA requirements, excavation support (shoring methods, if required), and lateral earth pressures;
- Groundwater Control, including the need for a Permit to Take Water (PTTW) or Environmental Activity Sector Registry (EASR) submission for construction dewatering, if required;
- Foundation design, including soil bearing capacity, subgrade preparation and allowable settlements;
- Concrete slab and basement construction (including lateral earth pressures and provisions for shallow groundwater conditions);
- Seismic design considerations based on borehole data and published information for soil conditions below the depth of exploration;
- Site servicing, including recommendations for pipe bedding and trench backfill;
- Pavement design recommendations for local roadways, construction access routes, and restoration of existing site pavements where servicing tie-ins may be expected to occur,
- Excess soil management discussion to assist contractors in understanding the characteristics of excess soils which may be generated from onsite excavations, and which may require disposal offsite.

4.1 Site Preparation

4.1.1 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 100 to 200 mm of surficial topsoil. Thicker topsoil areas may also be present between the borehole locations, in the fairways and built up tee-decks, 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.

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 the geotechnical inspector. 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 natural subgrade soils, comprised of silt, sand, sand and gravel, and silt till, 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 offsite transport and disposal. The testing requirements for disposal will depend on the requirements outlined by the receiver.

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 8, in Appendix A.

4.1.2 Excess Soils Management Considerations

In December of 2019, the Ministry of Environment, Conservation, and Parks (MECP) released a regulation under the Environmental Protection Act, titled *On-Site and Excess Soil Management* to support improved management of excess construction soil. The current version of Regulation 406/19 includes recent amendments (December 2024), and the regulation is now fully implemented.

Excess soil is defined as material that was generated during construction activities at a Site but will not be needed for grading, fill, or other purposes and therefore needs to be transported off-Site. The regulation requires a project leader to comply with specific requirements before removing 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.

The site is within a predominantly agricultural area; however, golf course operations are considered commercial land-use, and as such, management of excess soils will be required to adhere to the regulatory requirements. Preparation of the aforementioned planning documents are required. Coordination with a qualified person (QP) will be required to ensure that work is carried out in accordance with the regulatory requirements.

It is noted that under the Regulation, 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 property owner is expected to retain a Qualified Person (QP) to assist in the preparation of the aforementioned documents and in the soil characterization work (environmental testing on select soil samples), prior to any excess soils being removed from the Site. Stonecairn has staff that can provide this service, if required. Results of preliminary soil screening are provided in Section 3.1.4 of this report, and confirm that no impacted soils were identified in the geotechnical drilling program.

In the event that the site requires imported fill material to achieve design grades, the site would be characterized as a Beneficial Re-Use Site. As such, 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. 406/19 Table 2.1 Site Condition Standards, Residential/Parkland/Institutional Land Use (or better) is generally considered appropriate for this site. Within 30 m of Kettle Creek, imported fill should meet Table 1 Site Condition Standards.

4.2 Methane Abatement

No discernable methane concentrations were reported in the boreholes advanced at the site. The methane monitoring involved taking readings at completion of drilling using an RKI Eagle 2 total combustible gas meter, recently calibrated with hexane.

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.

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 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 compact silt and silt till encountered in each borehole are generally classified as Type 2 soil. For excavations which extend through or terminate in Type 2 soil, temporary excavation side slopes must be cut near vertical in the bottom 1.2 m, and sloped back at an inclination of 1H:1V above that level.

- The natural sand and gravel and sand soils are generally classified as Type 3 soil 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.
- The marl, if saturated may be expected to behave as a Type 4 soil. Excavations which extend through marl and organic materials should be cut back with a maximum inclination of 3H:1V from the base of the excavation, and may slough to flatter inclinations.

Where perched groundwater is present within the near-surface sandy soils, excavations may exhibit Type 4 Soil characteristics, with significant sloughing below the groundwater level. For excavations which extend through or terminate in the wet silt/sand, temporary excavation side slopes should be cut back at a maximum inclination of 3H:1V from the base of the excavation.

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. This is particularly important for excavations which extend through the Port Stanley Arena parking lot.

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 for service trench excavations, 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.

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

Table 12 – Soil Parameters for Excavation Support

Soil	φ	γ (kN/m^3)	K_a	K_o	K_p
Marl	25	17.5	0.40	0.55	2.50
Compact Silt/Silt Till	28	19.5	0.36	0.53	2.78
Compact Sand and Silty Sand	30	19.5	0.33	0.50	3.15
Compact Sand and Gravel	30	21.5	0.32	0.47	3.20
Compact Granular 'B' (OPSS 1010)	32	22.0	0.31	0.47	3.25

Notes: Φ denotes internal friction angle (degrees)
γ denotes soil bulk unit weight
K_a denotes active earth pressure coefficient (Rankine, dimensionless)
K_o denotes at-rest earth pressure coefficient (Rankine, dimensionless)
K_p denotes passive earth pressure coefficient (Rankine, dimensionless)

4.3.2 Groundwater Control

Groundwater is present within the near-surface sandy soils, and/or intermittent sand layers at variable depths throughout the soil strata. Based on the results of the investigation, shallow groundwater is expected to be encountered within typical servicing depths, from water-bearing soils located approximately 0.2 to 4.1 m below existing ground surface. The shallow groundwater is present on an intermittent basis, as it was observed that some of the boreholes were open and dry at completion of drilling, and the stabilized water level in borehole BH1 was found to be more than 14 m below existing ground surface.

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.

In the event that servicing excavations extend into areas where shallow groundwater is present, positive groundwater control methods may need to be utilized for construction dewatering. Soil permeability values in the natural subgrade soils are expected to be in the range of 10-4 to 10-7 m/s, based on laboratory testing (presented in Section 3.1.2). This information is provided to assist with determining appropriate construction dewatering methods. The use of sump pits and pumps and/or interceptor trenches to reroute groundwater seepage which can accumulate in open excavations is expected to be sufficient for groundwater control of typical excavations.

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.

Based on the most recent water levels recorded at the site (January 2024), shallow groundwater conditions were recorded in the boreholes, within the expected foundation and servicing depths based on current site grades. Although site grading and detailed design for site servicing is not yet available, the subdivision development should have regard for the seasonal high groundwater levels recorded at the site, to incorporate some groundwater separation, where possible.

At this time, the need for a Permit to Take Water has not been confirmed. If design grades require significant dewatering (daily water taking volumes in excess of 50,000 L/day), an Environmental Activity and Sector Registry (EASR) or Permit to Take Water (PTTW) will be required for construction dewatering. The EASR allows for daily pumping in the range of 50,000 to 400,000 L/day and can also be used for the management of stormwater run-off, if needed. Given the proximity of the natural features which border the site, having an EASR which assists in the management of stormwater runoff and containment can help mitigate risks associated with erosion and sediment control at the site. A Permit to Take Water is required for daily water taking volumes in excess of 400,000 L/day.

A Construction Dewatering and Discharge Plan is required for an EASR or PTTW. Stonecairn can assist with the preparation of the required documents. Preparation of the Construction Dewatering and Discharge Plan requires information from the contractor carrying out the excavation work, and the contractor responsible for providing groundwater control. The construction methodology, including details for the typical length and depth of service trenches, information about excavation support or cut-off systems (such as trench liner boxes) which may be utilized, and the method of groundwater control which will be utilized. This information is included, to inform the discussion which is provided in the Dewatering Plan, which is expected to include discussion on potential impacts to soil settlement, impact to existing groundwater users and surface water features, along with consideration for extreme weather events. The Plan will also identify the discharge location for pumped water, including sediment and erosion control measures which will be utilized where water is contained onsite in surface water features, or where filtering of discharge water is planned, for water being outletted to municipal infrastructure. Some preliminary dewatering calculations are provided in Section 5 of this report.

The existing wells at the site may be used for ongoing/future groundwater monitoring. Seasonal variations in the groundwater level are anticipated at the site. In this regard, consideration should be given to establishing a program of manual groundwater measurements or installation of dataloggers in select wells to provide a continuous record of seasonal groundwater levels that can be used to assist in the detailed design of the proposed residential development.

4.4 Building Design and Construction

4.4.1 Foundation Design

For design of 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) - 125 kPa (~2500 psf)
- Ultimate Limit States (ULS) - 145 kPa (~3000 psf)

The buried organic (marl) layer encountered within the near surface sand soils is not considered suitable to support the proposed site structures without the risk of settlement. If organic material is encountered within any building footprints, it must be removed entirely and excavations should be restored in accordance with the engineered fill recommendations identified in Section 4.1.1.

Otherwise deep foundation alternatives will need to be considered to reach suitable founding soils below the marl and organics.

It should be noted that loose to very loose soil conditions were encountered in the near-surface souls for the majority of the boreholes (with the exception of Boreholes BH1 and BH11) drilled across the site. A thorough proof-roll of the subgrade soils and subgrade improvement in the form of re-compaction or bridging with granular soils may be appropriate where loose soils are encountered. When site grading plans are available for review, additional recommendations may be appropriate to ensure suitable soil bearing capacities are available in areas with future buildings and structures. Site inspection during construction and review to confirm the condition of the subgrade soils at the proposed footing base level is recommended and should be undertaken by the geotechnical engineer at the time of excavation.

Higher bearing capacities (SLS up to 190 kPa) may be present in the compact and dense natural subgrade soils, subject to inspection by a geotechnical engineer during construction. If higher bearing is required, consideration may be given to deep foundation alternatives, to transfer building loads to lower competent subgrade soils. Additional boreholes would be required to provide geotechnical recommendations for deep foundation alternatives.

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 subgrade soils may be susceptible to disturbance by construction activities, especially during adverse weather conditions or when water seepage from excavation sidewalls is present. Considering the presence of shallow groundwater encountered at the site, and variable loose sandy soils at shallow depth, after the founding surfaces have been exposed, the soils should be thoroughly compacted to provide a uniform base, suitable to provide the bearing capacity noted above. Due to capillary rise which can occur in fine grained soils, the proof-roll should be carried out without vibration or excess disturbance to the subgrade soils. 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 sand soils 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 geotechnical engineer is recommended to confirm soil conditions and verify soil bearing capacity.

4.4.2 Concrete Slab Construction

Concrete floors for the new building 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. Structural fill placed below the concrete floor slab should be comprised of inorganic soils, placed and compacted in uniform lifts, to a minimum of 98 percent SPMDD.

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 Basement Construction

The single-family lots throughout the site may be constructed with full depth foundations with basements. The basement floors can be constructed using cast slab-on-grade techniques provided that the subgrade is stripped of unsuitable material. It is recommended that a minimum 200 mm (8 inch) thick compacted layer of 19 mm (¾ inch) clear stone be placed between the prepared subgrade and the floor slab to serve as a moisture barrier.

The portion of exterior basement walls below finished groundwater level should be damp-proofed and designed to resist a horizontal earth pressure ‘P’ at any depth ‘h’ below the surface as given by the following expression:

$$P = K(\gamma\, h+q)$$

where, P = lateral earth pressure in kPa acting at depth h;
γ = natural unit weight, a value of 20.0 kN/m^3 may be assumed;
h = depth of point of interest in m;
q = equivalent value of any surcharge on the ground surface in kPa.
K = earth pressure coefficient, assumed to be 0.4

The above expression assumes that the perimeter drainage system prevents build-up of any hydrostatic pressure behind the wall. Foundations should be provided with damp-proofing and foundation drainage tiles, in accordance with standard Ontario Building Code (OBC) requirements.

In general, the existing soils excavated from the building footprints (from above the stabilized water level) are generally expected to be suitable for re-use as foundation wall backfill. In the event that excavated materials contain topsoil, organics or otherwise unsuitable material, such materials should be stockpiled separately, and limited to re-use where settlements can be tolerated.

A review of the Site Grading Plans should be conducted to confirm that building foundations will be set above the stabilized groundwater level. If this can be confirmed, no special water-proofing measures are required. Foundations should be provided with damp-proofing and foundation drainage tiles, in accordance with standard Ontario Building Code (OBC) requirements. Perimeter drains should be wrapped with filter fabric, and set in stone to limit the movement of fines into the drain tiles.

4.4.4 Seismic Design Considerations

Subsoil and groundwater information at the Site have been examined in relation to Section 4.1.8.4 of the Ontario Building Code (OBC) 2024. The subsoils expected below the buildings will generally consist of compact silt, sand and silt till soils.

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2024 indicated that to determine the site classification, the average properties in the top 30 m are to be used. 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 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 classified as “D” as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2024. In the event that a higher Site Classification is being sought by the structural design engineer, additional deep 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.

4.4.5 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.

Concrete sampling and testing for foundations and concrete slabs (in accordance with CSA A23.1-04) is recommended.

4.5 Site Services

Subgrade soils beneath new services are generally expected to consist of silt, sand, and sand and gravel soils. Although no bearing problems are anticipated for flexible or rigid pipes founded on natural mineral soils, 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 with the geotechnical engineer.

It is also noted that marl was contacted up to 4 m depth in the Port Stanley Arena parking lot area. For services which extend through the parking lot, excavations will extend through these soils. Localized sub-excavation and/or base stabilization improvements should be anticipated if servicing is set into these organic soils. In addition, excavation support through these soils must be designed with consideration for the high insitu moisture content which is present in these soils.

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.

A well graded stone layer may be used in service trenches as bedding below the spring line of the pipe, if necessary, to provide stabilization to the excavation base in wet subgrade soils, where encountered. Geotextile may be considered for wrapping the pipe and to limit movement of fines from surrounding soils into the bedding material. Potential locations for use of stone bedding can be identified through site inspection during construction and will vary across the site due to seasonal conditions and variations in perched groundwater conditions.

If marl and organics are left in place below site servicing pipes, a configuration of stone or gravel sandwiched with geotextile and/or geogrid will be required to provide adequate support for the pipes. Consultation with the geotechnical consultant is required in this regard.

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 and silt till. 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. Stockpiling of material for prolonged periods of time should be avoided. This is particularly important if construction is carried out in wet, adverse weather. Backfilling operations during cold weather should avoid inclusions of frozen lumps of material, snow and ice.

The following table outlines the recommended levels of compaction within the trench backfill:

Table 13 - Trench Backfill Compaction Requirements

Scenario	Minimum Recommended Compaction Level	Soil Moisture Content
More than 1 m below underside of granular subbase, and in landscaped areas	95% SPMDD	Within +/- 5% of optimum moisture
Less than 1 m below underside of granular subbase	98% SPMDD	Within +/- 3% of optimum moisture

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.

Normal post-construction settlement of the compacted trench backfill should be anticipated, with the majority of such settlement taking place within about 6 months following the completion of trench backfilling operations. This settlement may be compensated for, where necessary, by placing additional granular material prior to asphalt paving. Alternatively, if the asphalt binder course is placed shortly following the completion of trench backfilling operations in these areas, any settlement that may be reflected by subsidence of the binder asphalt should be compensated for by placing an additional thickness of binder asphalt or by padding.

4.6 Pavement Design

The development will be accessed with an internal road network, accessing Carlow Road to the southeast. The exposed subgrade soils within the roadways are expected to be comprised of re-compacted soils comprised of silt, sand, and sand and gravel. The road subgrade should be thoroughly proof-rolled and reviewed by the geotechnical consultant. In the event that loose or soft areas are noted, additional work may be required to sub excavate and replace unstable soils with suitable compactable material. In general terms, subgrade soils supporting site pavements 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 14 – Pavement Design Recommendations

Pavement Component for Local Roads	Pavement Component Thicknesses		Compaction Requirements
	Local Roads & Restoration of Port Stanley Arena Parking Lot	Restoration at Carlow Road	
Asphaltic Concrete	40 mm HL 3 / 50 mm HL 8	50 mm HL3 / 60 mm HL8	97% Bulk Relative Density (BRD)
Granular A Base	150 mm	150 mm	100% SPMDD
Granular B Subbase	300 mm	400 mm	100% SPMDD

A thicker granular subbase (up to 450 mm) may be warranted for the local roads where site roads will be used for construction access when only a portion of the pavement structure is in place. The design thicknesses noted above are not intended to support heavy and concentrated construction traffic.

Where local roads connect to existing pavements, pavement component thicknesses should match existing. The recommendations above are provided for reference, and should be confirmed in the field by the geotechnical engineer. Subgrade levels and pavement components should be tapered to match / tie-into existing pavement structures to minimize differential settlements at the transition from existing to new pavement.

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. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas.

The use of subdrains will help to maintain the stability of silty subgrade soils (where encountered) at the site, by removing excess subsurface water. The subdrains should be comprised of 150 mm perforated pipe set in stone and wrapped in geotextile (Terrafix 270R or equivalent).

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. Field sampling and testing of concrete should be in accordance with OPSS 904.

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.

4.8 Erosion and Sediment Control

Sediment and erosion control measures will be required during construction, particularly around the perimeter of the site, to contain sediment and prevent discharge towards the neighbouring properties and surface water features. A multi-barrier approach is recommended. The design of the Sediment and Erosion Control Plan for the site will need to incorporate suitable erosion control practices and strategies which are suitable to site conditions, and have regard for contingency measures planned in the event that the integrity of the system is compromised.

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 15 - Sediment Control BMPs

Practice / Task	**During Site Grading**	**During Site Servicing**	**During Building & Pavement Constructions**	**Following Construction**
Measures to Protect Off-Site Sediment Release				
Establish controlled construction entrance/exit points, incorporating the use of mud-mats to help control the amount of loose soil being carried offsite from construction vehicles	✓	✓		
Prevent wind-blown dust.	✓	✓	✓	
Installing perimeter ESC measures such as silt fence and/or silt sock around temporary soil stockpiles, with dedicated points of access clearly marked onsite.	✓	✓		
Build-up boulevard areas to help limit sediment-laden stormwater run-off (from open or partially constructed areas) from discharging into catchbasins and stormwater infrastructure, and regular inspection and maintenance of silt bags/geotextile filters installed in catchbasins.			✓	✓
Measures to Protect Natural Features				
Monitoring of discharge water (for water quality – turbidity) from stormwater run-off and construction dewatering activities.	✓	✓	✓	
Delineate work areas to limit construction activities encroaching into the natural heritage features and setback areas, to prevent unnecessary vegetation removal.	✓	✓	✓	
Dedicated fuel storage and equipment fueling areas located away from natural or otherwise sensitive features. Contractors should have an emergency spills management plan.	✓	✓		
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.	✓	✓	✓	✓

Practice / Task	**During Site Grading**	**During Site Servicing**	**During Building & Pavement Constructions**	**Following Construction**
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.			✓	✓

To help maintain the cohesiveness of underlying soils and reduce runoff velocities, vegetation cover should be maintained in the undisturbed area which buffers natural or undisturbed parts of the site. Staging and scheduling of construction activities and restoration efforts are important in this regard.

Topsoil stripping should be conducted in a logical sequence in order to minimize the areas where soil is exposed. Topsoil removal should be organized and timed according to the schedule for grading and development works within the overall property.

An inspection and reporting schedule should be incorporated into the Sediment and Erosion Control Plan. Contractors working at the site will be required to adhere to the approved Plan. Regularly scheduled inspections of the sediment and erosion measures are recommended. Adjustments to the plan may be required to adapt to site conditions and seasonal conditions to ensure that the system and erosion control strategy remains effective through the various stages of construction.

Consultation with the municipality and Conservation Authority is recommended to confirm inspection, monitoring, and reporting requirements, required for approval.

4.9 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:

- Subgrade examination prior to engineered fill placement;
- 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;
- Concrete sampling and testing for curbs and sidewalks; and,
- Inspection and materials testing for base and surface asphalt.

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. SLOPE STABILITY

A geotechnical review of the condition of the slopes located within the southwest portion of the subject site has been carried out. This slope stability assessment has been conducted to support the proposed residential development located proximal to the top of the slope.

The Provincial Policy Statement (PPS, 2024) provides a framework for ensuring that future development is directed away from areas of natural hazards, to mitigate potential risks to public health and safety, and to minimize the risk of property damage. Under Section 3.1.1 which deals with natural hazards, development is generally directed to areas outside of hazardous lands adjacent to rivers and streams which are impacted by flooding hazards and /or erosion hazards, in accordance with guidance developed by the Province of Ontario. Development in the form of institutional uses, essential emergency services and production and storage of hazardous substances is strictly prohibited from being located within hazardous lands. However, the Policy allows for some flexibility for other forms of development, under Section 3.1.7, where the following can be demonstrated and achieved:

a) Development and site alteration is carried out in accordance with floodproofing standards, protection works standards, and access standards;

b) vehicles and people have a way of safely entering and exiting the area during times of flooding, erosion and other emergencies;

c) new hazards are not created and existing hazards are not aggravated; and,

d) no adverse environmental impacts will result.

The proposed development limits are located outside of the regulatory flood limits, and therefore, floodproofing requirements are not applicable to this development. Through the Slope Stability Analysis presented in this report, the Erosion Hazard Limit identified in this report addresses safe emergency access, as well as stable slope geometry to ensure that new hazards are not created and that existing slope stability is not aggravated. The EIS work completed by others, addresses how potential environmental impacts have been addressed and mitigated, as it relates to the proposed development.

Section 3.2 of the Central Elgin Official Plan outlines the requirements and policies associated with Natural Hazards, as it relates to defining slope and flooding hazards. The intent of the Natural Hazard policies related to slope stability are to determine appropriate development setbacks, as defined by the Erosion Hazard Limit. The Erosion Hazard limit is based on a 100-year planning horizon, and includes allowances for stable slope configurations, emergency access, and toe erosion which can occur along the toe of the slope. In defining the natural hazard, work is to be carried out by a qualified professional, with recognized experience, and using generally accepted methodologies. It is important to note the qualifications of the author of this report (and assessor of the slope stability analysis presented in this report) are presented in Section 1.2, and demonstrate that this requirement has been satisfied.

5.1 Site Reconnaissance

A site review was carried out on May 24th, 2023. A scoped topographic survey was conducted to facilitate preparation of several cross sections identified at critical locations. A Topographic plan and the cross sections are provided in Appendix E for reference. At the time of the site reconnaissance visit, the Kettle Creek valley slopes were observed to be well vegetated with a mixture of young and mature trees and shrubs. No water seepage or signs of significant overland erosion were observed in the face of the slopes; however, it is anticipated that minor localized seepage where surface water which has infiltrated through the weathered near-surface soils may daylight at the slopes under wet weather conditions.

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. Six locations were selected for review at locations which are representative of the localized critical slope conditions. A Slope Stability Rating Chart has been completed for each profile, and are included in Appendix E for reference.

The Slope Instability Ratings range from 15 to 35, indicating a variable low to moderate potential for instability. The upper slope ratings suggest that the following scope of work is appropriate to assess the slope's stability: borehole investigation, lab testing (as appropriate to characterize soils), surveying; and preparation of a detailed report. As part of the geotechnical field program, a series of boreholes were advanced throughout the site, including one in proximity to the subject slopes (BH1/MW), which is equipped with monitoring wells to assess the soil and groundwater conditions at the site.

Select site photographs are presented below for reference. Signs of previous landslides or instability were not observed during this review. The site observations are consistent with the slope rating charts, which identify a low to moderate risk of instability.

Photo 1
Base of slope conditions,
Near profile F-F

Facing West.

Photo 2

Base of slope conditions, Near profile C-C'

Facing South.

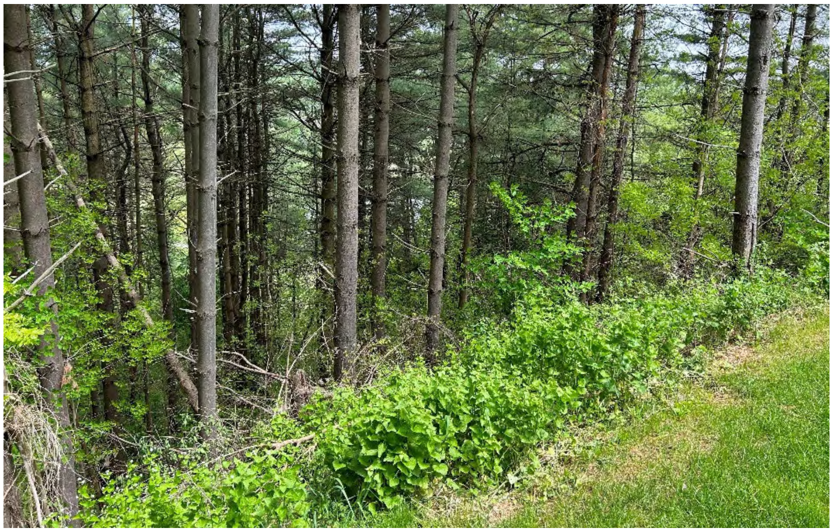


Photo 3

Top of slope conditions, near profile A-A'

Facing Northeast.

Photo 4

Top of slope conditions, near profile F-F'

Facing West.

5.2 Erosion Hazard Limit

5.2.1 Stable Slope Geometry

The slope stability analysis is based on the topographic information gathered at the site. A total of six cross sections were analysed. The location of these cross sections are shown on Drawing F1, and the cross sections are provided on Drawings F2 and F3, in Appendix F. Based on the cross sections provided, the following table summarizes the general slope conditions:

Table 16 – General Slope Conditions

Slope Section	Slope Height, m	Overall Inclination
Profile A-A	20.9	24° - 2.2H:1.0V
Profile B-B	13.1	33.3° - 1.5H:1.0V
Profile C-C	9.1	15.1° - 3.7H:1.0V
Profile D-D	2.0	21.0° - 2.6H:1.0V
Profile E-E	21.9	27.3° - 1.9H:1.0V
Profile F-F	14.0	34.0° - 1.5H:1.0V

Soil conditions within the boreholes located nearest to the slopes, generally revealed a layer of surficial topsoil which is underlain by sand, silt, and silt till. Stabilized groundwater was encountered within the sand layer in Borehole BH1/MW, approximately 15 m below ground surface. Soil strength parameters selected for the soil strata have been estimated based on the boreholes drilled near the slope, previously published information, our experience on similar projects and also by back-calculating from the existing steepest slopes. Static slope stability analyses were carried out for the soil stratigraphy using effective stress strength parameters as shown in table below:

Table 17 – Soil Parameters for Slope Stability Analysis

Predominant Soil Type	Unit Weight (kN/m^3)	Angle of Internal Friction	Cohesion (kPa)	Stable Slope Configuration
Compact Sand and Gravel	19.5	35°	0	2.4H : 1.0V
Compact to Dense Silt / Silt Till	20.0	28 °	5	2.3H : 1.0V

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. Slope profiles at Cross Sections A-A’ through F-F’ were analysed with the computer program GeoStudio Slope/W using the Simplified Bishop method. Minimum factors of safety are summarized below for the failure modes assessed.

Table 18 - Slope Stability Factors of Safety

Slope Section	Shallow Sliding Failure	Medium Depth Rotational Failure	Deep Rotational Failure
Profile A-A	1.69	**1.52**	1.56
Profile B-B	**1.10**	1.11	1.41
Profile C-C	2.78	**2.64**	2.68
Profile D-D	**1.47**	1.52	1.89
Profile E-E	**1.17**	1.36	1.64
Profile F-F	**1.26**	1.33	1.47

The calculated minimum FOS' for the slope failure surfaces are indicated in bold. Generally, the slope conditions for profiles A-A', C-C', and D-D' are stable, and profiles B-B', E-E' and F-F' exhibited results below an FoS of 1.4 for shallow and/or medium depth rotational failures. As a result, Profiles B-B', E-E' and F-F' have had a stable slope setback applied based on a stable slope configuration of 2.3H:1.0V.

5.2.3 Toe Erosion Allowance

Toe erosion allowance is applied to slopes which are in close proximity to watercourses, to allow for potential erosion or recession of the slope toe. None of the slopes assessed are located within 15m of an active watercourse, as such toe erosion setbacks have not been applied.

5.2.3 Emergency Access Allowance

The Ontario Government provides planning guidelines for development adjacent to slopes. The Provincial Policy Statement requires that an access allowance be included as part of the Erosion Hazard Limit. 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 flooding and/or slope failure.

In accordance with the PPS, 6 to 15 m setback is required in addition to the erosion and stability setbacks, which are discussed in the following sections. 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 each slope.

5.3 Development Setback Limit

The Erosion Hazard Limit defines the development setback limit, and is identified by combining the stable slope configuration, the toe erosion allowance and the emergency access allowance as described above. The following table summarizes the applicable setbacks for each assessed slope profile:

Table 19 – Calculated Development Setbacks

Slope Section	Toe Erosion Allowance, m	Stable Slope Allowance, m	Emergency Access Allowance, m	Erosion Hazard Limit, m (measured from existing top of slope)
Profile A-A	0.0	0.0	6.0	6.0
Profile B-B	0.0	9.8	6.0	15.8
Profile C-C	0.0	0.0	6.0	6.0
Profile D-D	0.0	0.0	6.0	6.0
Profile E-E	0.0	8.5	6.0	14.5
Profile F-F	0.0	8.9	6.0	14.9

From a geotechnical standpoint, the setback from the existing top of slope is considered to be the development setback. This development limit is indicated on Drawing F1, appended. However, it is important to note that additional setbacks may be required from an ecological standpoint.

5.4 Geotechnical Comments and Recommendations

The following geotechnical comments and recommendations are provided to help mitigate the potential occurrence of shallow sliding failures within the slope, and to help maintain the overall stable slope configuration.

- Care should be taken that materials and construction debris are not stockpiled adjacent to the top of the slope. Grades around the additions should be shaped to prevent surface water ponding at the top of the slope.

- 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.

- Where possible, uncontrolled surface water flows over the face of the slope should be minimized, to reduce the risk of surface erosion. In the event that future construction activities occur at the top of the slope or over the face of the slope (i.e. improvements or changes to the existing staircase), erosion control measures may be required during construction, to reduce the risk of surface water flows from washing out disturbed surfaces.

- Excavated soils should not be placed over the tableland near the crest of the slope. Any fill placement or changes to existing grades in proximity to the site slope may be subject to review and approval by the ABCA.

- Vegetation on the slope should be maintained. A program of plantation where appropriate, including deciduous trees and deep-rooted vegetation is recommended.

- Care must be taken when excavating for building footings and foundations in proximity to the top of the slope, to ensure that excavations are provided with adequate sidewall support. The design of any excavation support system should be prepared by a certified engineer, and should consider the loading associated with the sloped surface.

- 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 slope face.

- 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.

6. HYDROGEOLOGICAL DISCUSSION

6.1 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. Although the near surface weathered silt till and sandy subgrade soils encountered at the site are conducive for the presence of a shallow overburden aquifer, no free groundwater or groundwater accumulation within these soils was noted in the monitoring wells within depths of 4-8 metres below ground surface. As such, servicing excavations and excavations for building foundations are not expected to encounter shallow groundwater conditions.

As noted previously, during wet periods it is anticipated that surface water infiltration into the weathered or shallow sandy soils may occur at the site, which may cause a short-term / temporary presence of groundwater at shallow depths; however, this is not expected to be representative of a stabilized groundwater condition.

Deep Overburden Aquifers (30+ m depth)

In the western part of 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 4 in Appendix A, bedrock is estimated at more than 60 to 92 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.

6.2 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. Stonecairn 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.

6.2.1 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 located away from the wetland, 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.

6.2.2 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. Refer to additional discussion in Section 4.8.

6.3 Impact Assessment

6.3.1 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.

At a minimum, it is recommended that the contractor obtain an EASR to allow for dewatering efforts to pump in excess of 50,000 litres per day (and up to 400,000 litres per day). Preliminary dewatering estimates for water-taking volumes and zone of influence calculations are provided below; However, a more detailed analysis can be carried out when servicing depths and design grades are available. Under the EASR approval process, a Dewatering and Discharge Plan is required.

Assessment of Water Taking Volume

The water-bearing subgrade soils were generally comprised of sand, sandy silt/silt and sand and gravel. A saturated hydraulic conductivity with a geometric mean value of 3.27×10^{-5} cm/sec has been identified for these natural subgrade soils, based on correlations with grain size analyses and rising head single well response tests conducted at the site.

In establishing the preliminary estimate for the dewatering volumes which may be anticipated at the site, Stonecairn has relied upon the soil permeabilities determined from the single well response tests conducted at the site under three separate scenarios. It is assumed that the excavation limits are expected to be approximately 100 m in length, an average aquifer thickness of 8.8 m (based on the average elevation to the top of the silt till of 175.0 m) and a spring high water level in the range of 178.2 to 181.0 m across the site. Site servicing depths are generally expected to be in the range of 4 m maximum depth.

In this regard, total factored (FOS =3) dewatering volumes of 30,000 to 135,000 L/day are anticipated to control seepage from the excavation sidewalls. In addition, provision for an additional 50% is recommended to allow for variations in the soil conditions, and for handling stormwater run-off during/following typical (2-year) rain events, resulting in a net factored volume estimated in the range of 45,00 to 200,000 L/day.

Zone of Influence

To estimate the potential zone of influence for construction dewatering activities, a range of effective dewatering depths has been calculated, based on the Sischart and Kryieleis calculation method (Powers, Eq. 6.12), which uses the following equation:

$$R_o = 3000 (H-h_w) k^{1/2}$$

Based on the geometric mean for the water-bearing soils in the range of 3.27 x 10^{-5} m/sec, the following zone of influence distances have been determined:

- Effective dewatering depth of 3 m, unfactored zone of influence – 5.1 m
- Effective dewatering depth of 5 m, unfactored zone of influence – 8.6 m

These values are relatively low, based on the soil permeability of the natural subgrade soils which are noted above and observed in the boreholes.

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.

Some dewatering contractors have the capability to employ live-time monitoring of water quality, using Environmental Monitoring and Compliance (EMAC) monitoring equipment, which can be incorporated into the dewatering system, and accessed remotely to review flow, velocity and water quality parameters (including temperature, total suspended solids and turbidity, as well as other parameters).

5.3.2 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.

As noted in the MECP well records, several water supply wells In the general vicinity of the site set within the shallow overburden aquifer, however given the inferred groundwater flow direction (east/northeast), these wells are generally located upgradient of the site, and are therefore unlikely to be affected by construction dewatering activities.

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.

6.3.3 Well Decommissioning

Monitoring wells associated with the preparation of this report have been installed at the site, to document stabilized groundwater conditions. 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.

6.4 Low Impact Development Considerations

Consideration has been given to identify stormwater management options which allow secondary infiltration or reduced run-off under post-development conditions, to be incorporated into the stormwater management design. LID (Low Impact Development) strategies help to mitigate the impacts of increased runoff and stormwater pollution by managing runoff as close to its source as possible, by incorporating site features which enhance post-development infiltration, evapotranspiration, filtration and detention of stormwater. These practices can help to reduce contaminants in runoff, and can reduce the volume and intensity of stormwater flows.

The 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 permeability results presented in Section 3.1, the natural water-bearing subgrade soils have a saturated hydraulic conductivity in the range of 10^{-4} to 10^{-8} m/s, corresponding to factored infiltration rates in the range of 6 to 70 mm/hr. In addition, consideration must be given to the depth of shallow groundwater, and the potential drawdown effects of site servicing on perched groundwater levels.

It is also 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.

Where low permeability soils are present (such as the glacial till deposits), the use of infiltration-based features may not be effective. Alternative measures such as grassed swales, thickened topsoil, reduced lot grading and discharging water collected from roof leaders into landscaped areas are generally considered better suited to the soil conditions at the site. These alternative measures extend the retention time for surface water run-off, to help moderate and potentially reduce run-off volumes, and provide opportunities for evapotranspiration and limited infiltration.

The placement of fill soils throughout the site to raise grades, or to balance the cut-fill requirements across the site, may alter soil conditions and the effective depth to groundwater. Field confirmation of soil permeability and effective infiltration rates in the natural or reconstructed subgrade soils will need to be undertaken to confirm soil suitability for any infiltration-based LID measures which are considered at the site.

7. CLOSING

The geotechnical recommendations provided in this report are applicable to the project described in the text. Stonecairn 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 Stonecairn 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,

STONECAIRN CONSULTING INC.

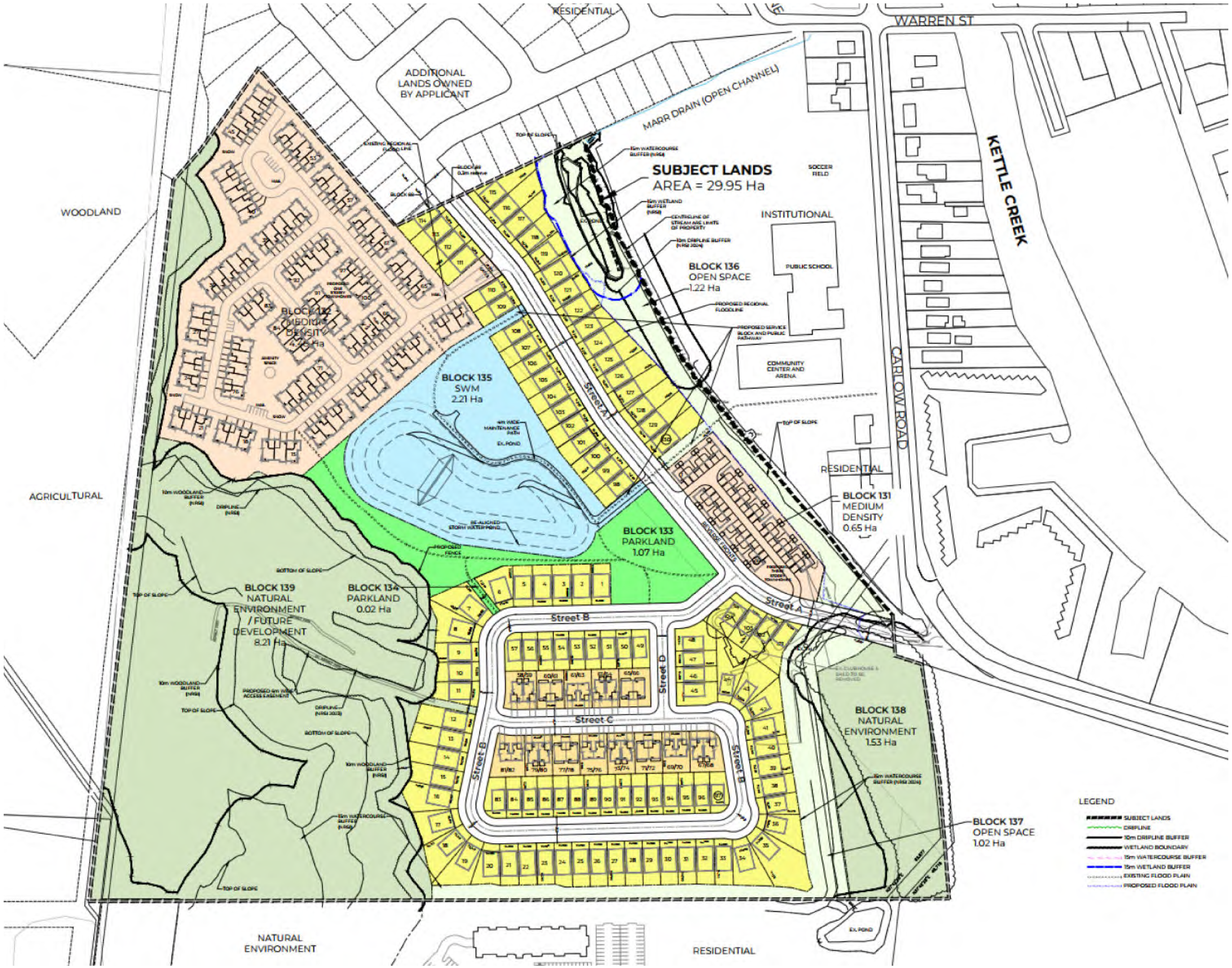
R. Walker

LICENSED PROFESSIONAL ENGINEER
Feb 12, 2025
R.A. WALKER
PROVINCE OF ONTARIO

Rebecca A. Walker, P. Eng., QP$_{ESA}$
President, Geotechnical Director
rebecca.walker@stonecairn.ca

APPENDIX A

DRAWINGS AND NOTES



SOURCE:

Produced from Draft Plan of Subdivision, prepared by Monteith Brown Planning Consultants, February 2025

STONECAIRN CONSULTING

PROJECT NAME

Proposed Residential & Recreational Development

PROJECT LOCATION

320 Carlow Road, Port Stanley, ON

DRAWING NAME

Draft Plan of Subdivision

SCALE	**PROJECT NO.**
As Shown	SC-02117
DATE	**DRAWING NO.**
February 2025	1

SOURCE

Google Earth Pro, Version 7.3.2.5776,
Coordinates 17T, 481565 m E, 4724409 m N,
Imagery date 7/2/2018

STONECAIRN CONSULTING

PROJECT NAME	Proposed Residential & Recreational Development
PROJECT LOCATION	320 Carlow Road, Port Stanley, ON
DRAWING NAME	Site Features

SCALE	**PROJECT NO.**
As Shown	SC-02117
DATE	**DRAWING NO.**
February 2025	2

LEGEND

🟥 KCCA Regulated Land

🟦 Flood Hazard Limits

— Approximate Site Boundary

SOURCE

Kettle Creek Conservation Authority Online Interactive Mapping, April 2023

STONECAIRN CONSULTING

PROJECT NAME

Proposed Residential & Recreational Development

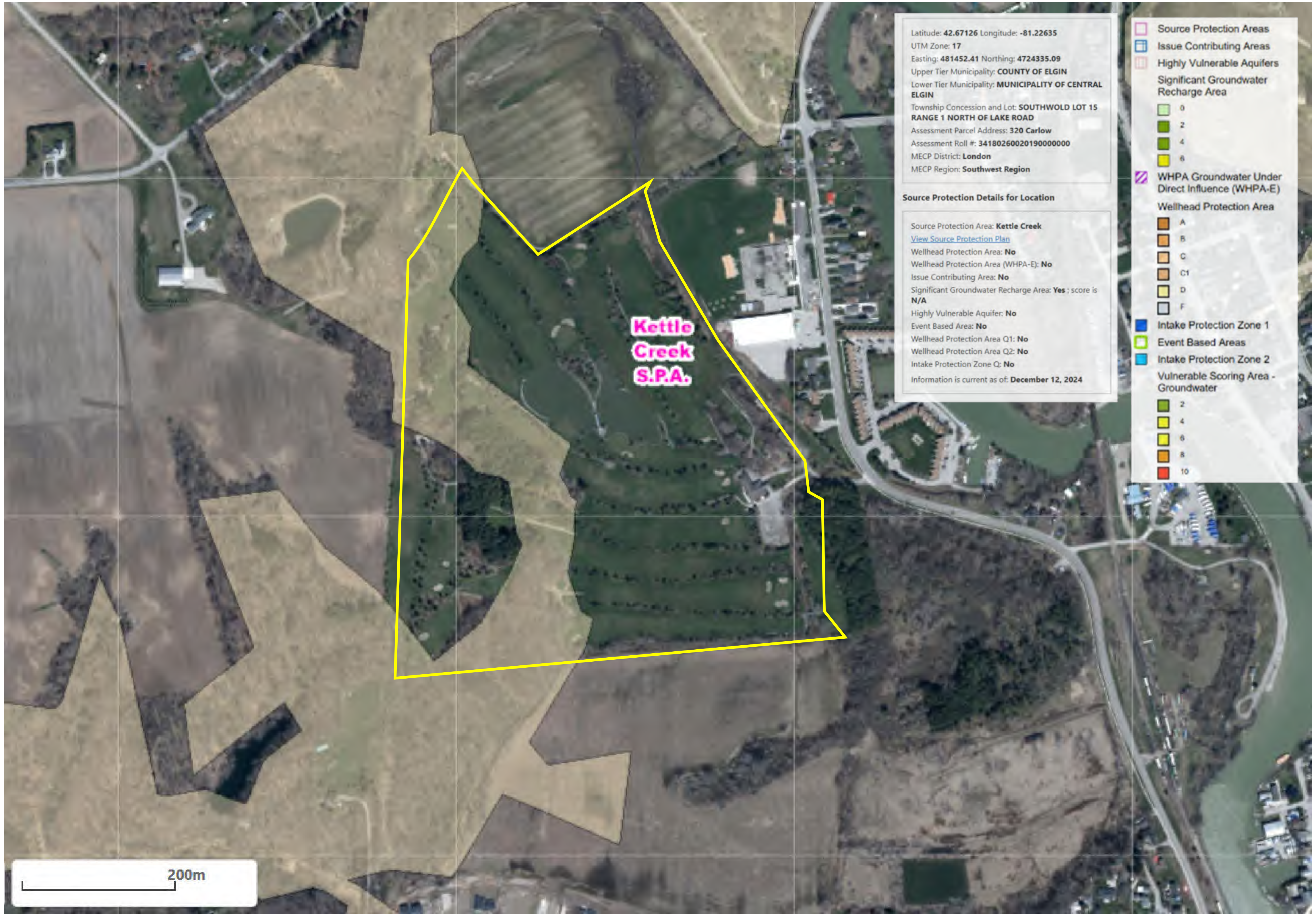
PROJECT LOCATION

320 Carlow Road, Port Stanley, ON

DRAWING NAME

KCCA Regulated Lands

SCALE As Shown	**PROJECT NO.** SC-02117
DATE February 2025	**DRAWING NO.** 3



Latitude: **42.67126** Longitude: **-81.22635**
UTM Zone: **17**
Easting: **481452.41** Northing: **4724335.09**
Upper Tier Municipality: **COUNTY OF ELGIN**
Lower Tier Municipality: **MUNICIPALITY OF CENTRAL ELGIN**
Township Concession and Lot: **SOUTHWOLD LOT 15 RANGE 1 NORTH OF LAKE ROAD**
Assessment Parcel Address: **320 Carlow**
Assessment Roll #: **3418026002019000000**
MECP District: **London**
MECP Region: **Southwest Region**

Source Protection Details for Location

Source Protection Area: **Kettle Creek**
View Source Protection Plan
Wellhead Protection Area: **No**
Wellhead Protection Area (WHPA-E): **No**
Issue Contributing Area: **No**
Significant Groundwater Recharge Area: **Yes** ; score is **N/A**
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: **December 12, 2024**

LEGEND

SOURCE

Ministry of Environment, Conservation and Parks – Source Water Protection Atlas, current to December 12, 2024.

STONECAIRN CONSULTING

PROJECT NAME

Proposed Residential & Recreational Development

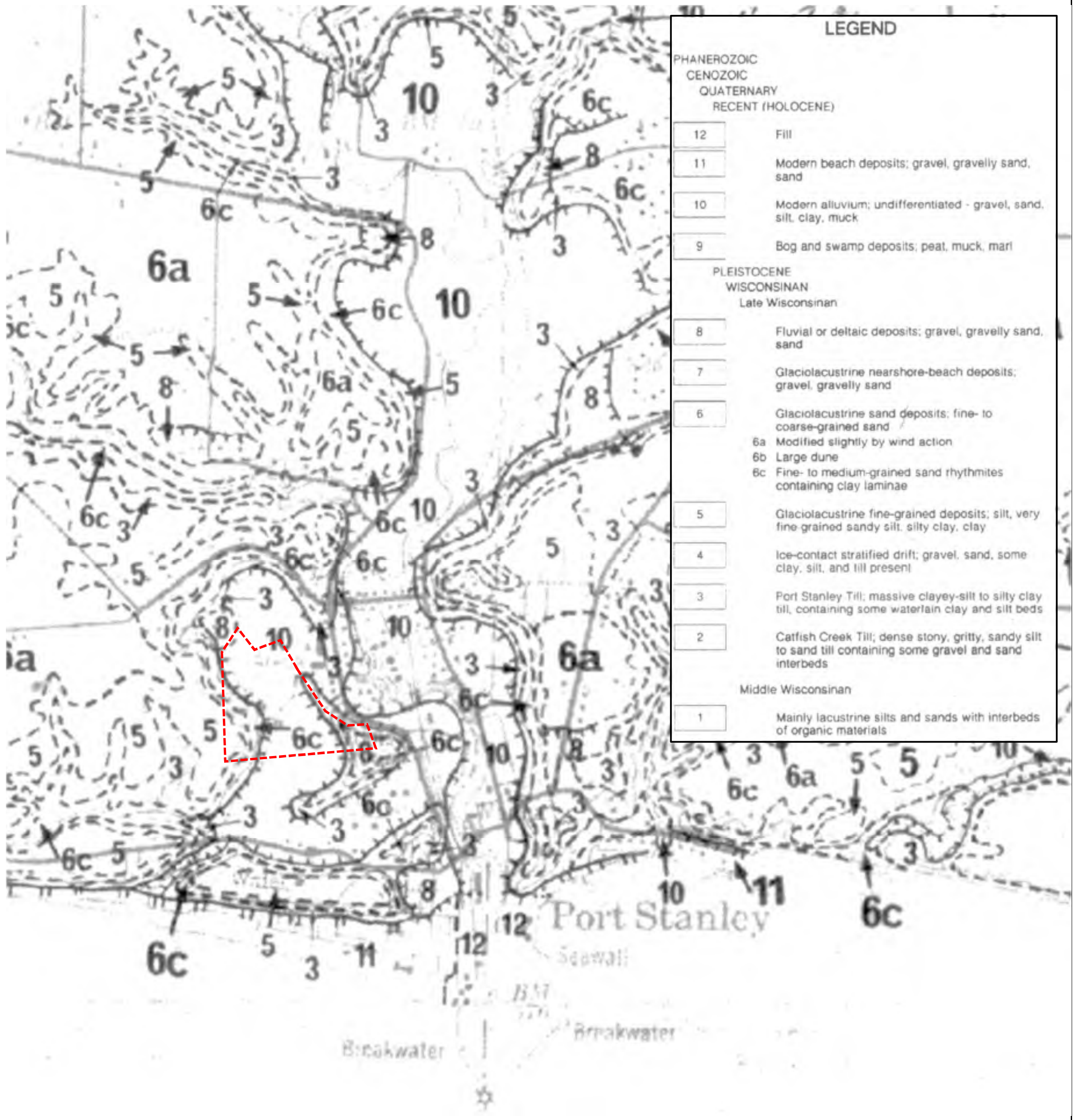
PROJECT LOCATION

320 Carlow Road, Port Stanley, ON

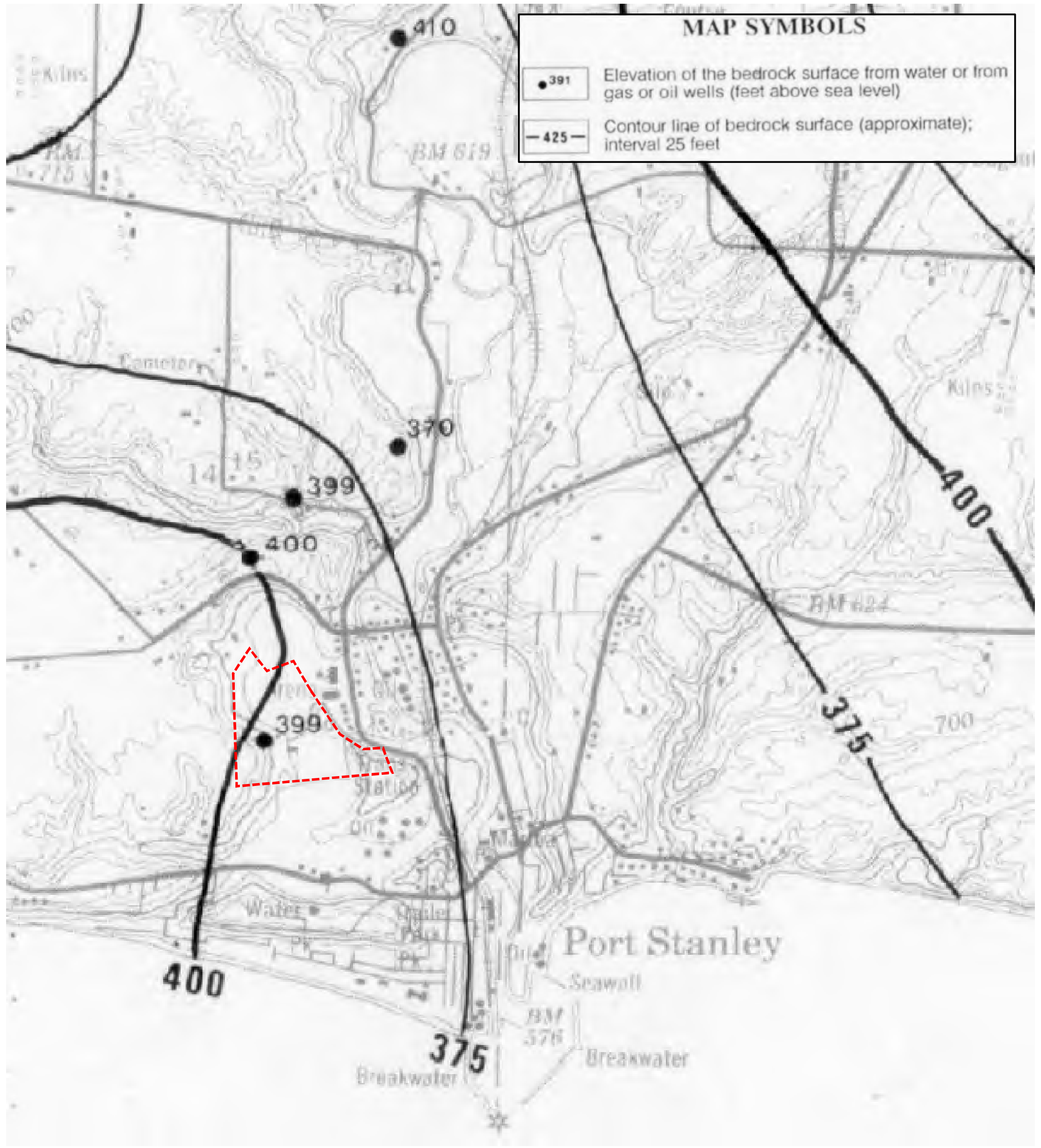
DRAWING NAME

Source Water Protection Mapping

SCALE	**PROJECT NO.**
As Shown	SC-02117
DATE	**DRAWING NO.**
February 2025	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

STONECAIRN CONSULTING

PROJECT NAME

Proposed Residential & Recreational Development

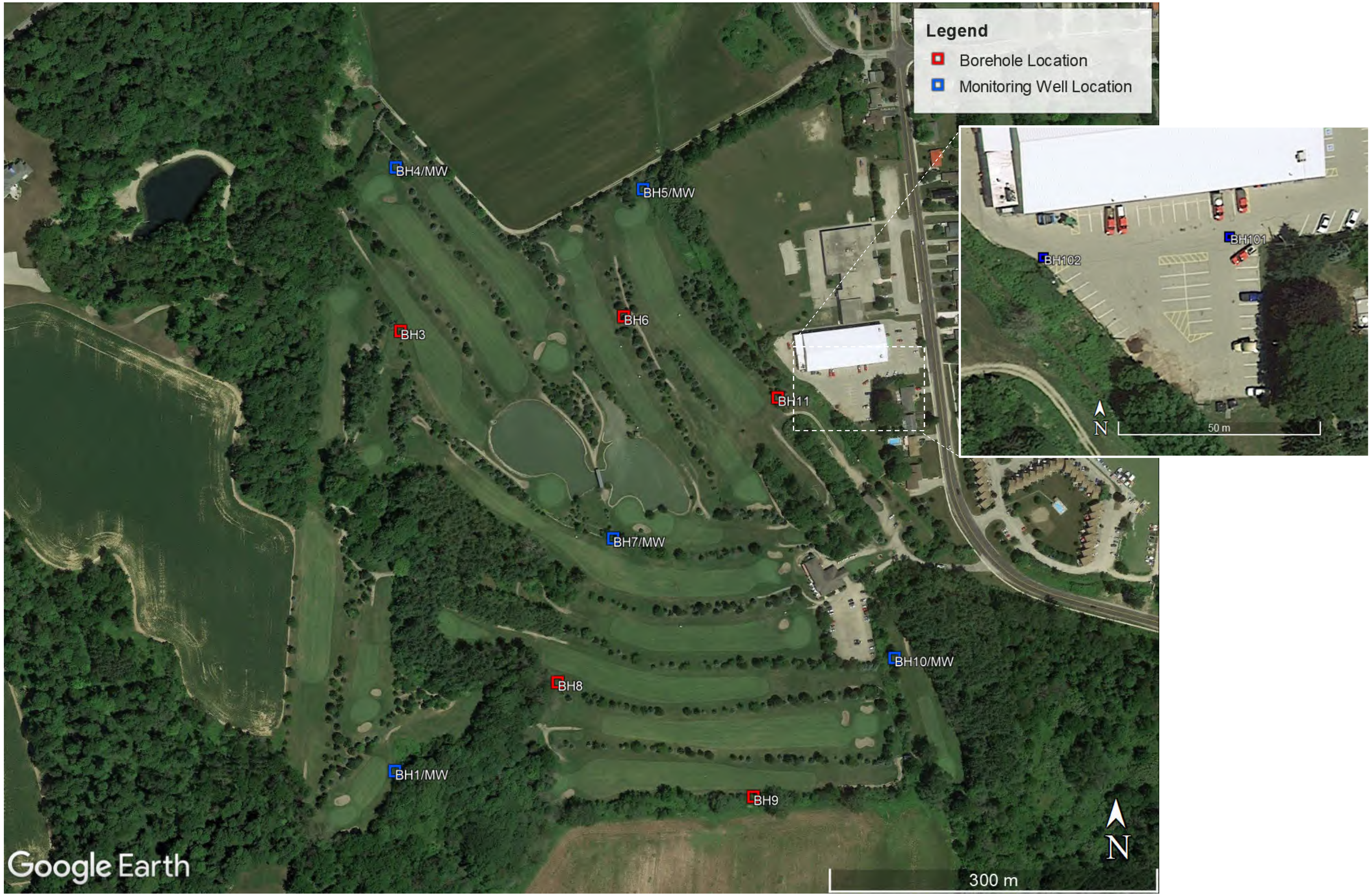
PROJECT LOCATION

320 Carlow Road, Port Stanley, ON

DRAWING NAME

Geological Mapping

SCALE	**PROJECT NO.**
1:50,000	SC-02117
DATE	**DRAWING NO.**
February 2025	5



Location	Northing, m N	Easting, m E	Ground Surface Elevation (m asl)
BH1/MW	4724097.56	481368.24	211.66
BH2 *(not drilled)*	-	-	-
BH3	4724494.56	481370.14	182.51
BH4/MW	4724644.82	481366.22	183.85
BH5/MW	4724625.11	481594.29	178.86
BH6	4724507.75	481576.17	180.10
BH7/MW	4724303.45	481565.77	180.17
BH8	4724171.94	481514.25	185.13
BH9	4724065.58	481694.46	181.73
BH10/MW	4724192.74	481825.00	181.38
BH11	4724432.59	481718.19	179.61
BH101	4724451.24	481797.59	178.41
BH102	4724446.92	481751.73	178.50

SOURCE:
Google Earth Pro, Version 7.3.2.5776, Coordinates 17T, 481565 m E, 4724409 m N, Imagery date 7/2/2018

NOTE:
BH Locations surveyed with Trimble R12 GPS Rover.

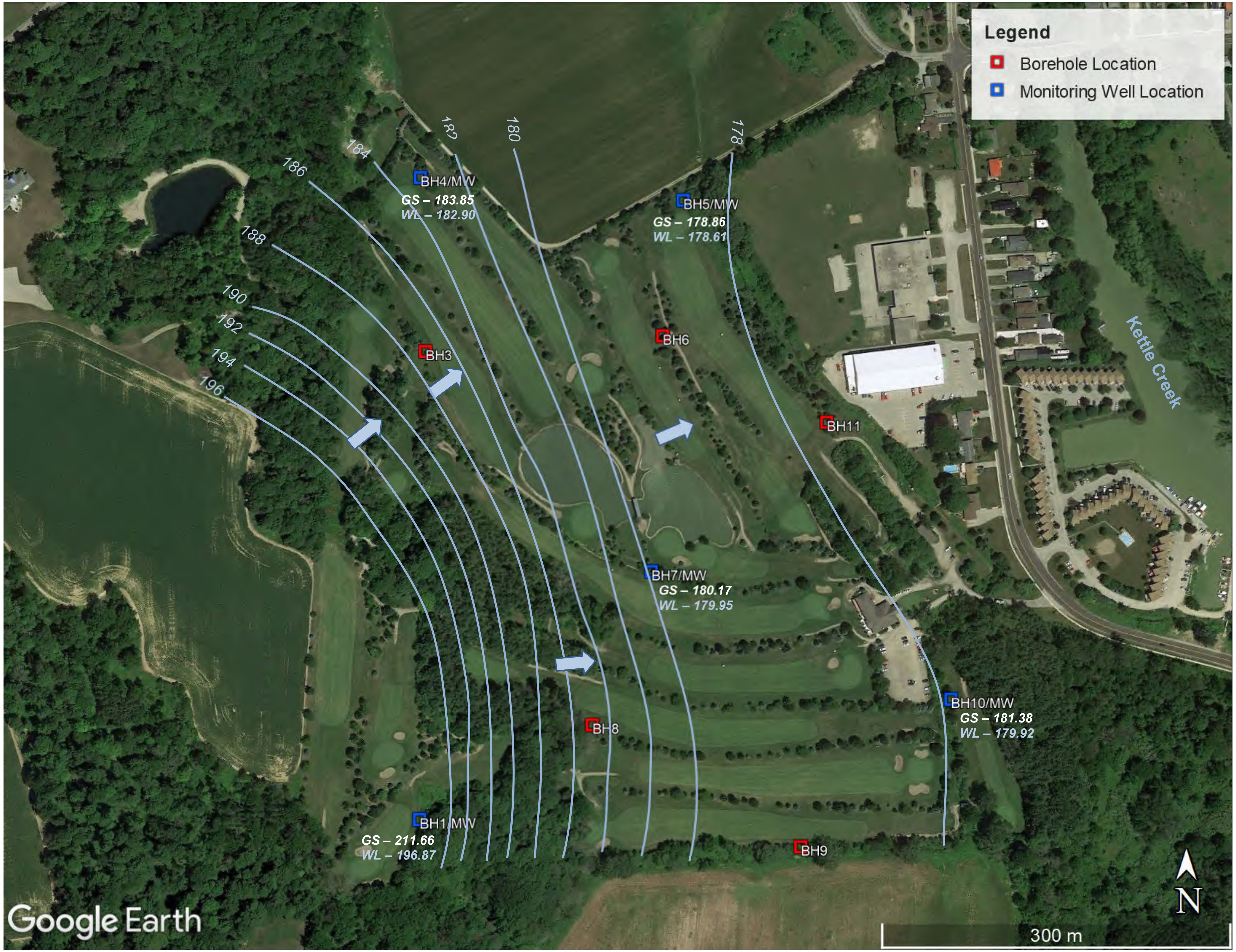
STONECAIRN CONSULTING

PROJECT NAME
Proposed Residential & Recreational Development

PROJECT LOCATION
320 Carlow Road, Port Stanley, ON

DRAWING NAME
Borehole Location Plan

SCALE	**PROJECT NO.**
As Shown	SC-02117
DATE	**DRAWING NO.**
February 2025	6



LEGEND

WL – 178.49	Ground Water Elevation, m asl (measured January 31, 2024)
╱	Groundwater Contour
➡	Groundwater Flow Direction

SOURCE:
Google Earth Pro, Version 7.3.2.5776,
Coordinates 17T, 481565 m E, 4724409 m N,
Imagery date 7/2/2018

STONECAIRN CONSULTING

PROJECT NAME

Proposed Residential & Recreational Development

PROJECT LOCATION

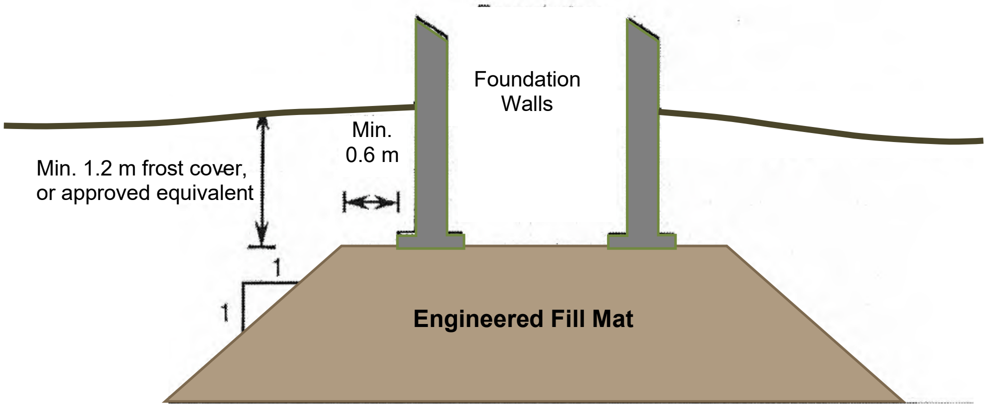
320 Carlow Road, Port Stanley, ON

DRAWING NAME

Groundwater Contour Plan
(January 2024)

SCALE	**PROJECT NO.**
As Shown	SC-02117
DATE	**DRAWING NO.**
February 2025	7

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.
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.

STONECAIRN CONSULTING	**PROJECT NAME** Proposed Residential & Recreational Development	**PROJECT NO.** SC-02117
	PROJECT LOCATION 320 Carlow Road, Port Stanley, ON	**DRAWING NO.** 8

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+

Project	**Proposed Residential & Recreational Development**	Borehole ID	
Project Location	**320 Carlow Road, Port Stanley**	**1/MW**	
Project Number	**GE-00920**	*Sheet 1 of 3*	

Date Drilled	**March 29, 2023**	Ground Surface Elevation	**211.66 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

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, silty loam, moist, 152 mm	
0.5						**SILTY SAND** - brown, fine grained, very moist	
1.0	☒	1	AS	-			*MC - 21.4%*
						- becoming dense below 1.4 m depth	
1.5							
	◩	2	80	33			*MC - 14.8%*
2.0							
2.5	☒	3	AS	-			*MC - 16.0%*
3.0							
	◩	4	80	43			*MC - 14.4%*
3.5							
4.0						- becoming compact with trace silt observed below 4.0 m depth	
4.5							
	◩	5	80	27			*MC - 39.1%*
5.0							
5.5							
6.0							
	◩	6	80	32			*MC - 16.5%*
6.5							
7.0							
7.5							
	◩	7	50	27			*MC - 14.3%*
8.0							

continued on the following page

Legend

- ◩ SPT Sample
- ☒ Bulk Sample
- ▥ Shelby Tube
- ▼ Stabilized Groundwater
- ▽ Inferred Groundwater

Well Construction Details

Pipe Diameter	50 mm CPVC Pipe
Installation Depth	18.29 m
Screen Length	3.05 m w/ No. 2 filter sand
Depth of Bentonite Seal	14.33 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

Water Levels:
May 18, 2023 - 14.99 m bgs
June 7, 2023 - 15.87 m bgs

Project	**Proposed Residential & Recreational Development**	Borehole ID
Project Location	**320 Carlow Road, Port Stanley**	**1/MW**
Project Number	**GE-00920**	*Sheet 2 of 3*

Date Drilled	**March 29, 2023**	Ground Surface Elevation	**211.66 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

Depth (m)	**Sample Type**	**Sample Number**	**Recovery (%)**	**SPT N-value (blows/0.3 m)**	**Graphic Log**	**Material Description**	**Remarks and Other Tests**
						continued from previous page	
8.5						- becoming stratified below 8.6 m depth	
9.0							
	◩	8	80	23			*MC - 5.2%*
9.5							
10.0							
10.5							
11.0						- becoming wet below 10.9 m depth	
11.5							
12.0							
	◩	9	70	26			*MC - 20.6%*
12.5							
13.0							
13.5							
14.0							
14.5					May 23' ▼		
15.0							
	◩	10	80	18			*MC - 23.9%*
15.5							
16.0							
						continued on the following page	

Legend

- ◩ SPT Sample
- ☒ Bulk Sample
- ▥ Shelby Tube
- ▼ Stabilized Groundwater
- ▽ Inferred Groundwater

Well Construction Details

Pipe Diameter	50 mm CPVC Pipe
Installation Depth	18.29 m
Screen Length	3.05 m w/ No. 2 filter sand
Depth of Bentonite Seal	14.33 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

Water Levels:
May 18, 2023 - 14.99 m bgs
June 7, 2023 - 15.87 m bgs

Project	**Proposed Residential & Recreational Development**	Borehole ID
Project Location	**320 Carlow Road, Port Stanley**	**1/MW**
Project Number	**GE-00920**	*Sheet 3 of 3*

Date Drilled	**March 29, 2023**	Ground Surface Elevation	**211.66 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

Depth (m)	**Sample Type**	**Sample Number**	**Recovery (%)**	**SPT N-value (blows/0.3 m)**	**Graphic Log**	**Material Description**	**Remarks and Other Tests**
16.5						*continued from previous page*	
17.0					*16.99 m*	**SILT** - grey, saturated, dense	
17.5							
18.0							
18.5	☒	11	AS	-	*18.75 m*	Gradation: 0% Gravel, 29% Sand, 71% Fines (Silt/Clay)	*MC - 13.6%*
19.0						BH Terminated at 18.75 m MW Installed at 18.29 m - refer to details below	
19.5							
20.0							
20.5							
21.0							
21.5							
22.0							
22.5							
23.0							
23.5							
24.0							

Legend

- ◩ SPT Sample
- ☒ Bulk Sample
- ▥ Shelby Tube
- ▼ Stabilized Groundwater
- ▽ Inferred Groundwater

Well Construction Details

Pipe Diameter	50 mm CPVC Pipe
Installation Depth	18.29 m
Screen Length	3.05 m w/ No. 2 filter sand
Depth of Bentonite Seal	14.33 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

Water Levels:
May 18, 2023 - 14.99 m bgs
June 7, 2023 - 15.87 m bgs

LDS

Project	**Proposed Residential & Recreational Development**	Borehole ID
Project Location	**320 Carlow Road, Port Stanley**	**3**
Project Number	**GE-00920**	*Sheet 1 of 1*

Date Drilled	**May 10, 2023**	Ground Surface Elevation	**182.51 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	**3.66 m bgs**
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

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, silty loam, moist, 178 mm	
0.5						**SILT** - brown, weathered, some sand, very moist, loose	
1.0	◩	1	60	5			*MC - 23.2%*
1.5	◩	2	60	WOH			*MC - 33.1%*
2.0						- becoming less weathered and sandy below 2.1 m depth	
2.5	◩	3	70	6			*MC - 21.8%*
						- becoming wet below 2.9 m depth	
3.0	◩	4	70	8			*MC - 20.5%*
3.5					▽		
4.0						- becoming grey below 4.0 m depth	
4.5	◩	5	70	6			*MC - 25.5%*
5.0							
5.5						- becoming compact below 5.6 m depth	
6.0	◩	6	70	13			*MC - 17.8%*
6.5					***6.55 m***		
						Borehole terminated at 6.55 m Borehole observed open to 3.96 m depth at time of completion Water measured at 3.66 m depth at time of completion	
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	

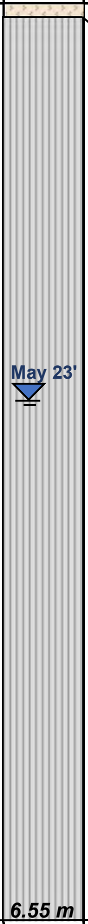
Additional Notes

MC - denotes moisture content

WOH - Weight of hammer

Project	**Proposed Residential & Recreational Development**	Borehole ID
Project Location	**320 Carlow Road, Port Stanley**	**4/MW**
Project Number	**GE-00920**	*Sheet 1 of 1*

Date Drilled	**May 10, 2023**	Ground Surface Elevation	**183.85 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

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, silty loam, moist, 178 mm	
0.5						**SILT** - grey, some sand, moist, very loose	
1.0	◪	1	70	3			*MC - 22.4%*
1.5	◪	2	60	3			*MC - 24.8%*
2.0						- becoming loose below 2.1 m depth	
2.5	◪	3	70	8			*MC - 23.6%*
3.0	◪	4	70	7		- becoming sandy below 2.9 m depth	*MC - 19.9%*
3.5							
4.0						- becoming saturated below 4.0 m depth	
4.5							
5.0	◪	5	80	6			*MC - 22.3%*
5.5						- becoming compact below 5.6 m depth	
6.0						Gradation: 2% Gravel, 33% Sand, 65% Fines (Silt/Clay)	
6.5	◪	6	70	12			*MC - 19.6%*
						Borehole terminated at 6.55 m MW Installed at 6.10 m - refer to details below	
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	6.10 m
Screen Length	3.05 m w/ No. 2 filter sand
Depth of Bentonite Seal	2.44 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

Water Levels:
May 18, 2023 - 2.84 m bgs
June 7, 2023 - 4.14 m bgs

Project	**Proposed Residential & Recreational Development**	Borehole ID
Project Location	**320 Carlow Road, Port Stanley**	**5/MW**
Project Number	**GE-00920**	*Sheet 1 of 1*

Date Drilled	**May 10, 2023**	Ground Surface Elevation	**178.86 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

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, silty loam, moist, 178 mm	
0.5					▼ May 23'	**SAND** - grey, medium to coarse grained, some silt, trace gravel, saturated, loose	
1.0	◩	1	60	5			*MC - 15.1%*
1.5						Gradation: 9% Gravel, 78% Sand, 13% Fines (Silt/Clay)	
	◩	2	70	6			*MC - 20.6%*
2.0						- becoming dense below 2.1 m depth	
2.5	◩	3	20	38			*MC - 18.3%*
					2.90 m		
3.0	◩	4	80	24		**SILT** - grey, trace sand, very moist, compact	*MC - 18.5%*
3.5							
4.0						- becoming dense below 4.0 m depth	
4.5							
	◩	5	70	36			*MC - 20.2%*
5.0							
5.5						- becoming compact below 5.6 m depth	
6.0							
6.5	◩	6	70	22	*6.55 m*		*MC - 18.6%*
7.0						Borehole terminated at 6.55 m MW Installed at 4.27 m - refer to details below	
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	4.27 m
Screen Length	1.52 m w/ No. 2 filter sand
Depth of Bentonite Seal	2.44 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

Water Levels:
May 18, 2023 - 0.37 m bgs
June 7, 2023 - 1.51 m bgs

Project	**Proposed Residential & Recreational Development**	Borehole ID
Project Location	**320 Carlow Road, Port Stanley**	**6**
Project Number	**GE-00920**	*Sheet 1 of 1*

Date Drilled	**May 10, 2023**	Ground Surface Elevation	**180.10 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	**1.83 m bgs**
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

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, silty loam, moist, 102 mm	
0.5						**SILT** - brown, trace topsoil inclusions, damp, loose	
1.0	◩	1	60	6			*MC - 26.9%*
1.5						- becoming grey with some organic inclusions (shells) encountered below 1.4 m depth	
	◩	2	70	6	▽		*MC - 33.8%*
2.0						- becoming very loose below 2.1 m depth	
2.5	◩	3	60	2	*2.76 m*		*MC - 20.0%*
3.0						**SILTY SAND** - grey, wet, loose	
	◩	4	40	9			*MC - 12.1%*
3.5							
4.0					*4.04 m*		
4.5						**SILT TILL** - grey, some sand, trace fine gravel, moist, dense	
	◩	5	60	40	*5.03 m*		*MC - 20.2%*
5.0						BH Terminated at 5.03 m Borehole observed open to 3.35 m at time of completion Water measured at 1.83 m depth at time of 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

LDS

Project	**Proposed Residential & Recreational Development**	Borehole ID
Project Location	**320 Carlow Road, Port Stanley**	**7/MW**
Project Number	**GE-00920**	*Sheet 1 of 1*

Date Drilled	**May 10, 2023**	Ground Surface Elevation	**180.17 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

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, silty loam, moist, 127 mm	
0.5					May 23' ▼	**SILTY SAND** - brown, fine to medium grained, moist, very loose	
1.0	◩	1	70	3	*1.37 m*		*MC - 24.5%*
1.5 2.0	◩	2	70	WOH	*2.13 m*	**MARL** - black, sandy texture, trace wood fragments, moist, very loose	*MC - 42.7%*
2.5	◩	3	70	7	*2.90 m*	**SAND AND GRAVEL** - grey, medium to coarse grained, wet, loose	*MC - 23.3%*
3.0 3.5	◩	4	70	6		**SILT** - grey, moist, loose	*MC - 18.0%*
4.0						- becoming compact below 4.0 m depth	
4.5 5.0	◩	5	70	12	*5.03 m*		*MC - 17.7%*
5.5						Borehole terminated at 5.03 m MW Installed at 4.27 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	4.27 m
Screen Length	1.52 m w/ No. 2 filter sand
Depth of Bentonite Seal	2.44 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

Water Levels:
May 18, 2023 - 0.74 m bgs
June 7, 2023 - 1.67 m bgs

Project	**Proposed Residential & Recreational Development**	Borehole ID
Project Location	**320 Carlow Road, Port Stanley**	**8**
Project Number	**GE-00920**	*Sheet 1 of 1*

Date Drilled	**March 27, 2023**	Ground Surface Elevation	**185.13 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	**2.74 m bgs**
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

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, silty loam, moist, 178 mm	
0.5						**SILTY SAND** - brown, fine to medium grained, trace gravel, moist, loose	
1.0	◣	1	80	5			*MC - 20.2%*
						- becoming damp and compact below 1.4 m depth	
1.5							
	◣	2	80	24			*MC - 14.9%*
2.0						- becoming very moist and dense below 2.1 m depth	
2.5	◣	3	80	33	▽		*MC - 20.1%*
3.0							
	◣	4	80	34			*MC - 18.9%*
3.5							
4.0						- becoming grey below 4.0 m depth	
4.5							
	◣	5	80	35	*5.03 m*		*MC - 19.5%*
5.0						BH Terminated at 5.03 m Borehole observed open to 3.96 m at time of completion Water measured at 2.74 m depth at time of 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

LDS	Project	**Proposed Residential & Recreational Development**	Borehole ID **9** *Sheet 1 of 1*
	Project Location	**320 Carlow Road, Port Stanley**	
	Project Number	**GE-00920**	

Date Drilled	**March 27, 2023**	Ground Surface Elevation	**181.73 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	**3.96 m bgs**
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

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, silty loam, moist, 203 mm	
0.5							
						SILTY SAND - brown, fine to medium grained, trace gravel, very moist, saturated	
1.0	◩	1	80	2			*MC - 31.1%*
1.5							
	◩	2	80	2		- becoming grey below 1.9 m depth	*MC - 29.1%*
2.0							
2.5	◩	3	80	1			*MC - 24.1%*
3.0							
	◩	4	80	WOH		- sandy marl inclusions and wood fragments encountered at 3.0 m depth	MC - 35.4%
3.5							
4.0					▽ *4.04 m*		
						SAND AND GRAVEL - grey, medium to coarse grained, saturated, loose	
4.5							
	◩	5	60	8			*MC - 9.2%*
5.0							
5.5						- becoming compact below 5.6 m depth	
6.0							
	◩	6	70	11			*MC - 32.6%*
6.5							
7.0							
7.5							
	◩	7	80	14	*8.08 m*	- becoming grey silt till below 8.1 m depth	*MC - 17.7%*
8.0							
						BH Terminated at 8.08 m Borehole observed open to 4.27 m at time of completion Water measured at 3.96 m depth at time of completion	

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

WOH - Weight of Hammer

Project	**Proposed Residential & Recreational Development**
Project Location	**320 Carlow Road, Port Stanley**
Project Number	**GE-00920**

Borehole ID

10/MW

Sheet 1 of 2

Date Drilled	**March 27, 2023**	Ground Surface Elevation	**181.38 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

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, silty loam, moist, 203 mm	
0.5						**SILTY SAND** - brown, fine grained, topsoil inclusions, trace wood fragments, damp, loose	
1.0	◩	1	80	7			MC - 28.2%
						- becoming moist below 1.4 m depth	
1.5							
	◩	2	80	5		- 200 mm silt seam encountered at 1.8 m depth	MC - 30.5%
2.0						- becoming very loose below 2.1 m depth	
2.5	◩	3	70	1		- becoming grey and very moist below 2.9 m depth	MC - 31.4%
3.0					May 23' ▼	- becoming saturated below 3.0 m depth	
	◩	4	80	WOH			MC - 25.3%
3.5							
4.0					4.04 m		
						MARL - black, sandy texture, trace wood fragments, saturated, very loose	
4.5					4.75 m		
	◩	5	30	WOH		**SILTY SAND** - grey, fine to medium grained, saturated, very loose	MC - 47.1%
5.0							
5.5							
6.0							
	◩	6	70	WOH			MC - 19.1%
6.5							
7.0					7.31 m		
7.5						**SAND AND GRAVEL** - grey, medium to coarse grained, saturated, compact	
	◩	7	50	10		Gradation: 35% Gravel, 59% Sand, 6% Fines (Silt/Clay)	MC - 11.8%
8.0							

continued on the following page

Legend

◩	SPT Sample
☒	Bulk Sample
▥	Shelby Tube
▼	Stabilized Groundwater
▽	Inferred Groundwater

Well Construction Details

Pipe Diameter	50 mm CPVC Pipe
Installation Depth	9.14 m
Screen Length	1.52 m w/ No. 2 filter sand
Depth of Bentonite Seal	7.01 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

WOH - Weight of Hammer

Water Levels:

May 18, 2023 - 3.14 m bgs

June 7, 2023 - 8.48 m bgs

LDS	Project	**Proposed Residential & Recreational Development**	Borehole ID
	Project Location	**320 Carlow Road, Port Stanley**	**10/MW**
	Project Number	**GE-00920**	*Sheet 2 of 2*

Date Drilled	**March 27, 2023**	Ground Surface Elevation	**181.38 m asl**
Drill Rig	**Geoprobe**	Groundwater Level at Completion	
Drilling Method	**Hollow Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**S. Hadden, P.Eng.**

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
					▨	*continued from previous page*	
8.5					*8.61 m*		
9.0					▥	**SILT TILL** - grey, trace fine gravel, moist, dense	
9.5	◪	8	80	37	*9.60 m*		*MC - 19.7%*
						BH Terminated at 9.60 m MW Installed at 9.14 m - refer to details below	
10.0							
10.5							
11.0							
11.5							
12.0							
12.5							
13.0							
13.5							
14.0							
14.5							
15.0							
15.5							
17.0							

Legend

- ◪ SPT Sample
- ☒ Bulk Sample
- ▥ Shelby Tube
- ▼ Stabilized Groundwater
- ▽ Inferred Groundwater

Well Construction Details

Pipe Diameter	50 mm CPVC Pipe
Installation Depth	9.14 m
Screen Length	1.52 m w/ No. 2 filter sand
Depth of Bentonite Seal	7.01 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

WOH - Weight of Hammer

Water Levels:
May 18, 2023 - 3.14 m bgs
June 7, 2023 - 8.48 m bgs

LDS

Project	**Proposed Residential & Recreational Development**	Borehole ID	**11**
Project Location	**320 Carlow Road, Port Stanley**		
Project Number	**GE-00920**		*Sheet 1 of 1*

Date Drilled	**March 27, 2023**	Ground Surface Elevation	**179.61 m asl**
Drill Rig	**Geoprobe**	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, P.Eng.**

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
						<u>TOPSOIL</u> - dark brown, silty loam, moist, 178 mm	
0.5						**<u>SILTY SAND</u>** - brown, fine to medium grained, moist, compact	
1.0	◩	1	70	10			*MC - 19.4%*
1.5							
	◩	2	70	23			*MC - 19.8%*
2.0						- becoming grey and very moist below 2.1 m depth	
2.5	◩	3	70	26			*MC - 19.3%*
					2.90 m		
3.0						**<u>SILT TILL</u>** - grey, trace fine gravel, damp, compact	
	◩	4	80	16			*MC - 14.4%*
3.5							
4.0						- becoming moist below 4.0 m depth	
					▽		
4.5							
	◩	5	80	20	***5.03 m***		*MC - 17.6%*
5.0						BH Terminated at 5.03 m Borehole observed open to 4.27 m at time of completion Water measured at 4.27 m depth at time of completion	
5.5							
6.0							
6.5							
7.0							
7.5							
8.0							

<u>Legend</u>

- ◩ SPT Sample
- ☒ Bulk Sample
- ▥ Shelby Tube
- ▼ Stabilized Groundwater
- ▽ Inferred Groundwater

<u>Well Construction Details</u>

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

<u>Additional Notes</u>

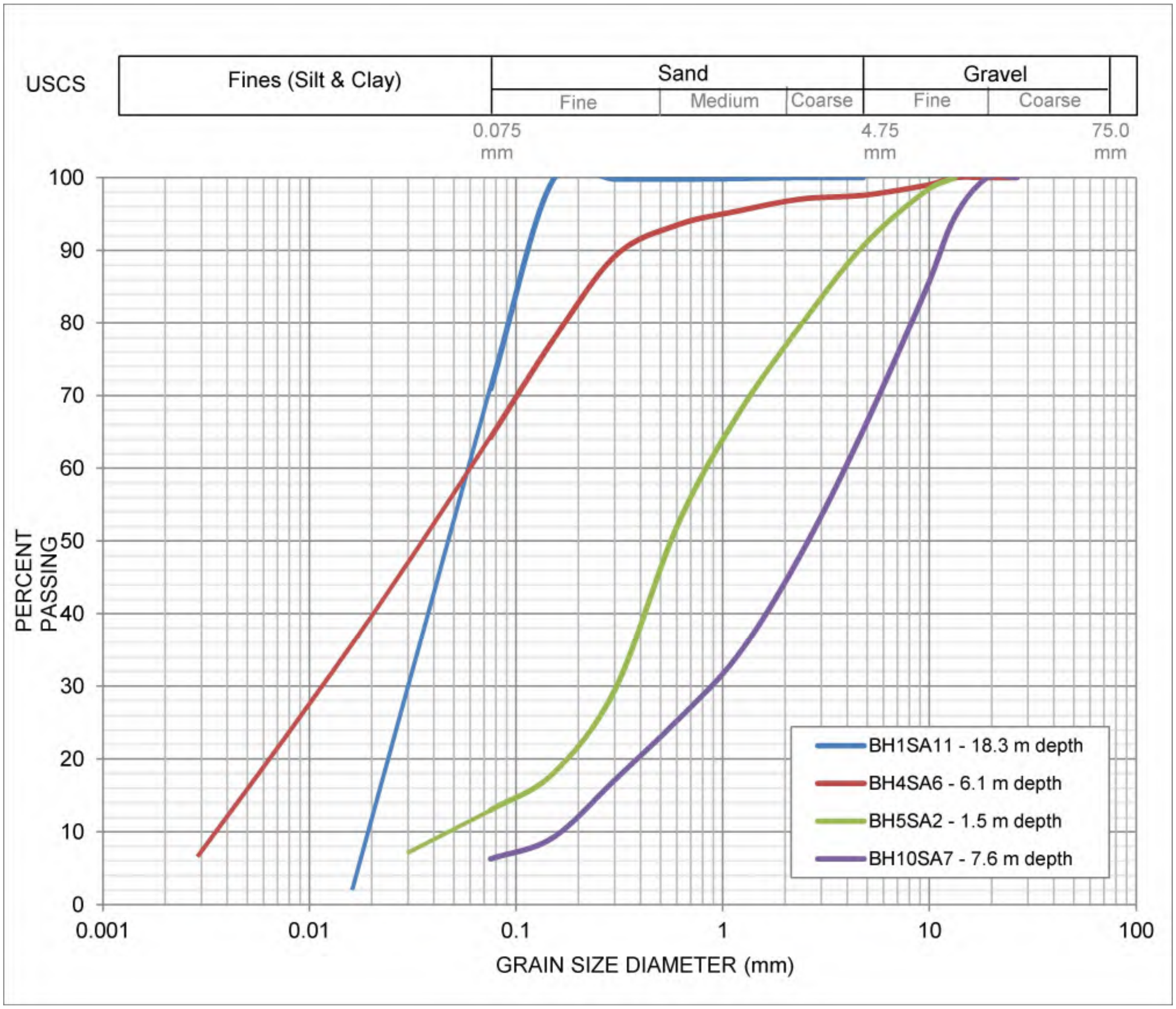
MC - denotes moisture content

Particle Size Distribution
Results of Sieve Analysis

Project Name: Proposed Residential & Recreational Development **Date:** 4-Jun-23

Project Location: 320 Carlow Road, Port Stanley **Project No.:** GE-00920

Sample ID	**Unified Soil Classification**				**Moisture Content (%)**
	Fines (Silt & Clay)	**% Sand**	**% Gravel**	**% Cobbles**	
BH1SA11 - 18.3 m depth	70.9%	29.1%	0.0%	0.0%	13.6
BH4SA6 - 6.1 m depth	64.3%	33.3%	2.4%	0.0%	19.6
BH5SA2 - 1.5 m depth	13.0%	77.6%	9.4%	0.0%	20.6
BH10SA7 - 7.6 m depth	6.3%	58.9%	34.8%	0.0%	11.8



STONECAIRN CONSULTING
85 Midpark Road – Unit 3, London, Ontario N6N 1B2

Servicing through Port Stanley Arena Parking Lot

332 Carlow Road, Port Stanley
SC-02117

Borehole ID
BH101
Sheet 1 of 1

Date Drilled	**23/01/2025**	Ground Surface Elevation	**178.41 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**1.83m bgs**
Drilling Method	**Solid Stem**	Site Supervisor	**J. May, EIT**
Drilling Contractor	**LST**	Checked By	**R. Walker, P.Eng.**

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
					■	ASPHALT - 51mm GRANULAR BASE - 483mm	
0.5					▨	FILL - mottled brown and grey silt, trace gravel, moist, loose	
1.0	◪	1	40	9	▨		MC - 27.4%
1.5	◪	2	60	5	▨	- becoming very moist with topsoil inclusions below 1.4 m depth ▽	MC - 30.6%
2.0					2.1 m	MARL- Brown, silty texture with mixed topsoil and organics, wet, loose	
2.5	◪	3	80	3	▨		MC - 126.1%
3.0	◪	4	40	16	▨	- becoming compact below compact 2.9 m depth	MC - 44.7%
3.5					▨		
4.0					4.04 m	SAND AND GRAVEL - grey, trace silt, fine to medium grained, wet, compact	
4.5	◪	5	50	19	5.03 m		MC - 11.4%
5.0						BH Terminated at 5.03m At completion, borehole caved to 3.65 m Water level measured at 1.83 m	
5.5							
6.0							
6.5							
7.0							

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

STONECAIRN CONSULTING

85 Midpark Road – Unit 3, London, Ontario N6N 1B2

Servicing through Port Stanley Arena Parking Lot	**332 Carlow Road, Port Stanley** **SC-02117**	Borehole ID **BH102** *Sheet 1 of 1*

Date Drilled	**23/01/2025**	Ground Surface Elevation	**178.50 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**2.26m bgs**
Drilling Method	**Solid Stem**	Site Supervisor	**J. May, EIT**
Drilling Contractor	**LST**	Checked By	**R. Walker, P.Eng.**

Depth (m)	**Sample Type**	**Sample Number**	**Recovery (%)**	**SPT N-value (blows/0.3 m)**	**Graphic Log**	**Material Description**	**Remarks and Other Tests**
					■ ▨	ASPHALT - 51mm GRANULAR BASE - 305mm	
0.5						SILT - mottled brown and grey, trace gravel, moist, compact	
1.0	◩	1	50	10			MC - 34.8%
					1.40 m		
1.5						SILTY MARL - brown, moist, wet, loose	
	◩	2	60	3			MC - 80.4%
2.0						▽	
						- root inclusions below 2.4 m depth	
2.5	◩	3	10	3			MC - 72.4%
						- grey, silt texture, trace organic inclusions, below 2.6 m depth	
3.0							
	◩	4	60	5	3.40 m		MC - 40.3%
3.5						SAND AND GRAVEL - grey, fine to medium grained, wet, compact	
4.0							
4.5					4.62 m		
	◩	5	80	15	5.03 m	SILT TILL - grey, moist, compact	MC - 17.9%
5.0						BH Terminated at 5.03m At completion, borehole caved to 3.96 m Water level measured at 2.26 m	
5.5							
6.0							
6.5							
7.0							

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

APPENDIX C

MECP WELL RECORD SUMMARY

MECP WATER SUPPLY WELLS

Well	Registration Year	Well Use	Depth of Well (m)	Depth Water Found (m)	Static Water Level (m)	Pump Rate (L/min)
2000775	1967-07-08	Domestic	20.7	17.7	16.5	7.6
2002022	1973-03-01	Domestic	6.4	4.3	4.3	11.4
2002434	1975-07-04	Domestic	8.5	3.7	3.7	18.9
2003464	1980-05-31	Domestic	59.7	58.5	12.8	22.7
2004417	1989-06-12	Domestic	71.9	71.6	24.4	22.7
2004518	1990-06-06	Domestic	72.5	71.3	25.9	18.9
2004626	1990-11-21	Domestic	50.0	46.6	0.3	22.7
2005352	1997-03-04	Domestic	9.1	2.4	2.4	18.9
2005356	1996-09-22	Domestic	9.1	2.4	2.4	18.9
2005397	1997-05-14	Domestic	13.7	1.5	1.5	11.4
2005459	1997-07-25	Domestic	9.1	4.9	0.3	30.3
7100892	2007-04-25	Domestic	71.3	68.0	21.0	37.9

NR: Not Recorded

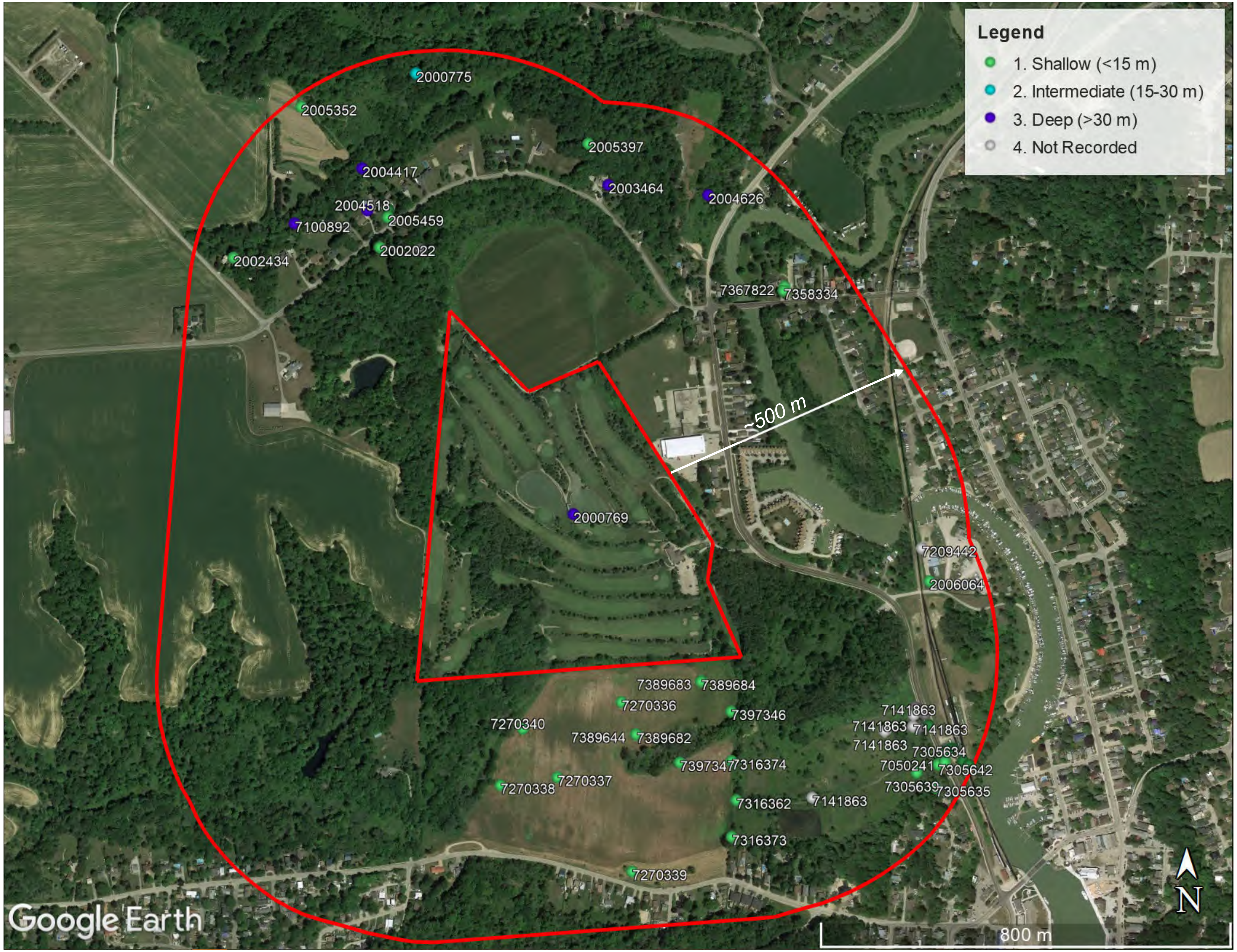
MECP OBSERVATION AND MONITORING WELLS

MECP Well ID	Registration Year	Well Type	Depth of Well (m)	Depth Water Found (m)	Static Water Level (m)	Pump Rate (L/min)
2006064	2004-01-29	Observation Well	3.65	1.85	*NR*	*NR*
7270336	2016-07-21	Monitoring and Test Hole	6.1	*NR*	*NR*	*NR*
7270337	2016-08-27	Monitoring and Test Hole	6.1	*NR*	*NR*	*NR*
7270338	2016-07-20	Monitoring and Test Hole	6.1	*NR*	*NR*	*NR*
7270339	2016-07-21	Monitoring and Test Hole	4.6	*NR*	*NR*	*NR*
7270340	2016-07-21	Monitoring and Test Hole	4.6	*NR*	*NR*	*NR*
7305633	2018-01-25	Observation Well	9.1	*NR*	*NR*	*NR*
7305634	2018-01-25	Observation Well	10.7	*NR*	*NR*	*NR*
7305635	2018-01-26	Observation Well	11.9	*NR*	*NR*	*NR*
7305636	2017-12-06	Observation Well	8.8	*NR*	*NR*	*NR*
7305637	2017-12-05	Observation Well	5.2	*NR*	*NR*	*NR*
7305639	2017-12-05	Observation Well	4.3	*NR*	*NR*	*NR*
7305640	2017-12-05	Observation Well	7.6	*NR*	*NR*	*NR*
7305641	2017-12-04	Observation Well	4.3	*NR*	*NR*	*NR*

MECP Well ID	Registration Year	Well Type	Depth of Well (m)	Depth Water Found (m)	Static Water Level (m)	Pump Rate (L/min)
7305642	2017-12-04	Observation Well	4.6	*NR*	*NR*	*NR*
7305643	2017-12-04	Observation Well	4.6	*NR*	*NR*	*NR*
7305644	2017-12-01	Observation Well	4.6	*NR*	*NR*	*NR*
7316362	2018-06-01	Observation Well	2.7	*NR*	*NR*	*NR*
7316373	2018-06-15	Observation Well	4.6	*NR*	*NR*	*NR*
7316374	2018-06-01	Observation Well	4.6	*NR*	*NR*	*NR*
7358334	2020-03-05	Monitoring and Test Hole	7.0	*NR*	*NR*	*NR*
7367822	2020-02-13	Monitoring and Test Hole	3.0	1.5	1.5	*NR*
7389644	2021-04-08	Observation Well	4.6	1.5	1.5	*NR*
7389682	2021-04-08	Observation Well	9.1	1.5	1.5	*NR*
7389683	2021-04-08	Observation Well	7.6	1.5	1.5	*NR*
7389684	2021-04-08	Observation Well	3.6	1.5	1.5	*NR*
7397346	2021-08-16	Observation Well	6.1	3.0	3.0	*NR*
7397347	2021-08-16	Observation Well	7.6	*NR*	*NR*	*NR*
NR: Not recorded						

MECP TEST HOLES AND ABANDONMENT RECORDS

MECP Well ID	Registration Year	Well Type	Depth of Well (m)	Depth Water Found (m)	Static Water Level (m)	Pump Rate (L/min)
2000769	1955-05-20	Abandoned-Supply	109.7	*NR*	*NR*	*NR*
7050241	2007-08-23	Abandoned-Other	3.6	*NR*	*NR*	*NR*
7141863	2010-02-24	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7141863	2010-02-22	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7141863	2010-02-22	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7141863	2010-02-22	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7141863	2010-02-22	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7141863	2010-02-23	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7141863	2010-02-23	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7141863	2010-02-23	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7141863	2010-02-24	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7141863	2010-02-24	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7190865	2012-10-12	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7209442	2013-09-18	Abandoned-Other	*NR*	1.8	*NR*	*NR*
NR: Not recorded						



SOURCE:
Google Earth Pro, Version 7.3.2.5776,
Coordinates 17T, 481652 m E, 4724425 m N,
Imagery date 7/2/2018

STONECAIRN CONSULTING

PROJECT NAME

Proposed Residential & Recreational Development

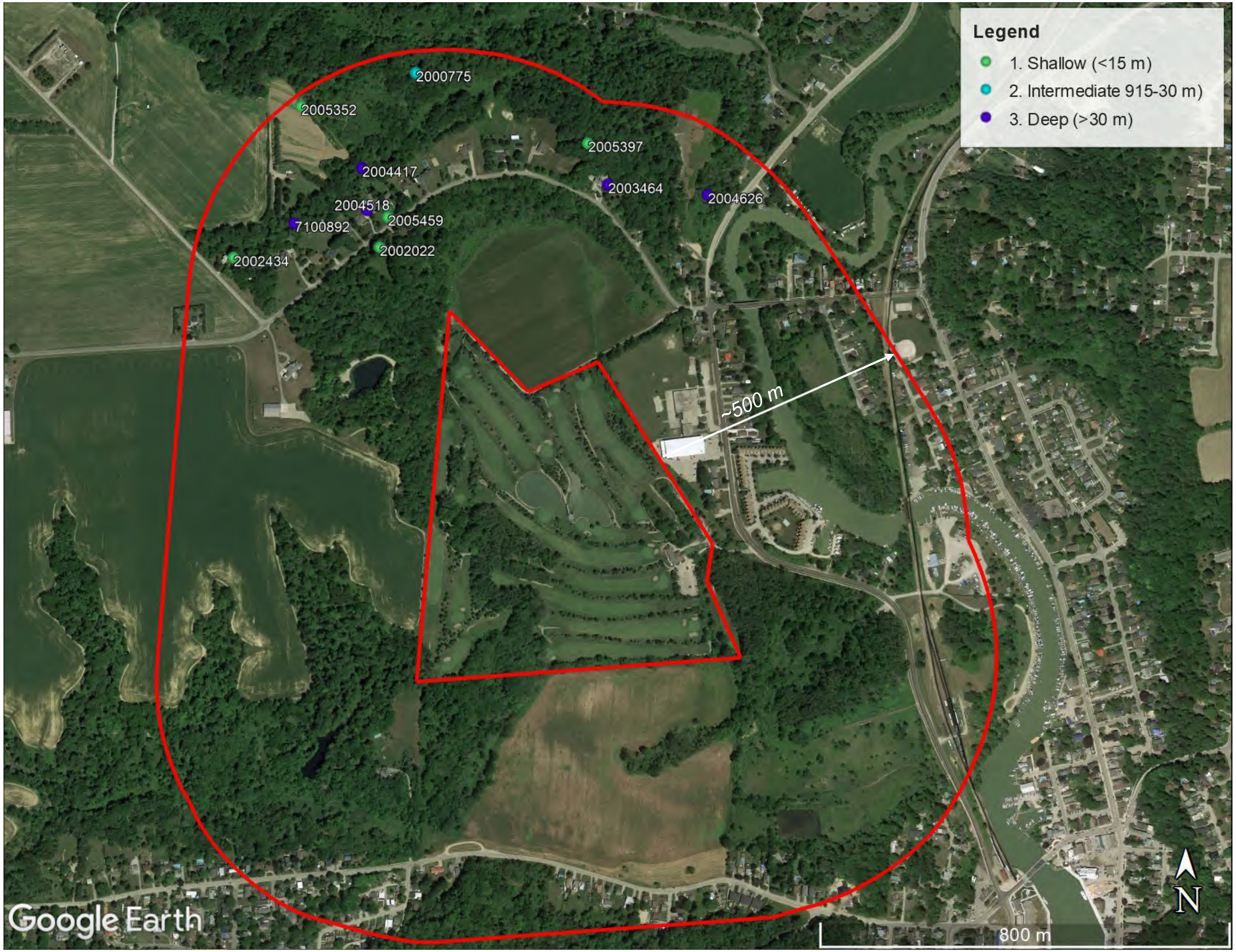
PROJECT LOCATION

320 Carlow Road, Port Stanley, ON

DRAWING NAME

MECP Well Locations – All Wells

SCALE	**PROJECT NO.**
As Shown	GE-00814
DATE	**DRAWING NO.**
February 2025	C1



SOURCE:
Google Earth Pro, Version 7.3.2.5776,
Coordinates 17T, 481652 m E, 4724425 m N,
Imagery date 7/2/2018

STONECAIRN CONSULTING

PROJECT NAME

Proposed Residential & Recreational Development

PROJECT LOCATION

320 Carlow Road, Port Stanley, ON

DRAWING NAME

MECP Well Locations – Water Supply Wells

SCALE	**PROJECT NO.**
As Shown	SC-02117
DATE	**DRAWING NO.**
February 2025	C2

APPENDIX D

SINGLE WELL RESPONSE TEST RESULTS

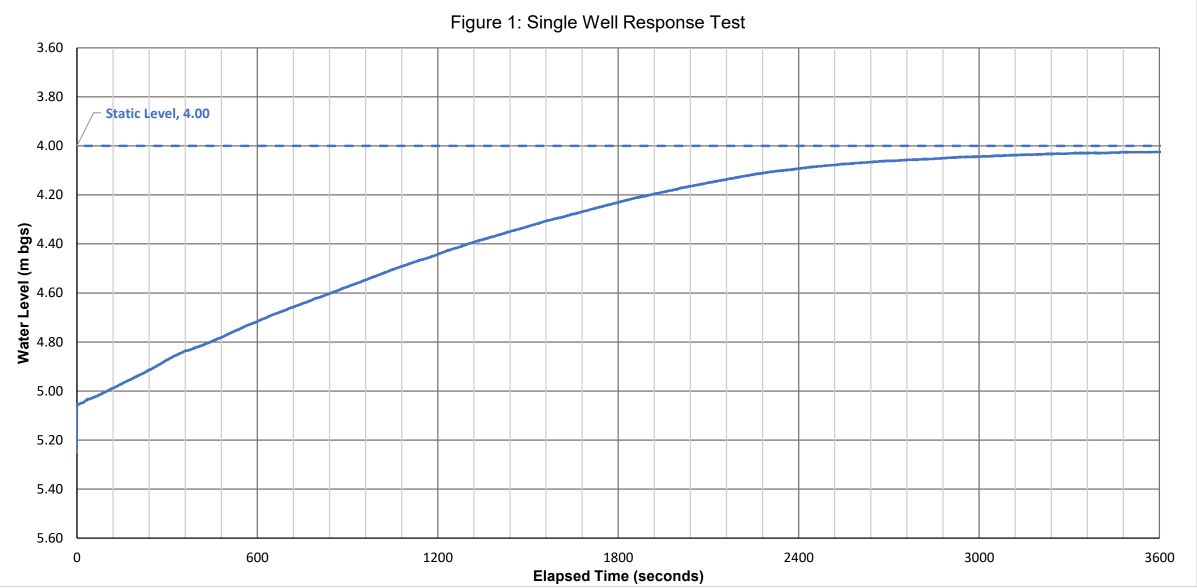
SINGLE WELL RESPONSE TEST
Estimation of Saturated Hydraulic Conductivity from Well Recovery Data

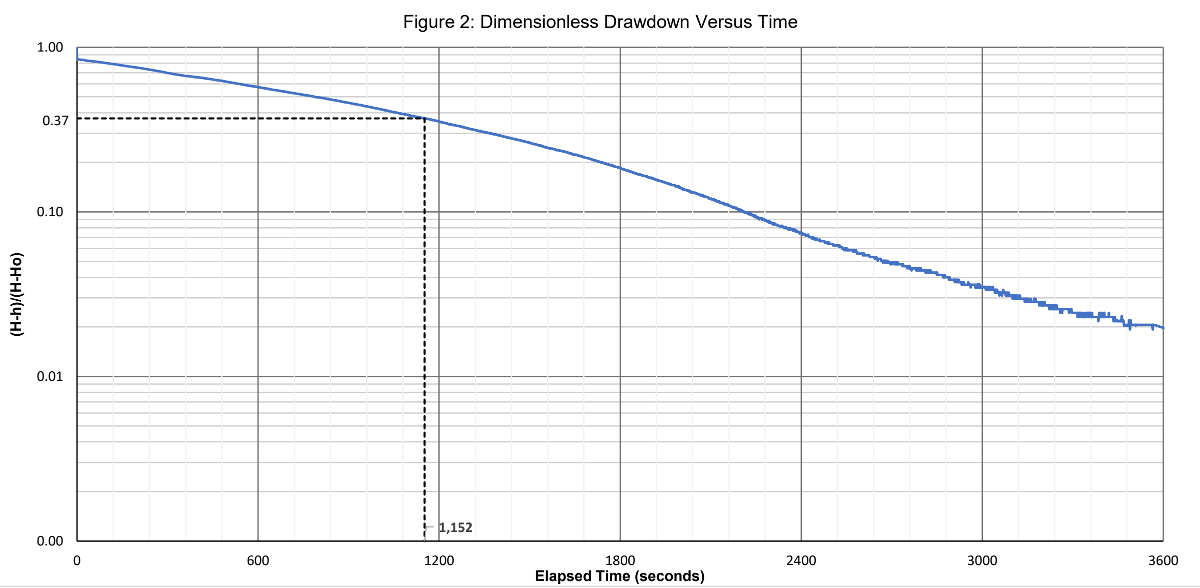
STONECAIRN CONSULTING
85 Midpark Road – Unit 3, London, Ontario N6N 1B2

Project No.	**GE-00920**	**Monitoring Well ID**	**MW4**
Project Name	**Kettle Creek Golf Course**	**Date of Testing**	**02-Jun-23**

Well Data:

Radius of well casing, r,(m) =	0.025	Data Logger S/N:	20246303
Radius of filter pack/borehole, R, (m) =	0.101	Reading Interval:	1 - 10 seconds
Length of well screen, L, (m) =	1.52		
Submerged well screen length, Ls, (m) =	1.32	Gravel Pack Correctior $R_{equiv} = [(1-n)r^2 + nR^2]^{½}$	0.070
Static water level, h, (m) =	4.00		
Porosity of Gravel Pack, n (%) =	0.45		





Estimation of Saturated Hydraulic Conductivity From Hvorslev's Equation

Hvorslev's Equation: $K = \frac{r^2(\ln(L/R))}{2(To)(L)}$ for L/R > 8

where

K = Hydraulic Conductivity (m/s)
r = Radius of the well casing (m)
R = Radius of the well screen (m)
L = Length of the submerged portion of the well screen under static conditions (m)
To = Basic time lag; time for water level to rise or fall to 37% of the initial change (Sec)

Check if L/R > 8	L/R =	15.0	
Evaluation of basic time lag	To=	1152	sec
Estimated Saturated Hydraulic Conductivity	**K=**	**6.03E-07**	**m/sec**

SINGLE WELL RESPONSE TEST
Estimation of Saturated Hydraulic Conductivity from Well Recovery Data

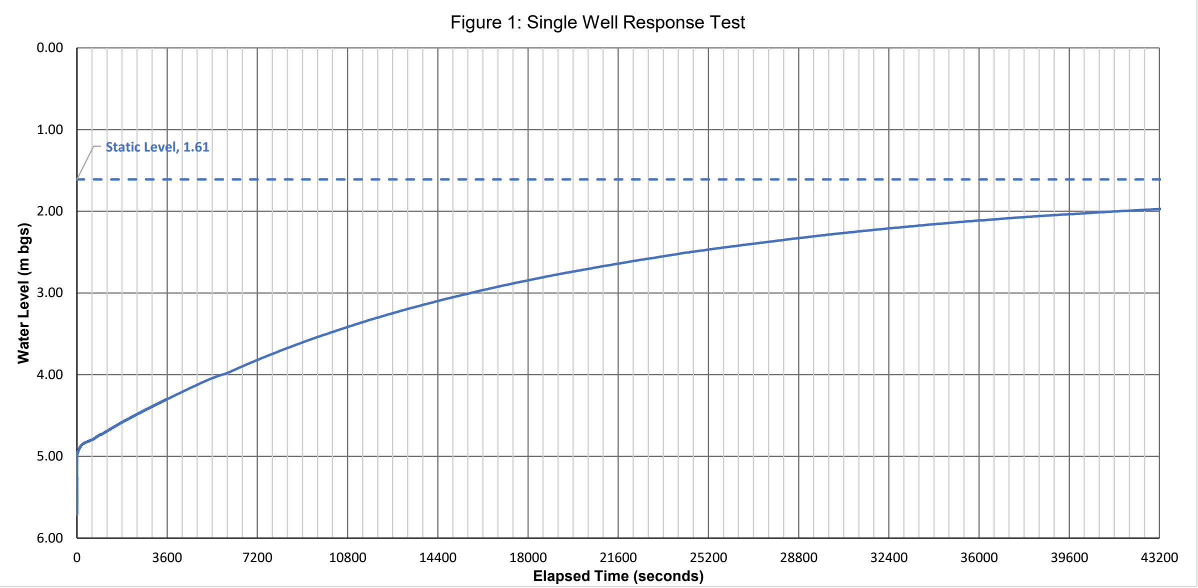
Project No. **GE-00920** **Monitoring Well ID** **MW7**
Project Name **Kettle Creek Golf Course** **Date of Testing** **02-Jun-23**

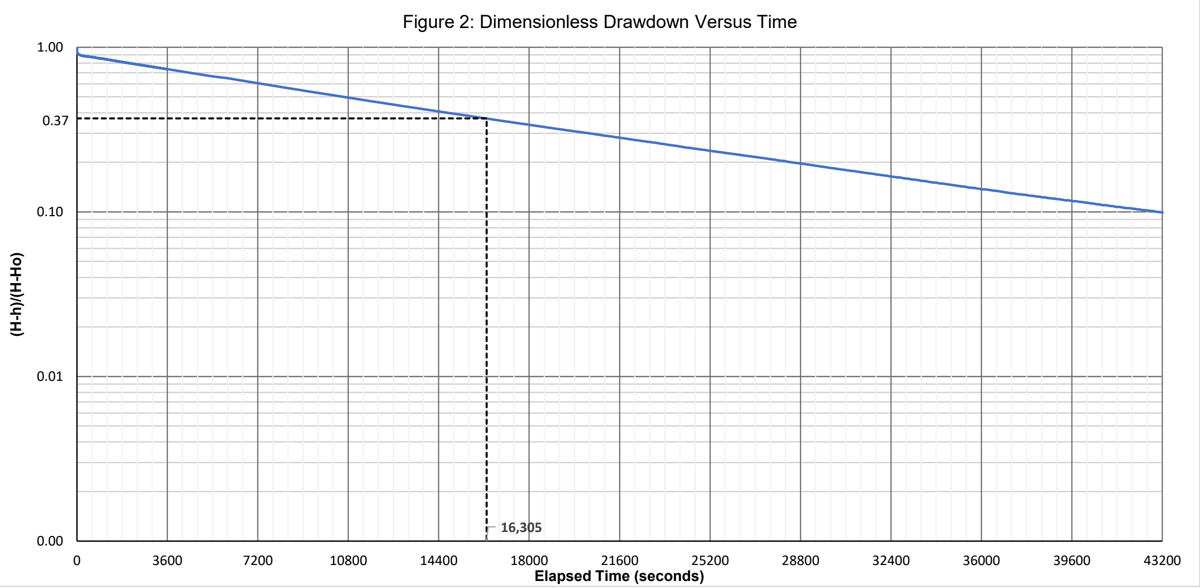
Well Data:

Radius of well casing, r,(m) = 0.025
Radius of filter pack/borehole, R, (m) = 0.101
Length of well screen, L, (m) = 1.52
Submerged well screen length, Ls, (m) = 1.32
Static water level, h, (m) = 1.61
Porosity of Gravel Pack, n (%) = 0.45

Data Logger S/N: 20246303
Reading Interval: 1 - 10 seconds

Gravel Pack Correctior $R_{equiv} = [(1-n)r^2 + nR^2]^{½}$ 0.070





Estimation of Saturated Hydraulic Conductivity From Hvorslev's Equation

Hvorslev's Equation: $K = \frac{r^2(\ln(L/R))}{2(To)(L)}$ for L/R > 8

where

K = Hydraulic Conductivity (m/s)
r = Radius of the well casing (m)
R = Radius of the well screen (m)
L = Length of the submerged portion of the well screen under static conditions (m)
To = Basic time lag; time for water level to rise or fall to 37% of the initial change (Sec)

Check if L/R > 8	L/R =	15.0	
Evaluation of basic time lag	To=	16305	sec
Estimated Saturated Hydraulic Conductivity	**K=**	**4.26E-08**	**m/sec**

APPENDIX E

ANALYTICAL TEST RESULTS

351 Nash Road North, unit 9B
Hamilton, ON L8H 7P4
1-800-749-1947
www.paracellabs.com

Certificate of Analysis

LDS Consultants Inc. (London)
2323 Trafalgar Street
London, ON N5V O1E
Attn: Natascha Ungerer

Report Date: 24-May-2023
Order Date: 15-May-2023

Client PO:
Project: GE-00920
Custody:

Order #: 2320049

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
2320049-01	BH4 SA1
2320049-02	BH5 SA1
2320049-03	BH7 SA1

Approved By:

Milan Ralitsch, PhD

Senior Technical Manager

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
BTEX by P&T GC-MS	EPA 8260 - P&T GC-MS	16-May-23	17-May-23
Conductivity	MOE E3138 - probe @25 °C, water ext	18-May-23	18-May-23
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	23-May-23	23-May-23
PHC F1	CWS Tier 1 - P&T GC-FID	16-May-23	17-May-23
PHCs F2 to F4	CWS Tier 1 - GC-FID, extraction	19-May-23	23-May-23
REG 153: Metals by ICP/MS, soil	EPA 6020 - Digestion - ICP-MS	18-May-23	18-May-23
SAR	Calculated	18-May-23	19-May-23
Solids, %	CWS Tier 1 - Gravimetric	19-May-23	23-May-23

Certificate of Analysis

Client: **LDS Consultants Inc. (London)**

Client PO:

Report Date: 24-May-2023

Order Date: 15-May-2023

Project Description: GE-00920

Summary of Criteria Exceedances

(If this page is blank then there are no exceedances)

Only those criteria that a sample exceeds will be highlighted in red

Regulatory Comparison:

Paracel Laboratories has provided regulatory guidelines on this report for informational purposes only and makes no representations or warranties that the data is accurate or reflects the current regulatory values. The user is advised to consult with the appropriate official regulations to evaluate compliance. Sample results that are highlighted have exceeded the selected regulatory limit. Calculated uncertainty estimations have not been applied for determining regulatory exceedances.

Sample	Analyte	MDL / Units	Result	Reg 406/19 -T2.1 Res/Park/Inst	-

Certificate of Analysis

Report Date: 24-May-2023

Client: **LDS Consultants Inc. (London)**

Order Date: 15-May-2023

Client PO:

Project Description: GE-00920

Client ID:		BH4 SA1	BH5 SA1	BH7 SA1	-	**Criteria:**	
Sample Date:		10-May-23 00:00	10-May-23 00:00	10-May-23 00:00	-	**Reg 406/19 -T2.1 Res/Park/Inst**	**-**
Sample ID:		2320049-01	2320049-02	2320049-03	-		
Matrix:		Soil	Soil	Soil	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	80.8	53.6	77.6	-	-	-
General Inorganics							
SAR	0.01 N/A	0.14	0.51	0.35	-	5 N/A	-
Conductivity	5 uS/cm	409	419	195	-	0.7 mS/cm	-
pH	0.05 pH Units	7.70	7.28	7.45	-	5.00 - 9.00 pH Units	-
Metals							
Antimony	1 ug/g	<1.0	<1.0	<1.0	-	7.5 ug/g	-
Arsenic	1 ug/g	3.9	5.0	7.7	-	18 ug/g	-
Barium	1 ug/g	97.8	143	87.1	-	390 ug/g	-
Beryllium	0.5 ug/g	0.5	0.5	0.7	-	4 ug/g	-
Boron	5 ug/g	7.1	<5.0	8.3	-	120 ug/g	-
Cadmium	0.5 ug/g	<0.5	<0.5	<0.5	-	1.2 ug/g	-
Chromium	5 ug/g	19.1	18.9	27.0	-	160 ug/g	-
Cobalt	1 ug/g	7.6	6.4	11.7	-	22 ug/g	-
Copper	5 ug/g	17.0	21.8	16.7	-	140 ug/g	-
Lead	1 ug/g	7.8	6.8	21.2	-	120 ug/g	-
Molybdenum	1 ug/g	<1.0	<1.0	<1.0	-	6.9 ug/g	-
Nickel	5 ug/g	17.4	15.0	21.7	-	100 ug/g	-
Selenium	1 ug/g	<1.0	<1.0	<1.0	-	2.4 ug/g	-
Silver	0.3 ug/g	<0.3	<0.3	<0.3	-	20 ug/g	-
Thallium	1 ug/g	<1.0	<1.0	<1.0	-	1 ug/g	-
Uranium	1 ug/g	<1.0	<1.0	<1.0	-	23 ug/g	-
Vanadium	10 ug/g	27.9	33.0	39.0	-	86 ug/g	-
Zinc	20 ug/g	46.9	59.6	84.4	-	340 ug/g	-
Volatiles							

Client ID:		BH4 SA1	BH5 SA1	BH7 SA1	-	**Criteria:**	
Sample Date:		10-May-23 00:00	10-May-23 00:00	10-May-23 00:00	-	**Reg 406/19 -T2.1 Res/Park/Inst**	-
Sample ID:		2320049-01	2320049-02	2320049-03	-		
Matrix:		Soil	Soil	Soil	-		
	MDL/Units						
Volatiles							
Benzene	0.02 ug/g	<0.02	<0.02	<0.02	-	0.02 ug/g	-
Ethylbenzene	0.05 ug/g	<0.05	<0.05	<0.05	-	0.05 ug/g	-
Toluene	0.05 ug/g	<0.05	<0.05	<0.05	-	0.2 ug/g	-
m,p-Xylenes	0.05 ug/g	<0.05	<0.05	<0.05	-	-	-
o-Xylene	0.05 ug/g	<0.05	<0.05	<0.05	-	-	-
Xylenes, total	0.05 ug/g	<0.05	<0.05	<0.05	-	0.091 ug/g	-
Toluene-d8	Surrogate	97.8%	100%	99.6%	-	-	-
Hydrocarbons							
F1 PHCs (C6-C10)	7 ug/g	<7	<7	<7	-	25 ug/g	-
F2 PHCs (C10-C16)	4 ug/g	<4	<4	<4	-	10 ug/g	-
F3 PHCs (C16-C34)	8 ug/g	<8	<8	<8	-	240 ug/g	-
F4 PHCs (C34-C50)	6 ug/g	<6	<6	<6	-	2800 ug/g	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%REC	%REC Limit	RPD	RPD Limit	Notes
General Inorganics								
SAR	ND	0.01	N/A					
Conductivity	ND	5	uS/cm					
Hydrocarbons								
F1 PHCs (C6-C10)	ND	7	ug/g					
F2 PHCs (C10-C16)	ND	4	ug/g					
F3 PHCs (C16-C34)	ND	8	ug/g					
F4 PHCs (C34-C50)	ND	6	ug/g					
Metals								
Antimony	ND	1.0	ug/g					
Arsenic	ND	1.0	ug/g					
Barium	ND	1.0	ug/g					
Beryllium	ND	0.5	ug/g					
Boron	ND	5.0	ug/g					
Cadmium	ND	0.5	ug/g					
Chromium	ND	5.0	ug/g					
Cobalt	ND	1.0	ug/g					
Copper	ND	5.0	ug/g					
Lead	ND	1.0	ug/g					
Molybdenum	ND	1.0	ug/g					
Nickel	ND	5.0	ug/g					
Selenium	ND	1.0	ug/g					
Silver	ND	0.3	ug/g					
Thallium	ND	1.0	ug/g					
Uranium	ND	1.0	ug/g					
Vanadium	ND	10.0	ug/g					
Zinc	ND	20.0	ug/g					
Volatiles								
Benzene	ND	0.02	ug/g					
Ethylbenzene	ND	0.05	ug/g					
Toluene	ND	0.05	ug/g					
m,p-Xylenes	ND	0.05	ug/g					
o-Xylene	ND	0.05	ug/g					

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	%REC	%REC Limit	RPD	RPD Limit	Notes
Xylenes, total	ND	0.05	ug/g					
Surrogate: Toluene-d8	*7.80*		*%*	*97.4*	*50-140*			

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
General Inorganics									
SAR	1.40	0.01	N/A	1.42			1.4	30	
Conductivity	608	5	uS/cm	611			0.4	5	
pH	7.84	0.05	pH Units	7.84			0.0	10	
Hydrocarbons									
F1 PHCs (C6-C10)	ND	7	ug/g	ND			NC	40	
F2 PHCs (C10-C16)	ND	4	ug/g	ND			NC	30	
F3 PHCs (C16-C34)	ND	8	ug/g	ND			NC	30	
F4 PHCs (C34-C50)	ND	6	ug/g	ND			NC	30	
Metals									
Antimony	ND	1.0	ug/g	ND			NC	30	
Arsenic	4.3	1.0	ug/g	4.2			2.2	30	
Barium	115	1.0	ug/g	111			3.4	30	
Beryllium	0.9	0.5	ug/g	0.8			16.0	30	
Boron	31.4	5.0	ug/g	28.6			9.4	30	
Cadmium	ND	0.5	ug/g	ND			NC	30	
Chromium	20.1	5.0	ug/g	21.0			4.6	30	
Cobalt	10.9	1.0	ug/g	11.1			1.7	30	
Copper	9.3	5.0	ug/g	9.3			0.3	30	
Lead	8.9	1.0	ug/g	8.2			8.4	30	
Molybdenum	ND	1.0	ug/g	ND			NC	30	
Nickel	22.4	5.0	ug/g	23.1			3.2	30	
Selenium	ND	1.0	ug/g	ND			NC	30	
Silver	0.5	0.3	ug/g	ND			NC	30	
Thallium	ND	1.0	ug/g	ND			NC	30	
Uranium	1.6	1.0	ug/g	ND			NC	30	
Vanadium	27.0	10.0	ug/g	28.6			5.7	30	
Zinc	48.5	20.0	ug/g	50.5			4.1	30	
Physical Characteristics									
% Solids	89.0	0.1	% by Wt.	88.0			1.2	25	
Volatiles									

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Benzene	ND	0.02	ug/g	ND			NC	50	
Ethylbenzene	ND	0.05	ug/g	ND			NC	50	
Toluene	ND	0.05	ug/g	ND			NC	50	
m,p-Xylenes	ND	0.05	ug/g	ND			NC	50	
o-Xylene	ND	0.05	ug/g	ND			NC	50	
Surrogate: Toluene-d8	*8.36*		*%*		*97.5*	*50-140*			

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Hydrocarbons									
F1 PHCs (C6-C10)	64	7	ug/g	ND	90.4	80-120			
F2 PHCs (C10-C16)	91	4	ug/g	ND	105	60-140			
F3 PHCs (C16-C34)	204	8	ug/g	ND	104	60-140			
F4 PHCs (C34-C50)	129	6	ug/g	ND	91.6	60-140			
Metals									
Antimony	122	1.0	ug/g	ND	97.3	70-130			
Arsenic	140	1.0	ug/g	4.2	109	70-130			
Barium	263	1.0	ug/g	111	121	70-130			
Beryllium	116	0.5	ug/g	0.8	92.4	70-130			
Boron	144	5.0	ug/g	28.6	92.6	70-130			
Cadmium	132	0.5	ug/g	ND	106	70-130			
Chromium	154	5.0	ug/g	21.0	107	70-130			
Cobalt	141	1.0	ug/g	11.1	104	70-130			
Copper	142	5.0	ug/g	9.3	106	70-130			
Lead	132	1.0	ug/g	8.2	98.8	70-130			
Molybdenum	135	1.0	ug/g	ND	108	70-130			
Nickel	159	5.0	ug/g	23.1	109	70-130			
Selenium	142	1.0	ug/g	ND	113	70-130			
Silver	99.7	0.3	ug/g	ND	79.7	70-130			
Thallium	127	1.0	ug/g	ND	102	70-130			
Uranium	142	1.0	ug/g	ND	114	70-130			
Vanadium	165	10.0	ug/g	28.6	109	70-130			
Zinc	190	20.0	ug/g	50.5	112	70-130			
Volatiles									
Benzene	3.54	0.02	ug/g	ND	88.6	60-130			
Ethylbenzene	3.54	0.05	ug/g	ND	88.6	60-130			
Toluene	3.54	0.05	ug/g	ND	88.5	60-130			
m,p-Xylenes	7.08	0.05	ug/g	ND	88.2	60-130			
o-Xylene	3.61	0.05	ug/g	ND	90.2	60-130			
Surrogate: Toluene-d8	*7.89*		*%*		*98.6*	*50-140*			

Certificate of Analysis

Report Date: 24-May-2023

Client: **LDS Consultants Inc. (London)**

Order Date: 15-May-2023

Client PO:

Project Description: GE-00920

Qualifier Notes:

Sample Data Revisions:

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

NC: Not Calculated

Soil results are reported on a dry weight basis unlesss otherwise noted.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

CCME PHC additional information:

- The method for the analysis of PHCs complies with the Reference Method for the CWS PHC and is validated for use in the laboratory. All prescribed quality criteria identified in the method has been met.
- F1 range corrected for BTEX.
- F2 to F3 ranges corrected for appropriate PAHs where available.
- The gravimetric heavy hydrocarbons (F4G) are not to be added to C6 to C50 hydrocarbons.
- In the case where F4 and F4G are both reported, the greater of the two results is to be used for comparison to CWS PHC criteria.
- When reported, data for F4G has been processed using a silica gel cleanup.

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Chain Of Custody
(Lab Use Only)

Client Name: LDS Consultants Inc.

Contact Name: Natascha Ungerer

Address: 2323 Trafalgar Street
London, ON

Telephone: 226-289-2952

Project Ref: GE-00920

Quote #:

PO #:

E-mail: rebecca.walker@ldsconsultants.ca
natascha.ungerer@ldsconsultants.ca

Page 1 of 1

Turnaround Time

☐ 1 day ☐ 3 day
☐ 2 day ☒ Regular

Date Required:

☐ REG 153/04 ☒ REG 406/19

☐ Table 1 ☒ Res/Park ☐ Med/Fine
☒ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/Other
☐ Table ___
For RSC: ☐ Yes ☐ No

Other Regulation

☐ REG 558 ☐ PWQO
☐ CCME ☐ MISA
☐ SU - Sani ☐ SU - Storm
Mun:
☐ Other:

Matrix Type: **S** (Soil/Sed.) **GW** (Ground Water)
SW (Surface Water) **SS** (Storm/Sanitary Sewer)
P (Paint) **A** (Air) **O** (Other)

Required Analysis

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken Date	Sample Taken Time	ICP Metals	BTEX, PHCs	EC, SAR	m-SPLP metals		PAHs	pH	PCBs	ON HOLD			
1	BH4 SA1	S		2	May 10, 2023		☒	☒	☒	☐	☐	☐	☒	☐	☐	☐	☐	☐
2	BH5 SA1	S		2	↓		☒	☒	☒	☐	☐	☐	☒	☐	☐	☐	☐	☐
3	BH7 SA1	S		2	↓		☒	☒	☒	☐	☐	☐	☒	☐	☐	☐	☐	☐
4							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
5							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
6							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
7							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
8							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
9							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
10							☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments: cooling initiated

Method of Delivery: walk in

Relinquished By (Sign):

Relinquished By (Print): Natascha Ungerer

Date/Time: May 15, 2023 (13:20)

Received By Driver/Depot:

Date/Time: May 15/23 13:36

Temperature: 17.4 °C

Received at Lab: TMcCalla

Date/Time: 05/16/23 1022

Temperature: 9.8

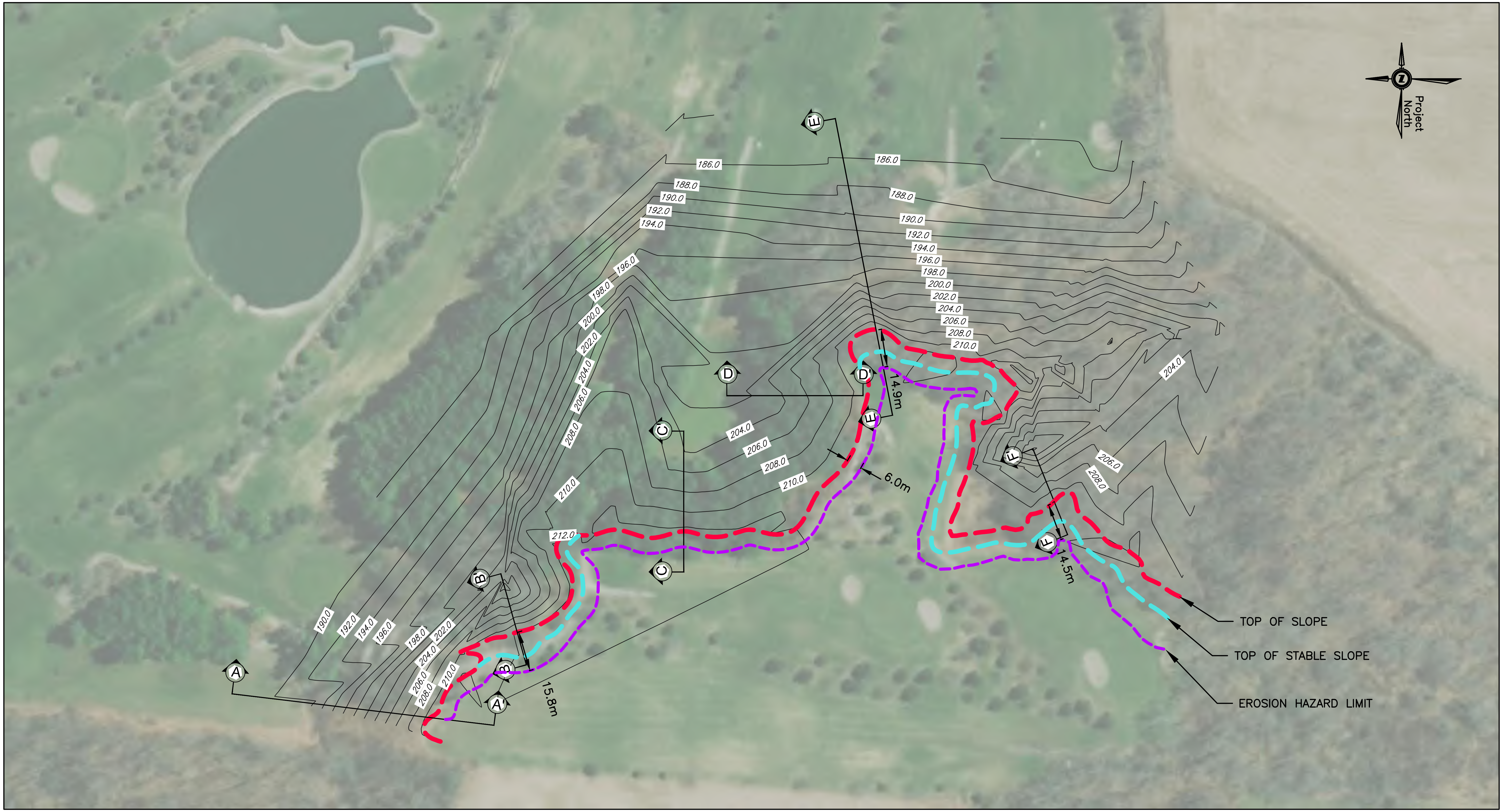
Verified By: TMcCalla

Date/Time: 05/16/23 1030

pH Verified: ☐ By: NA

APPENDIX F

SLOPE STABILITY RATING CHARTS



LEGEND

🟥 – TOP OF SLOPE

🟦 – TOP OF STABLE SLOPE

🟪 – EROSION HAZARD LIMIT

LICENSED PROFESSIONAL ENGINEER
R.A. WALKER
PROVINCE OF ONTARIO

CONSULTANT OR DIVISION

STONECAIRN CONSULTING
85 Midpark Road – Unit 3, London, Ontario N6N 1B2

SOURCES

Civil 3D Geolocation – Base Map
LDS Topographic Survey May 2023

SCALE

HORZ – 1 : 1500
15 0 30m

TITLE

SLOPE PLAN

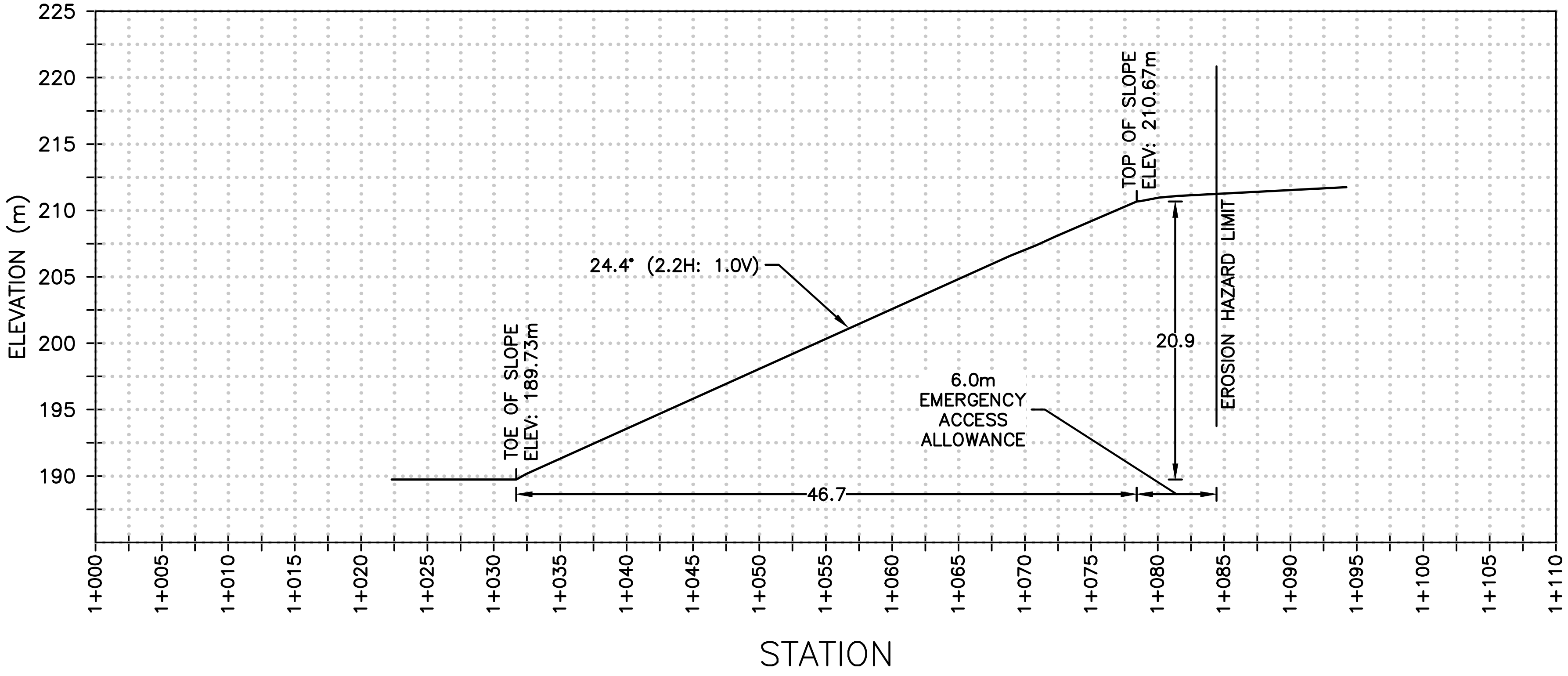
320 CARLOW ROAD
PORT STANLEY, ONTARIO

PROJFCT No
SC-02117

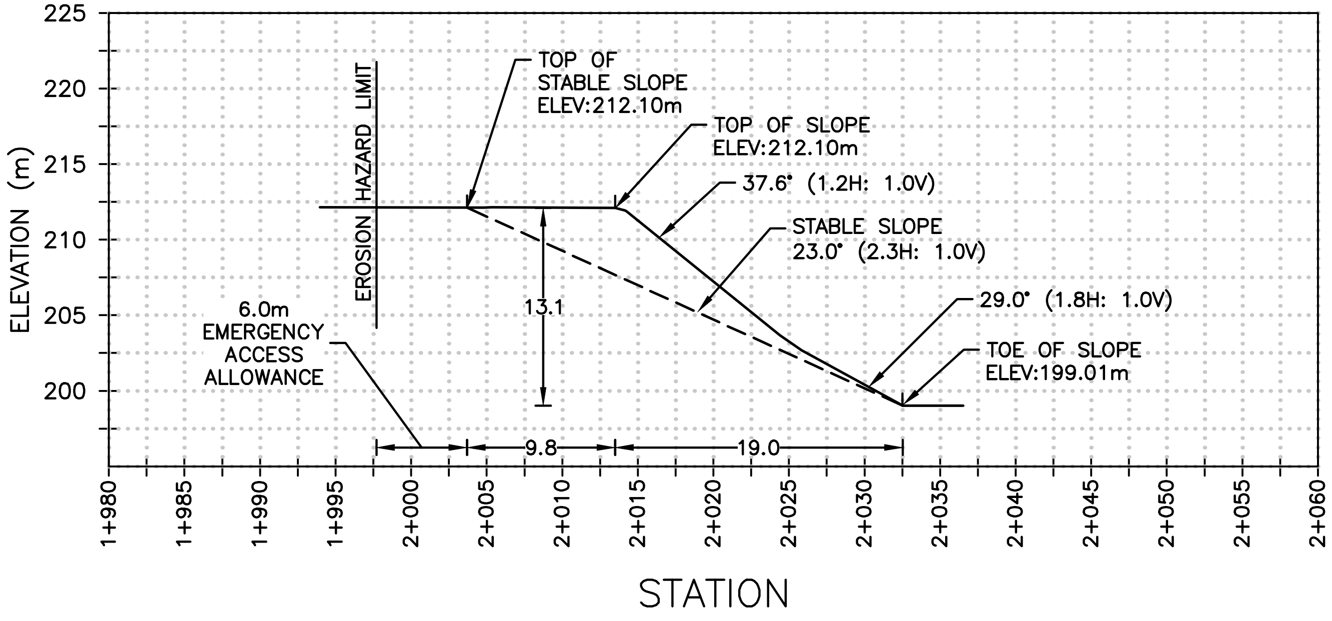
FIGURE No.
F1

FIG. Date
FEB 2025

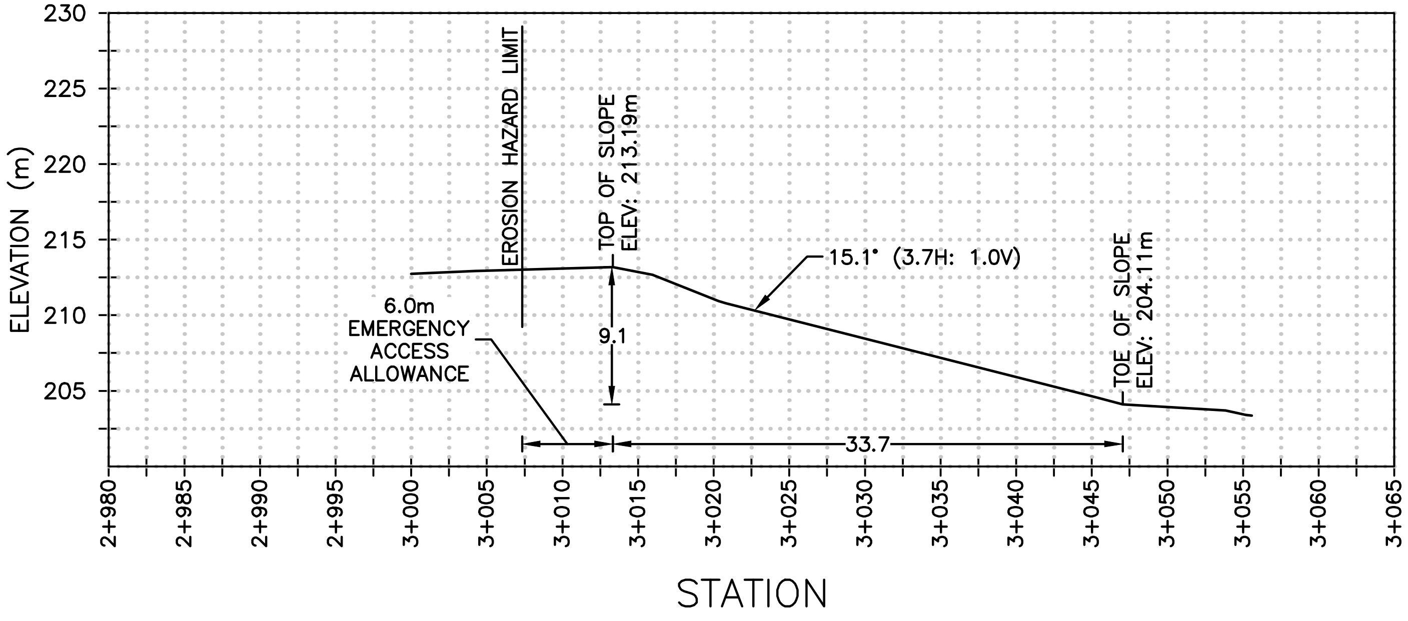
PROFILE A–A'



PROFILE B–B'



PROFILE C–C'



LEGEND

LICENSED PROFESSIONAL ENGINEER
R.A. WALKER
PROVINCE OF ONTARIO

CONSULTANT OR DIVISION

STONECAIRN CONSULTING
85 Midpark Road – Unit 3, London, Ontario N6N 1B2

SOURCES
Civil 3D Geolocation – Base Map

SCALE
HORZ – 1 : 500
5 0 10m
VERT – 1 : 50
0.5 0 1m

TITLE
PROFILES A–A', B–B', C–C'

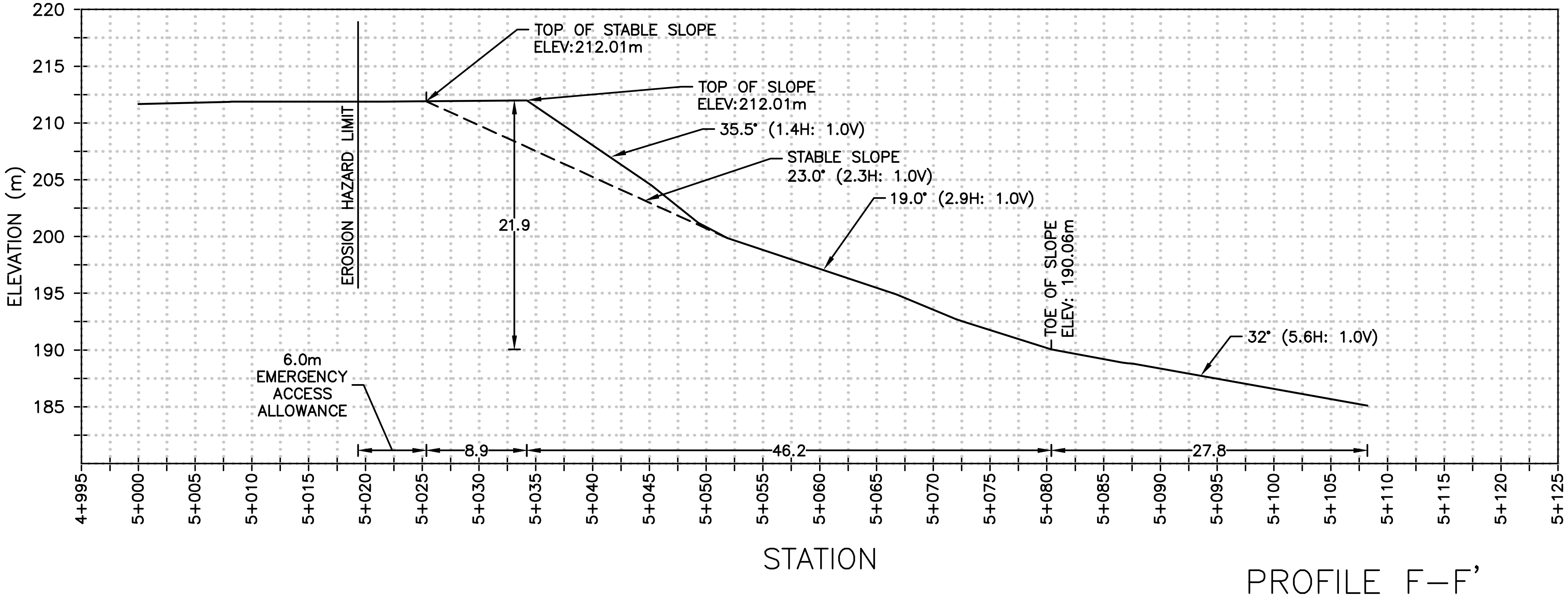
320 CARLOW ROAD
PORT STANLEY, ONTARIO

PROJECT No.
SC-02117

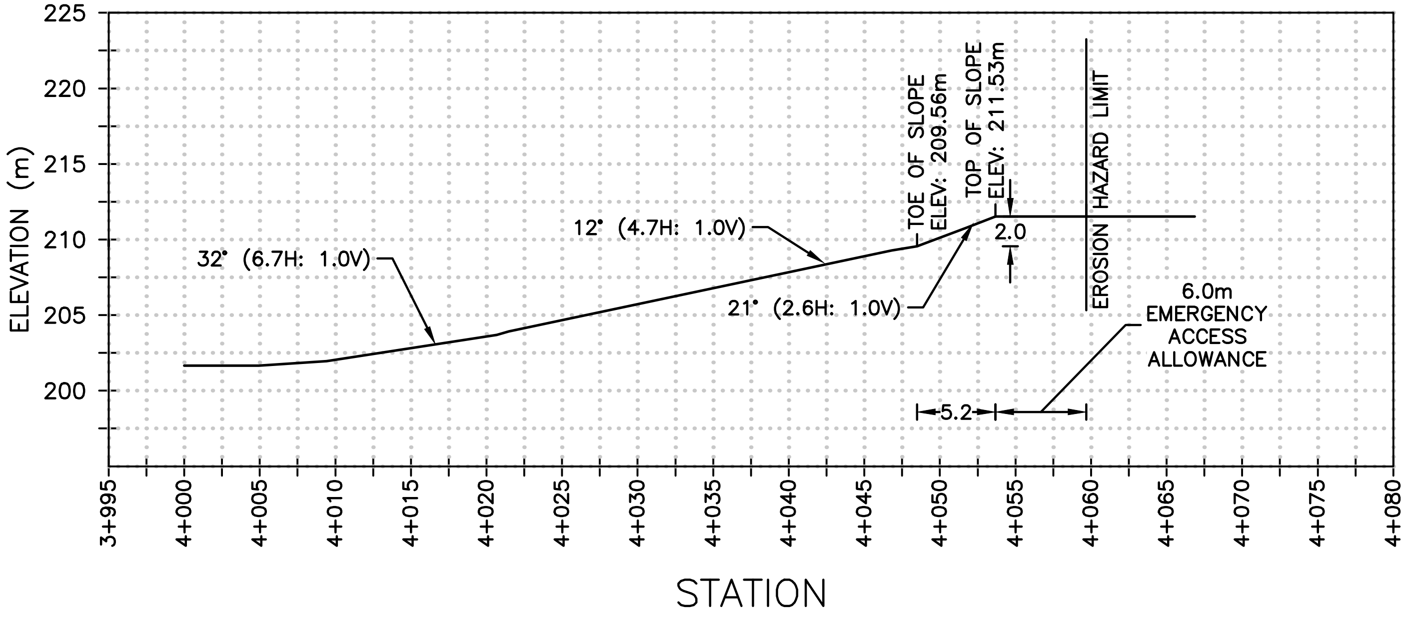
FIGURE No.
F2

FIG. Date
FEB 2025

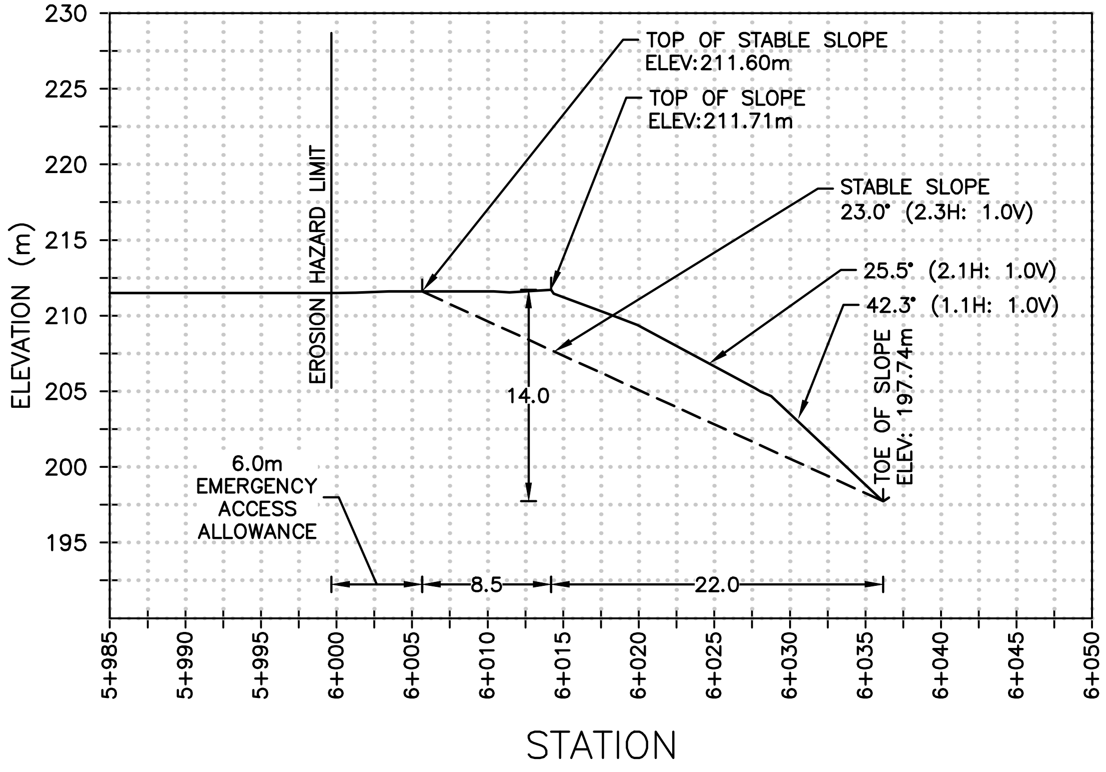
PROFILE E–E'



PROFILE D–D'



PROFILE F–F'



LEGEND

LICENSED PROFESSIONAL ENGINEER
R.A. WALKER
PROVINCE OF ONTARIO

CONSULTANT OR DIVISION

STONECAIRN
CONSULTING
85 Midpark Road – Unit 3, London, Ontario N6N 1B2

SOURCES

Civil 3D Geolocation – Base Map

SCALE

HORZ – 1 : 500
5 0 10m

VERT – 1 : 50
0.5 0 1m

TITLE

PROFILES D–D', E–E', F–F'

320 CARLOW ROAD
PORT STANLEY, ONTARIO

PROJECT No
SC-02117

FIGURE No.
F3

FIG. Date
FEB 2025

Slope Stability Rating Chart, A – A’

Geotechnical Principles for Stable Slopes
Ontario Ministry of Natural Resources

Site Location: **Kettle Creek Golf Course**
Town/City: **Port Stanley, Ontario**
Inspected by: **AH/BS**

Project No.: **GE-00920**
Inspection Date: **May 24, 2023**
Weather: **Partly Cloudy 15°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	**6**
28 degrees or more (steeper than 2H:1V)	16	
Soil Stratigraphy		
shale / limestone	0	
sand, gravel	6	
till	9	**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	**2**
drainage over slope, active erosion, gullies	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		**25**

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: Kettle Creek Golf Course	**Project No.: GE-00920**
Town/City: Port Stanley, Ontario	**Inspection Date: May 24, 2023**
Inspected by: AH/BS	**Weather: Partly Cloudy 15°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	
till	9	**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	**2**
drainage over slope, active erosion, gullies	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		**35**

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: Kettle Creek Golf Course	**Project No.: GE-00920**
Town/City: Port Stanley, Ontario	**Inspection Date: May 24, 2023**
Inspected by: AH/BS	**Weather: Partly Cloudy 15°C**

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

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, D-D’

Geotechnical Principles for Stable Slopes
Ontario Ministry of Natural Resources

Site Location: Kettle Creek Golf Course
Town/City: Port Stanley, Ontario
Inspected by: AH/BS

Project No.: GE-00920
Inspection Date: May 24, 2023
Weather: Partly Cloudy 15°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	**6**
28 degrees or more (steeper than 2H:1V)	16	
Soil Stratigraphy		
shale / limestone	0	
sand, gravel	6	
till	9	**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	**0**
2.1 to 5 m	2	
5.1 to 10 m	4	
more than 10 m	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	**2**
drainage over slope, active erosion, gullies	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		**17**

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, D-D’

Geotechnical Principles for Stable Slopes
Ontario Ministry of Natural Resources

Site Location: Kettle Creek Golf Course	**Project No.: GE-00920**
Town/City: Port Stanley, Ontario	**Inspection Date: May 24, 2023**
Inspected by: AH/BS	**Weather: Partly Cloudy 15°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	**6**
28 degrees or more (steeper than 2H:1V)	16	
Soil Stratigraphy		
shale / limestone	0	
sand, gravel	6	
till	9	**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	**2**
drainage over slope, active erosion, gullies	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		**25**

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, F-F’

Geotechnical Principles for Stable Slopes
Ontario Ministry of Natural Resources

Site Location: Kettle Creek Golf Course	**Project No.: GE-00920**
Town/City: Port Stanley, Ontario	**Inspection Date: May 24, 2023**
Inspected by: AH/BS	**Weather: Partly Cloudy 15°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	
till	9	**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	**2**
drainage over slope, active erosion, gullies	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		**35**

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.

STONECAIRN
CONSULTING

85 Midpark Road – Unit 3

London, Ontario N6N 1B2

www.stonecairn.ca