GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENTIAL DEVELOPMENT
22447 KOMOKA ROAD, KOMOKA

LDS PROJECT NO. GE-00240

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Submitted to:

1571145 ONTARIO LTD.

Distribution (via email):

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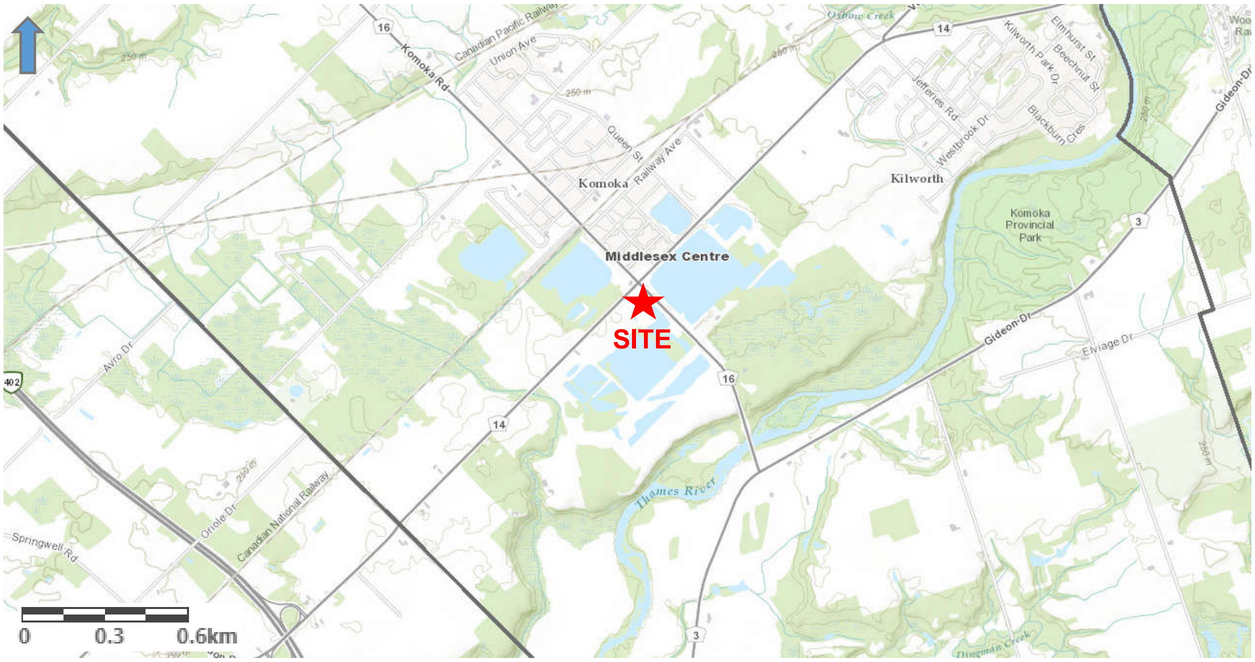
1. INTRODUCTION

LDS Consultants Inc. (LDS) has been retained by 1571145 Ontario Ltd. to conduct a Geotechnical Assessment for a proposed residential development. The subject site is located south of the intersection at Glendon Drive and Komoka Road, on the south end of the community of Komoka, Municipal Number (MN) 22447 Komoka Road. A Key Plan showing the site location is provided on Figure 1, below.

Figure 1: Key Plan



Conceptual development plans for the site include the proposed construction of two multi-storey apartment buildings, with associated site surface parking. The site is expected to be accessed from an internal roadway, which will connect to Komoka Road at the eastern end of the site, with potential connections with the existing commercial developments to the north.

The scope of work for the Geotechnical Investigation was outlined in LDS' email proposal, dated January 14, 2021. This report contains the findings of the Geotechnical Investigation. Authorization to carry out this work was received from Mr. Todd Powell, of 1571145 Ontario Ltd., on January 19, 2021.

1.1 Terms of Reference

This document has been prepared for the purposes of providing geotechnical comments and recommendations for the design and construction of a proposed residential development at MN 22447 Komoka Road.

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 development, including: site preparation (including the re-use of excavated materials as engineered fill, structural fill, and trench backfill), excavations and excavation support (including maximum slope inclinations to provide stable excavation side slopes in accordance with OHSA requirements, shoring methods, if required, and lateral earth pressures), Construction dewatering and groundwater control (including the need for a Permit to Take Water or Environmental Activity Sector Registry submission for construction dewatering, if required), foundation design (including soil bearing capacity), considerations for deep foundation alternatives (including raft slab or deep foundations, if appropriate, and allowable settlements), concrete slab construction (including modulus of subgrade reaction, and recommendations for vapour/waterproof membranes), elevator pit design and recommendations, seismic design considerations, site servicing (including the re-use of onsite soils in service trenches

The format and content of this report has been guided to address specific client needs. LDS has provided engineering guidelines for the geotechnical design and construction at the site. Laboratory testing, where applicable, follows applicable ASTM or CSA Standards. The information in this report in no way reflects on the environmental aspects of the soil.

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. If there are any changes in the design features as a result of municipal review and approval, LDS should be afforded the opportunity to review such changes to confirm that geotechnical requirements remain appropriate to support the design.

1.2 Previous Studies

In May 2019, LDS prepared a Geotechnical Design Brief for the property which assessed the feasibility of future residential development occurring at the property. The preparation of this report was a requirement by the Municipality, for the owner to demonstrate that the development potential at the site was not impaired by existing site conditions, resulting from the former gravel pit operation which was historically present at the site. The report concludes that based on a review of the available published information, and understanding of the soil and groundwater conditions which are typical for the area and anticipated at the site, that the north part of the site was considered suitable for future development. The existing pond in the south part of the site provides a beneficial amenity space, and also provides an opportunity to supplement the stormwater design elements of the site. The report also provided recommendations for more detailed Geotechnical Investigation work, which has been followed (in part) to assist in the scoping of this current Geotechnical Report.

1.3 Site Description

The site contains a former aggregate extraction operation. Based on a review of aerial photographs from 2006 to present, a former pond was identified in the northwest corner of the site, which has since been filled to restore the grade to be consistent with the surrounding lands. The site is roughly rectangular in shape, and comprises an area of approximately 5.9 hectares. From a topographical perspective, the site exhibits a gentle relief of approximately 2 m from north to south, across the site. Much of the site has grass cover, and at-source infiltration into the natural subgrade soils occurs throughout.

The site is bordered to the north by commercial land-uses, to the west by a residential development and a large pond area, to the south by a residential property and a pond area connected to the lands to the west. The site is also occupied in part by a separate large pond, which occupies the south half of the site.

The developments along the north and northwest sides of the subject property have two stormwater outlets (**with inline water quality treatment units**), which drain through existing surface channels, to the existing pond onsite. The Municipality has an **untreated** stormwater outlet located on the east side of the subject property, which discharges into the existing pond onsite.

The quality of granular material in the region has resulted in the Komoka area being a primary aggregate resource over the years. This is apparent from the number of open ponds which remain in the region, as a result of aggregate extraction below the stabilized groundwater level, and directly contributes to the conditions observed in the south end of the property. Drawing 1 (appended) shows existing site features, for reference,

2. INVESTIGATION PROGRAM

2.1 Review of Published Information

2.1.1 Review of Available Mapping

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

Site Physiography

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 the site is located the central part of the Physiographic Region known as the Caradoc Sand Plains and London Annex, and the local geological setting is within a sand plain. The subgrade soils in the area generally consist of coarse-textured glaciolacustrine deposits comprised of sand and gravel, with minor silt and clay deposits.

Quaternary Geology

Quaternary geology mapping for the London area *(Quaternary Geology, Ontario Geological Survey Map 1964, St. Thomas Area (west half), Scale 1:50,000)* indicates that the study area consists of glaciolacustrine and glaciofluvial deposits of gravel and gravelly sand from the Late Wisconsin glaciation period. The site is located near the border of a glaciofluvial outwash deposits (characterized by sand, gravel, and deltaic deposits for lands within, south and west of the village of Komoka), and Rannoch Till (characterized by silt and clayey silt deposits for lands to the north and east of the village of Komoka). An excerpt from the Quaternary Geology mapping is provided on Drawing 4, in Appendix A.

Bedrock Geology

Bedrock geology mapping for Southwestern Ontario *(Ontario Geological Survey. 1:250 000 scale, Bedrock Geology of Ontario. Ontario Geological Survey, Miscellaneous Release Data 126, Revised 2006)* indicates that bedrock in the general area consists of limestone, dolostone and shale from the Hamilton Group. The Hamilton Group (from the middle to lower Devonian period) is characterized by limestones, dolostones, and shale, which can be upwards of 15 m thick, as documented in portions of Middlesex County.

Geological publications and well records in the area indicate that the bedrock surface is below 51-54 m of overburden soils in the vicinity of the site. Bedrock was not encountered during the fieldwork for this investigation.

2.1.2 UTRCA Generic Regulation

In May 2006, Ontario Regulation 157/06 came into effect in the Upper Thames River Conservation Authority (UTRCA) watershed, which locally implements the Generic Regulation (Development, Interference with Wetlands and Alterations to Shoreline and Watercourses). This regulation replaces the former Fill, Construction and Alteration to Waterways regulations, and is intended to ensure public safety, prevent property damage and social disruption, due to natural hazards such as flooding and erosion. Ontario Regulation 157/06 is implemented by the local Conservation Authority, by means of permit issuance for works in or near watercourses, valleys, wetlands, or shorelines, when required.

As shown on Drawing 2, the pond located on the south half of the site, as well as the ponds located to the west and east of the site, are within the UTRCA regulated lands. Property owners must obtain permission from UTRCA before beginning any development, site alteration, construction, or placement of fill within the regulated area. The proposed development in the north end of the site is outside of the regulated area, and unless the plans expand into the UTRCA regulated lands around the pond, Section 28 Permits are not anticipated for the site development, with the exception of permitting associated with stormwater outletting at or near the pond area.

2.1.3 Source Water Protection Mapping

LDS has reviewed the Ministry of Environment, Conservation and Parks (MECP) Source Water Protection Information Atlas and Thames-Sydenham and Region mapping to determine whether the site is located in any identified areas of source water concern, as they relate to local groundwater quality (current to February 4, 2021). An excerpt of the mapping is provided on Drawing 3, in Appendix A.

The following observations were recorded by LDS:

- The Property is located within the Upper Thames River Source Protection Area.
- The Property is not located in any of the following designated areas listed in the MECP Source Protection mapping:
  - Wellhead Protection Area, Wellhead Protection Area E (GUDI), Wellhead Protection Area Q1 or Wellhead Protection Area Q2;
  - Intake Protection Zone or Intake Protection Zone Q;
  - Issue Contributing Area; or, Event Based Area.
- The Property is located within a Significant Groundwater Recharge Area, with a rating/score of 6, indicative of a high vulnerability rating.
- The Property is located within a Highly Vulnerable Aquifer with a rating/score of 6, indicative of a high vulnerability rating.

Based on the site being located within high vulnerability areas, it is recommended that the development of the site consider opportunities to incorporate suitable measures to allow for ‘clean’ stormwater run-off to recharge and/or infiltrate to replenish the shallow groundwater, where appropriate. Further, any construction activities carried out at the site should be carried out with measures to ensure that surface water quality (within the pond, and with stormwater runoff) does not create an adverse impact to the quality of the existing surface water features or shallow groundwater – such measures are expected to include robust erosion and sediment control measures, spills management plans, etc.).

The continued discharge of untreated stormwater from the Municipality’s outlet on the east side of the site being directed to the onsite pond, is notably contrary to the above comments and recommendations; however, it is recognized that the current landowner is not responsible to improve upon the current practices utilized by the municipality to handle the stormwater run-off generated from the broader community.

2.1.4 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. Drawing C1 in Appendix C shows the location of the wells (with corresponding Well Registration No.) which are in close proximity to the site. The water supply wells are summarized in Appendix C, for reference.

The majority of the water supply wells in the area are set into shallow (<15 m depth) unconfined or intermediate (15-30 m depth) overburden aquifers at depths ranging from 4.6 to 19.8 m. Static water levels in these water supply wells are generally reported at depths ranging from 2.4 to 6.7 m. There is no indication that artesian groundwater conditions are present in the area. 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.

The remaining well records are recorded as observation wells or test holes, as shown in Appendix C. Observation wells and test holes are recorded at variable depths within the well records.

2.2 Field Program

2.2.1 Borehole Program

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

LDS carried out a field program consisting of a series of boreholes, drilled on February 18 and 19, 2021. The boreholes were advanced at the site by a local drilling-contractor, using a track-mounted drill-rig. Five boreholes (denoted as BH1 through BH5) were advanced to depths ranging from 6.6 m (21.5 feet) to 15.7 m (51.5 feet) below existing grade. The fieldwork was supervised by members of LDS' technical staff.

Soil samples were collected from the boreholes at regular depth intervals, using Standard Penetration Test (SPT) methods, and bulk sampling methods. The SPT testing was carried out using a 63.5 kg drop hammer, falling from a height of 0.76 m. The soil stratigraphy in the boreholes was documented and logged in the field by LDS geotechnical personnel.

During collection, all samples were assessed from a visual and olfactory perspective to determine if there were any obvious signs of contamination or environmental impact. No discernible impacts were identified in the collected samples.

Soil samples were returned to LDS' laboratory to confirm the soil characteristics and for laboratory testing, which included in-situ moisture content determinations and gradation analysis on select samples. Results of the laboratory testing are provided on the borehole logs and in Appendix B.

The soil samples taken for this investigation will be stored for a period of three months following the issuance of the report. After this time, they will be discarded unless prior arrangements have been made for longer storage.

Ground surface elevations at the borehole locations were surveyed by LDS using a Trimble R10 GPS rover. The location of the boreholes is summarized below, and illustrated on Drawing 5, in Appendix A.

Table 1: Borehole Locations

Location	Northing, m N	Easting, m E	Ground Surface Elevation (m asl)
BH1/MW	4754567.06	464842.65	236.48
BH2	4754689.59	464881.82	237.63
BH3	4754680.30	464844.82	238.98
BH4	4754629.56	464833.84	236.89
BH5	4754551.19	464773.93	236.84

A monitoring well was installed in Borehole BH1 to allow for monitoring the stabilized groundwater level at the site. The well is 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 well has been equipped with a lockable cap, and has been registered with the Ministry of Environment, Conservation, and Parks (MECP), in accordance with Ontario Regulation (O.Reg.) 903. The following table summarizes the well construction details.

Table 2: Monitoring Well Installation Details

Borehole	Ground Surface Elevation, m	Well Installation Depth, m	Screened Length, m	Screened Strata
BH1/MW	236.48	6.10	3.05	Sand and gravel and silt

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

This Geotechnical Investigation does not include any environmental / chemical testing (i.e.sampling or testing of air, soil, surface water or building materials).

3. SUMMARIZED CONDITIONS

3.1 Borehole Findings

A series of five boreholes were advanced at the site to examine soil and shallow groundwater conditions. The borehole locations are shown on Drawing 5, appended. In general, soils observed in the boreholes consisted of topsoil/fill overlying sand and gravel, silt and sand. General descriptions of subsurface conditions are summarized in the following sections. Borehole logs are provided in Appendix B, for reference.

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

Topsoil

Boreholes BH2, BH3 and BH5 were surfaced with a layer of topsoil. The topsoil consisted of brown sandy loam, and the thickness generally ranging from 150 to 300 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. If required, a more detailed analysis (involving additional shallow test pits) is recommended to accurately quantify the amount of topsoil to be removed for construction purposes.

Fill

A layer of fill was encountered below the topsoil in Boreholes BH2, BH3 and BH5 and extends to depths ranging from 1.4 to 3.2 m below ground surface. The composition of the fill was generally described as dark brown sand, with trace gravel, trace to some silt, and some topsoil inclusions, with a fine-grained texture. In Borehole BH5, a second layer of sill was encountered overlying the sand fill, which was described as dark brown silt with some sand and trace gravel.

The fill is described as being in a variable loose to compact state, based on Standard Penetration Test (SPT) N-values in the range of 4 to 15 blows per 0.3 m of split-spoon sampler penetration. Moisture content determinations conducted on recovered samples of the fill generally range between 11.2 to 19.7 percent, generally indicative of moist to very moist soil conditions.

Sand and Gravel

A layer of sand and gravel was encountered underlying the topsoil in Boreholes BH2, BH3 and BH5, and Boreholes BH1 and BH4 were surfaced within this layer. The thickness of the sand and gravel layer generally ranged from 1.6 to 4.9 m across the site. The sand and gravel was described as brown in colour, with a medium to coarse grained texture, and containing trace to some silt.

Two samples of the sand and gravel were 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 and Gravel

Sample ID	Unified Soil Classification			
	% Silt	% Sand	% Gravel	% Cobbles
BH2, Sample 2	3.2	70.1	26.7	0.0%
BH4, Sample 2	14.0	71.8	14.2	0.0%

The sand and gravel is in a variable compact to dense state, based on SPT N-values in the range of 17 to 66 blows per 0.3 m of split-spoon sampler penetration. In Borehole BH3, A loose zone (SPT N < 10 blows) was encountered within the sand and gravel layer at 5.6 m below ground surface.

Moisture content determinations conducted on recovered samples of the sand and gravel generally range between 2.9 to 9.4 percent, generally indicative of damp to moist soil conditions above the stabilized groundwater elevation, and in the order of 16.8 to 21.8 percent below the stabilized groundwater level.

Silty Sand

A layer of silty sand was encountered underlying the sand and gravel in Borehole BH5, and Borehole BH5 terminated within this layer. The sand was described as brown to grey with depth, with a fine-grained texture. Trace clay was observed within the sand layer below 10.9 m. The silty sand is in a compact to very dense state, based on SPT N-values in the range of 26 to 51 blows per 0.3 m of split-spoon sampler penetration. A loose zone (SPT N < 10 blows) was encountered within the sand layer between 4.9 and 7.1 m below ground surface.

Moisture content determinations conducted on recovered samples of the sand generally range between 15.4 to 21.0 percent, generally indicative of very moist soil conditions.

Silt

A layer of silt was encountered underlying the sand and gravel in Boreholes BH1, BH2 and BH4, and interbedded within the silty sand in Borehole BH5. Boreholes BH1, BH2 and BH4 were terminated within this layer. The silt was generally described as brown to grey with depth, and containing trace to some sand and trace clay, with a noted increase in the clay content below 5.6 m.

The silt is in a compact state, based on SPT N-values in the range of 10 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 20.5 to 26.2 percent, generally indicative of very moist soil conditions.

3.2 Soil Permeability

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

Based on the gradation results presented in Section 3.1, the following values for saturated hydraulic conductivity have been calculated. Hazen's method was used to correlate the grain size analysis to the hydraulic conductivity of the sand soils. This correlation is based on the following relationship:

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

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

Table 5: Hydraulic Conductivity & Factored Infiltration Rates

Sample ID	Sample Composition			Parameter		
	% Silt	% Sand	% Gravel	D_{10} (mm)	Saturated Hydraulic Conductivity (m/sec)	Factored Infiltration Rate (mm/hr)
BH2, Sample 2	3.2	70.1	26.7	0.171	2.92×10^{-4}	83
BH4, Sample 2	14.0	71.8	14.2	~0.050	2.50×10^{-5}	43

The natural water-bearing sand soils and sand seams have a saturated hydraulic conductivity in the range of 10^{-3} to 10^{-5} m/s, based on the above results.

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

3.3 Shallow Groundwater Conditions

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

Table 7: Short Term Groundwater Observations

Borehole	Ground Surface Elevation, m asl	Groundwater Observations, m bgs	Groundwater Elevation, m asl
BH2	237.63	Dry	--
BH3	238.98	Dry	--
BH4	236.89	1.52	235.37
BH5	236.84	2.44	234.40

The stabilized water level was recorded in the monitoring well installed in Borehole BH1 during a follow up visit to the site on March 2, 2021, and are summarized in the following table.

Table 8: Stabilized Groundwater Observations

Monitoring Well	Ground Surface Elev. (m, asl)	Depth to Groundwater (m, bgs) *Groundwater Elevation (m, asl)*	
		March 2, 2021	March 15, 2021
BH1/MW	236.48	1.72 *234.76*	1.70 *234.78*

Shallow groundwater is present within the near-surface sand and gravel soils, below Elevation 234.8 m. Shallow groundwater will vary in response to climatic or seasonal conditions, and, as such, may differ at the time of construction, with higher levels possible during mild weather conditions which create melting conditions, and during wet periods.

It is recommended that the monitoring well which has been installed at the site be maintained for the purposes of documenting changes in the stabilized groundwater level, under seasonal conditions. Consideration may be given to installing a datalogger in the monitoring well to record continuous water level data at the site.

4. GEOTECHNICAL COMMENTS AND DISCUSSION

The proposed development at the site is expected to include the construction of two multi-storey (2 to 5 storey) residential buildings, with local roadways and municipal servicing. Underground parking is not planned for the site. Surface car parking is expected to be predominantly located along the north side of the site, closest to the existing commercial lands. A local roadway within the site is expected to access Komoka Road, provide access to site parking, and may connect to the existing site pavements and private roadway on the lands to the north. A Conceptual Site Layout is provided on Drawing 6, for reference.

The boreholes generally revealed a layer of surficial topsoil/fill which is underlain by layers of sand and gravel, sand and silt. Based on stabilized groundwater levels measured in the monitoring well BH1, shallow groundwater is located approximately 1.7 m below existing ground surface (below Elevation 234.8 m.) **Based on the available soil and groundwater information, future development at the site (as described in the text of this report) is considered feasible from a geotechnical standpoint.**

The following sections of this report provide geotechnical comments and recommendations to assist with design and construction of the proposed residential development, including: site preparation (including the re-use of excavated materials as engineered fill, structural fill, and trench backfill), excavations and excavation support (including maximum slope inclinations to provide stable excavation side slopes in accordance with OHSA requirements, shoring methods, if required, and lateral earth pressures), Construction dewatering and groundwater control (including the need for a Permit to Take Water or Environmental Activity Sector Registry submission for construction dewatering, if required), foundation design (including soil bearing capacity), considerations for deep foundation alternatives (including raft slab or deep foundations, if appropriate, and allowable settlements), concrete slab construction (including modulus of subgrade reaction, and recommendations for vapour/waterproof membranes), elevator pit design and recommendations, seismic design considerations, site servicing (including the re-use of onsite soils in service trenches, pipe bedding, and trench backfill), preliminary guidance on excess soils management, and pavement design.

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. Topsoil and fill material were observed in the boreholes, and additional fill material may be anticipated between the borehole locations based on previous site activity. Topsoil stripping is anticipated throughout the area to be developed. In general, this is expected to require the removal of about 150 to 300 mm of surficial topsoil. Thicker topsoil areas may be present between the borehole locations.

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.

Existing fill material encountered in the boreholes was noted to be in a loose state, and contains intermittent pockets with organic inclusions. As such, if the existing fill material is proposed to remain onsite within the proposed building footprints, it is recommended that the fill be excavated and examined by a geotechnical engineer to confirm its suitability. Where the fill is free of topsoil and organics, it can be re-used as engineered fill (and placed in accordance wit the recommendations noted below). Pockets containing organics should be removed, and limited for re-use to areas where some settlements can be tolerated.

Exposed subgrade soils should be thoroughly proof-rolled and inspected by the geotechnical consultant. Any loose or soft zones noted during the inspection should be over excavated and replaced with approved fill.

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 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 in Appendix A.

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

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

If site grading and subgrade preparation work is carried out in winter months, care should be taken to ensure that the exposed subgrade soils, and any fill material being used within the engineered fill pad is free of frozen and frost-laden material. The possible re-use of onsite soils should be subject to review and approval by the geotechnical consultant.

4.1.2 Rerouting of Existing Drains

An existing drainage channel (downstream of the stormwater outlets for the lands to the north and northwest of the property) crosses the plateau area and drains stormwater run-off to the onsite pond. In the event that as part of the development, the existing ditch needs to be rerouted or diverted through a series of pipes, to provide clearance for the proposed buildings, the open channel should be properly decommissioned, including removing sediment build-up and restoration to design grades with approved fill material. Geotechnical oversight, including inspection and testing will be required for this work.

4.1.3 Excess Soil Management Considerations

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

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;
- Preparation and implementation of a sampling and analysis plan;
- Preparation of a soil characterization report;
- Preparation of an excess soil destination assessment report; and,
- Development and implementation of a tracking system.

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. LDS has staff that can provide this service, if required.

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

Table 9: Minimum Number of Samples

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

Table 10: Minimum Analytical Requirements

Parameters to be analyzed	Surface and Subsurface Soils	SWM Ponds
Metals (including Hydrides)	✓	✓
Benzene, Toluene, Ethylbenzene, and Xylenes (BTEX)	✓	✓
Petroleum Hydrocarbons (PHCs) F1 – F4	✓	✓
Polycyclic Aromatic Hydrocarbons (PAHs)	-	✓
pH, EC, SAR	✓	✓
Cyanide (WAD)	-	✓
Leachate Analysis	See Note 1	See Note 2

Notes
1. Leachate analysis is conditional on contaminant of potential concern being identified by the QP, the volume of excess soil exceeding 350m^3 and applicable standards
2. Leachate analysis is always required for metals and hydride-forming metals

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

In the event that the grades at the Site are to be raised, and the Site will be in need of imported fill, the Regulation also outlines requirements which need to be met as a Beneficial Re-Use (receiving) Site. As a Beneficial Re-Use Site, you are expected to retain a Qualified Person (QP) to prepare an Excess Soil Destination Assessment Report (ESDAR), which outlines the geotechnical requirements for beneficial reuse of imported materials onsite (much of which can be taken directly from this Geotechnical Report for the Site), along with the environmental soil quality criteria (including the applicable O.Reg. 153/04 Site Condition Standards) for material you are willing to accept at the Site. This is generally prescribed by the Site setting.

For the purpose of importing and stockpiling materials at the site, consideration should be given to accepting material which has concentrations consistent with, or less than the standard concentrations identified in O. Reg. 153 (last amended April 15, 2011) for Table 1 (residential land-use) Standard Site Conditions. This standard is recommended, due to the presence of the existing pond within the southern part of the site.

4.2 Methane Abatement

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

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

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

4.3 Excavations and Groundwater Control

Excavations for the proposed buildings are generally expected to extend into the natural sand, sand and gravel, and silt, or possible engineered fill material, depending on final site grades.

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

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

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

Health and Safety Act. The engineered shoring system, if required, must be in place prior to commencement of the installation operations.

4.3.1 Excavation Support

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

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

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

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

Table 11: Soil Parameters for Excavation Support

Soil	***φ***	***γ (kN/m^3)***	***K_a***	***K_o***	***K_p***
Compact Silty Sand / Silt	28	19.5	0.36	0.53	2.78
Compact Sand and Gravel	30	20.5	0.33	0.50	3.15
Compact Granular ‘B’ (OPSS 1010)	32	22.0	0.31	0.47	3.25

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

4.3.2 Groundwater Control

Based on the results of the investigation, shallow groundwater is located approximately 1.7 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.

Where excavations extend below the stabilized groundwater table, or where groundwater levels are elevated, positive groundwater control methods may need to be utilized for construction dewatering. Soil permeability values in the natural sandy subgrade soils are expected to be in the range of 10^{-3} to 10^{-5} m/s, based on laboratory and field testing (presented in Section 3.1.2). This information is provided to assist with determining appropriate construction dewatering methods.

Groundwater control measures at the site should help to maintain stable excavated slopes; reduce saturated soil conditions to allow for possible reuse of excavated material; and provide a dry and stable base for excavations and construction operations. A dewatering plan should be submitted by excavating contractors involved in site servicing work for the subdivision. To assist in preparation of the dewatering plan, consideration should be given to carrying out a series of pre-tender test pits for contractors to obtain a better appreciation of the behaviour of excavations and to confirm dewatering requirements. Contractors (including specialist dewatering contractors) who might be involved in the job should witness these test pits.

It should be noted that for projects requiring positive groundwater control with a removal rate in excess of 50,000 litres per day, a Permit to Take Water (PTTW) or a submission to the Environmental Activity and Sector Registry (EASR) will be required. If excavations for site servicing extend below the stabilized groundwater table, an EASR will be required for water taking volumes up to 400,00 litres per day. If larger volumes are required as a result of excavation depth and anticipated open cut excavation lengths, a Category 3 PTTW would be required for groundwater control over 400,000 litres per day. PTTW applications are submitted to and approved by MECP according to Sections 34 and 98 of the Ontario Water Resources Act R.S.O. 1990 and Water Taking and Transfer Regulation O. Reg. 387/04.

Some of the factors which directly contribute to the volume required for a Permit to Take Water include the following:

- Localized variations in soil conditions;
- Seasonal influences on stabilized water table;
- Design depth for excavations;
- Length and staging to advance continuous open-cut excavations (i.e.: excavations for site servicing); and,
- Methodology and experience of the contractor.

For construction dewatering requiring an EASR or PTTW, a construction dewatering plan and discharge plan will be required to estimate the quantity of water to be removed. The dewatering plan should also include calculations for the zone of influence, identify potential impacts to existing structures, and identify potential qualitative and quantitative impacts to nearby properties which rely on the shallow groundwater table as a potable water source. Details regarding volume monitoring, water quality analyses and method / location of discharge water will also be required as part of the Permit to Take Water submission.

4.3.3 Seasonal Groundwater Monitoring

As noted previously, the existing well BH1 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. LDS would be pleased to assist in providing a scope and budget for this work.

4.4 Building Design and Construction

4.4.1 Foundation Design

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. For design of footings on the natural subgrade soils below 1.2 m below existing grades, the following allowable bearing pressures (net stress increase) can be used for design of footings at the depths noted below:

Table 12: Soil Bearing Capacity

Location	Serviceability Limit States (SLS) / Ultimate Limit States (ULS)	
	190 kPa / 225 kPa	245 kPa / 280 kPa
BH1	1.2 m bgs	2..3 m bgs
BH2	1.4 m bgs See Note 1	3.0 m bgs
BH3	2.1 m bgs See Note 1	2.4 m bgs
BH4	1.2 m bgs	1.2 m bgs
BH5	3.2 m bgs	~ 10 m bgs

Notes:

1. *Bearing of 190 kPa may be possible above this level if unstable fill material is removed and replaced with approved engineered fill.*

For footings set on engineered fill, an SLS design net bearing pressure of 190 kPa should be utilized for design purposes. It should be noted that the recommended bearing capacities for the approved engineered fill mat, are based on full time inspection and testing. If this work is not carried out with geotechnical supervision and/or the geotechnical consultant is unable to provide geotechnical certification of the engineered fill mat, additional site review and intrusive testing may be required to verify the soil bearing capacity for foundations set on fill material All footings exposed to seasonal freezing conditions should be protected from frost action by at least 1.2 m 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 base and/or sidewalls are present. Consequently, after the founding surfaces have been exposed, the soils should be thoroughly recompacted to provide a uniform base, suitable to provide the bearing capacity noted above. Consideration should be given to placing concrete foundations as soon as possible following excavation and subgrade inspection.

Excessive differential settlements can occur where the subgrade support material types differ below the underside of continuous strip footings, (i.e., natural 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 LDS' geotechnical engineer is recommended to confirm soil conditions and verify soil bearing capacity.

In the event that higher bearing capacities are desired, consideration may be given to the use of deep foundation schemes extending down into dense subgrade soils. Additional depth boreholes would be required in this regard, to assess the soil bearing at greater depth. LDS can assist in preparing a scope and budget for this additional work if required.

4.4.2 Concrete Slab Construction

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

In preparation for the construction of the floor slab, any unstable soils should be removed and/or recompacted to ensure that founding soils which will support the floor slab are suitable. In the event that the exposed subgrade soils are wet they will exhibit a greater sensitivity to disturbance.

Care should be taken to protect the subgrades below the floor slabs 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 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.

For water-tight foundations, the earth pressures against the exterior walls will need to account for the build-up of porewater pressures in the adjacent soils. Stabilized seasonal high groundwater levels should be used in the design, to account for permanent conditions at the site.

4.4.3 Foundations and Shallow Groundwater

A review of the Site grading 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. Shallow groundwater may be present at/near the design underside of footing elevation, where wet soils are present. The addition of subfloor drains, connected to the sump pump may be advised. Site review by the geotechnical consultant can assist in this regard. Consideration may be given to enhanced damp-proofing measures (such as subfloor drains), where there is reasonable concern that in-ground levels may conflict with the high groundwater level on an intermittent basis.

Where in-ground levels are expected to extend below the stabilized groundwater level, waterproofing will be required, and the effects of buoyancy will need to be considered within the design.

When buildings are designed with water-tight foundations, care should be taken in the design and installation of the water-proofing measures to avoid moisture problems, or actual water seepage through any normal shrinkage cracks which may develop in the concrete walls, floor slabs, foundations and/or construction joints.

Care will be required during the installation of the water-proofing membrane to ensure that it is not cut or otherwise damaged during installation, during the construction of the raft slab foundation, or during the construction of the building foundations and backfilling activities.

Where penetrations through the water-proofing membranes do occur to accommodate pipes or pits, the penetration areas will need to be properly sealed, to provide a continuous extension of the water-tight measures through the base of the excavation, and adjacent to the foundation sidewalls.

Inspection of the water-proofing installation is recommended.

4.4.4 Elevator Shafts

Based on the available soil conditions, the following general comments are provided regarding the excavation of elevator shafts:

- Shaft design may consider the use of drilled shafts to limit the extent of open cut excavation below the lowest floor slab level.
- If drilled shafts are utilized, sufficient liners should be provided by the contractor to prevent loss of ground. The liners should be advanced before the auger and removal of any materials.
- If open cut excavations are utilized, excavation support may be required to limit excavation size and amount of disturbed soil around the pit location.

Elevator pits are expected to extend below the lowest parking level; and may be set below the stabilized groundwater level depending on the depth. Water proofing may be required depending on the design depth.

Once elevator shaft locations and details are known, additional comments and recommendations may be provided to supplement the information available in this report.

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 and equipment foundations should have regard to the above referenced standard, and should be reviewed by the structural engineer for conformance to the CSA standards.

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

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

4.4.6 Seismic Design Considerations

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

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2012 indicated that to determine the site classification, the average properties in the top 30 m are to be used. The boreholes at the site were advanced to a maximum depth of 15.7 m. The Site Classification recommendation is based on the available information as well as our interpretation of conditions at and below the boreholes, and based on a review of geological mapping and MECP well records, and our knowledge of the soil conditions in the area.

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

4.4.7 Foundation Wall Backfill and Perimeter Drainage

In general, the existing soils excavated from the building footprint are generally expected to be suitable for re-use as exterior foundation wall backfill. The materials to be re-used as foundation wall backfill should be within three percent of optimum moisture content for best compaction results. If the weather conditions are very wet during construction, site review by the geotechnical consultant may be advised to confirm the suitability of onsite soils for reuse.

It is recommended that heavy compaction equipment be restricted within 0.5 m of the wall. Backfill should be brought up evenly on both sides of the foundation walls which have not been designed to resist lateral earth pressures.

The near-surface soils may be susceptible to frost effects, which can impact hard landscaping adjacent to the building. At locations where the proposed building is expected to have exterior entrances, care should be taken in detailing the exterior slabs and/or sidewalks providing insulation, drainage and non-frost susceptible backfill to maintain flush transitions in cold weather conditions.

For building foundations set above the stabilized groundwater level, 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. The drains should be provided with a frost-free outlet from which the water can be removed. It is anticipated that water could be collected in a sump pit located in the underground parking garage.

Any discharge which is directed into municipal infrastructure will be subject to municipal approvals and/or permitting, and must adhere to sewer discharge by-laws.

4.5 Site Services

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

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

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.

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 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.

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

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

4.6 Pavement Design

The development will be accessed with an internal road network, accessing Komoka Road, as well as the commercial development to the north. The exposed subgrade soils within the roadways are expected to be comprised of re-compacted soils comprised of sand, silt 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 13: Pavement Design Recommendations

Pavement Component for Local Roads	Minimum Thickness		Compaction Requirements
	Main Access Route	Car Parking	
Asphaltic Concrete	50 mm HL 3 60 mm HL 8	40 mm HL 3 50 mm HL 8	97% BRD *(92.0 – 96.5 % MRD)*
Granular ‘A’ Base	150 mm	150 mm	100% SPMDD
Granular ‘B’ Subbase	500 mm	250 mm	100% SPMDD

Other granular configurations may be possible provided the granular base equivalency (GBE) thickness is maintained. These recommendations on thickness design are not intended to support heavy and concentrated construction traffic, particularly where only a portion of the pavement section is installed. If frequent construction traffic is anticipated while only a portion of the site pavements are in place, or if construction is undertaken in poor weather conditions, thickening of the granular subbase may be appropriate and can be reviewed during construction, by the geotechnical consultant.

Where local roads connect to existing pavements, 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 which were encountered near surface in the boreholes advanced at the site, by removing excess subsurface water. Where sandy subgrade soils are present, there may be opportunity to take advantage of the improved subsurface drainage available in those soils, and the need for pavement subdrains may be waived in those locations.

4.7 Curbs and Sidewalks

The concrete for any new exterior curbs and sidewalks should be proportioned, mixed placed and cured in accordance with the requirements of OPSS 353, OPSS 1350 and the Municipality. During cold weather, the freshly placed concrete should be covered with insulating blankets to protect against freezing.

The subgrade for the sidewalks should consist of undisturbed natural soil or well compacted fill. A minimum 100 mm thick layer of compacted (minimum 98 percent SPMDD) Granular 'A' is recommended below sidewalk slabs.

4.8 Construction Monitoring

4.8.1 Sediment and Erosion Control Considerations

Sediment and erosion control measures will be required during construction, particularly around the perimeter of the site, as well as the existing pond on the south half of the site, to contain sediment and prevent discharge towards the neighbouring properties

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 (Table 14, presented on the next page) summarizes general mitigation measures which are suggested as best management practices. 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. 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. The frequency of inspections will depend on weather and site conditions. The following minimum inspection intervals are recommended:

- Before and immediately after rainfall and snowmelt events (timing for inspections before are based on predicted weather forecasts);
- Daily during extended rain or snowmelt periods;
- Daily during any construction activity that would potentially yield significant run-off volumes or otherwise impact the quality of the run-off leaving the site;
- Daily while deficiencies are present which fail to contain, filter or otherwise treat run-off, or contribute to sediment loading in surface water;
- Weekly during dry periods while construction activity is occurring at the site; and,
- Monthly during inactive periods (> 30 days).

Consultation with local approval authorities is recommended to confirm inspection, monitoring, and reporting requirements.

The following table outlines a number of recommended best management practices to help alleviate and prevent uncontrolled sediment release from the site.

Table 14: Best Management Practices for Sediment Containment

Practice / Tasks	During Site Grading	During Site Servicing	During Home Construction & Partial Pavements	Following Construction
Delineate work areas to limit construction activities	✓	✓	✓	
Monitoring of discharge water (for water quality – turbidity) from stormwater run-off and construction dewatering activities.	✓	✓	✓	
Installing perimeter ESC measures such as silt fence and/or silt sock around temporary soil stockpiles, with dedicated points of access clearly marked onsite.	✓	✓		
Use of mud-mats at construction entrance/exit points to help control the amount of loose soil being carried offsite from construction vehicles	✓	✓		
Dedicated fuel storage and equipment fuelling areas.. 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, mulch, or other materials such as bonded fibre mix (BFM).	✓	✓	✓	✓
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.			✓	✓

Removal or decommissioning of ESC measures should not be carried out until site conditions are stabilized, and/or construction is complete.

In accordance with Provincial Regulations, in the event of an uncontrolled sediment discharge offsite, the incident must be reported to the Ontario Spills Action Centre. Reporting requirements include the date and time of the reportable incident, including the source, current status and impact which has been identified. Other pertinent details, such as weather conditions should also be included in the reporting.

4.8.2 Spills Management

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 area, 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.

4.8.3 Inspection and Testing

An effective inspection and testing program is an essential part of construction monitoring. The recommended inspection and testing program should include the following items:

- Subgrade examination prior to engineered fill placement and footing base confirmations for any foundations constructed on engineered fill;
- 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;
- Inspection and testing during road construction, including compaction testing and laboratory testing for pavement components and concrete sampling and testing for curbs and sidewalks;
- Inspection and materials testing for base and surface asphalt;
- Environmental monitoring, including sediment and erosion control inspections.

Sufficient geotechnical inspections and materials testing is recommended for this project, to verify that project specifications have been satisfied during construction.

4.9 Well Decommissioning

The monitoring well installed at the site to document stabilized groundwater conditions will need to be decommissioned in accordance with the requirements of O.Reg. 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.

If the well is to be maintained for monitoring purposes during construction, a site plan showing the well location to be maintained and protected should be provided to the contractors working at the site. Wells which are maintained onsite during construction can be used to assess the impacts of construction dewatering activities, if required. In this regard, they can be equipped with data loggers to monitor changes in water level and the lateral extent of the zone of influence of the construction activities, and/or used to collect water quality samples.

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5. CLOSING

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

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

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

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

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

Respectfully Submitted,

LICENSED PROFESSIONAL ENGINEER
Mar 19/21
R.A. WALKER
PROVINCE OF ONTARIO

Shadden

R. Walker

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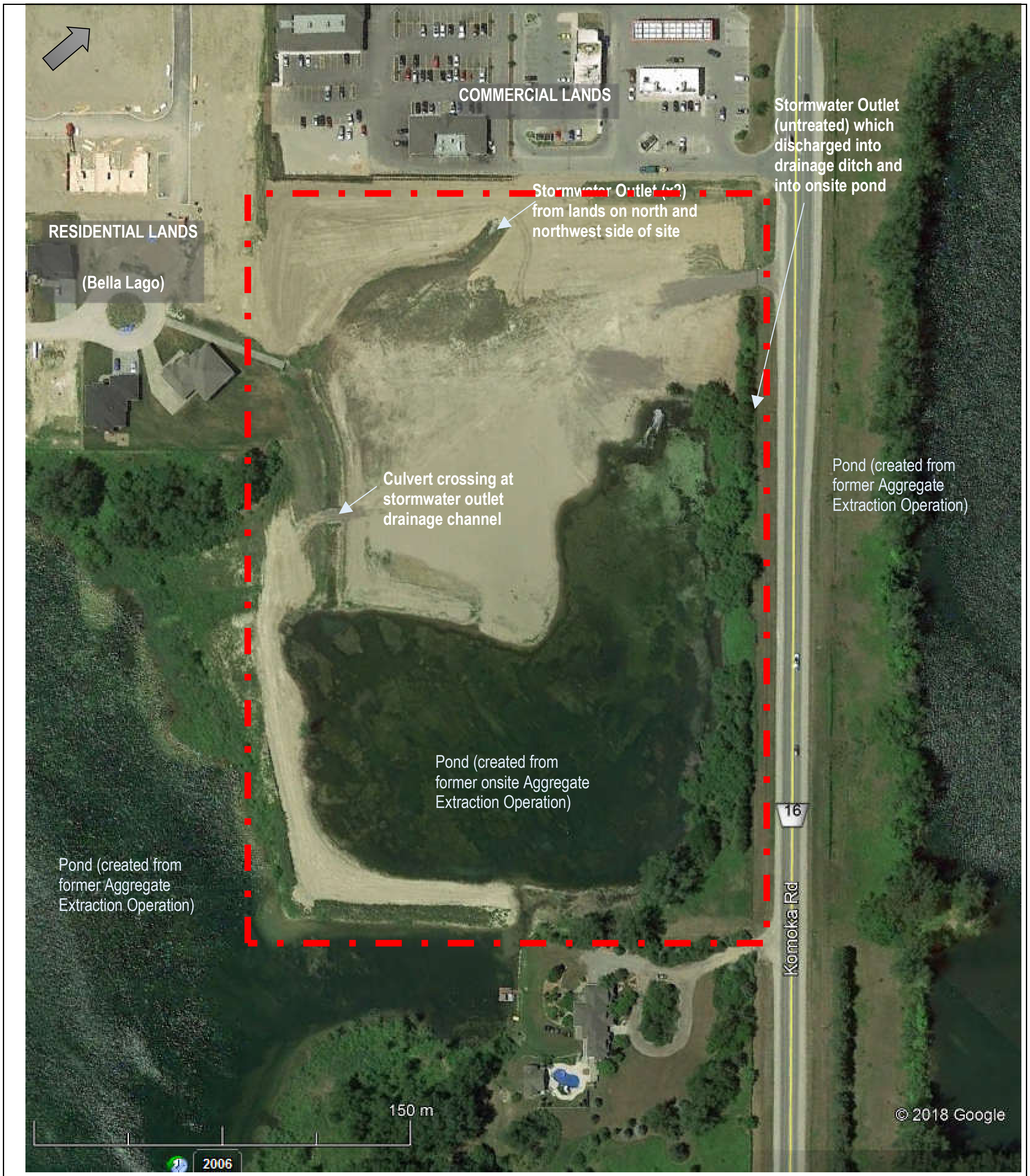
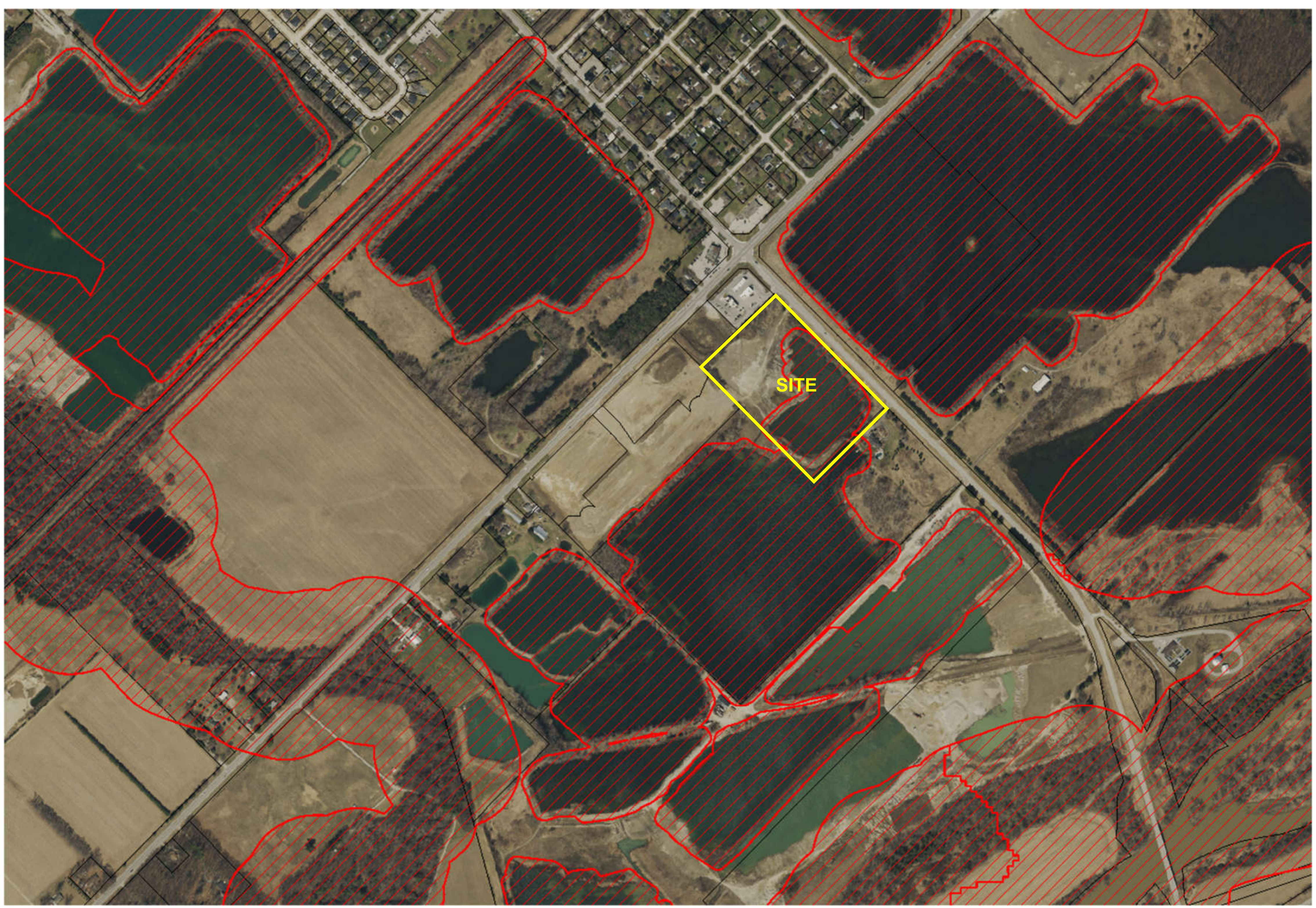
APPENDIX A

DRAWINGS AND NOTES



PROJECT NAME Proposed Residential Development	**PROJECT LOCATION** 22447 Komoka Road, Komoka	**SCALE** As Shown	**PROJECT NO.** GE-00240
LDS	**DRAWING NAME** Site Features	**DATE** May 2019	**DRAWING NO.** 1



LEGEND

▨ UTRCA Regulated Land

— Site Boundary

SOURCE
Upper Thames River Conservation Authority Online Interactive Mapping, March 2021

LDS

PROJECT NAME

Proposed Residential Development

PROJECT LOCATION

22447 Komoka Road, Komoka

DRAWING NAME

UTRCA Regulated Lands

SCALE	**PROJECT NO.**
As Shown	GE-00240
DATE	**DRAWING NO.**
March 2021	2

Latitude: **42.94267** Longitude: **-81.43001**
UTM Zone: **17** Easting: **464917.88**
Northing: **4754538.36**
Upper Tier Municipality: **COUNTY OF MIDDLESEX**
Lower/Single Tier Municipality: **MUNICIPALITY OF MIDDLESEX CENTRE**
Township Concession and Lot: **LOBO CON 1, LOT 4**
Assessment Parcel Address: **22447 KOMOKA RD**
Assessment Roll #: **3939000020010000000**
MECP District: **London**
MECP Region: **Southwestern**

Source Protection Details for Location

Source Protection Area: **Upper Thames River**
Wellhead Protection Area: **No**
Wellhead Protection Area E (GUDI): **No**
Intake Protection Zone: **No**
Issue Contributing Area: **No**
Significant Groundwater Recharge Area: **Yes** ; score is **6**
Highly Vulnerable Aquifer: **Yes** ; score is **6**
Event Based Area: **No**
Wellhead Protection Area Q1: **No**
Wellhead Protection Area Q2: **No**
Intake Protection Zone Q: **No**

Use the Policy search tab to see if any policies apply – for more details see the source protection plan

Information is current as of: **February 4, 2021**

LEGEND

Significant Groundwater Recharge Area

🟩	0
🟩	2
🟩	4
🟨	6
▨	Highly Vulnerable Aquifers

SOURCE

Ministry of Environment, Conservation and Parks, Source Protection Information Atlas, interactive mapping, current to February 4, 2021.

LDS

PROJECT NAME

Proposed Residential Development

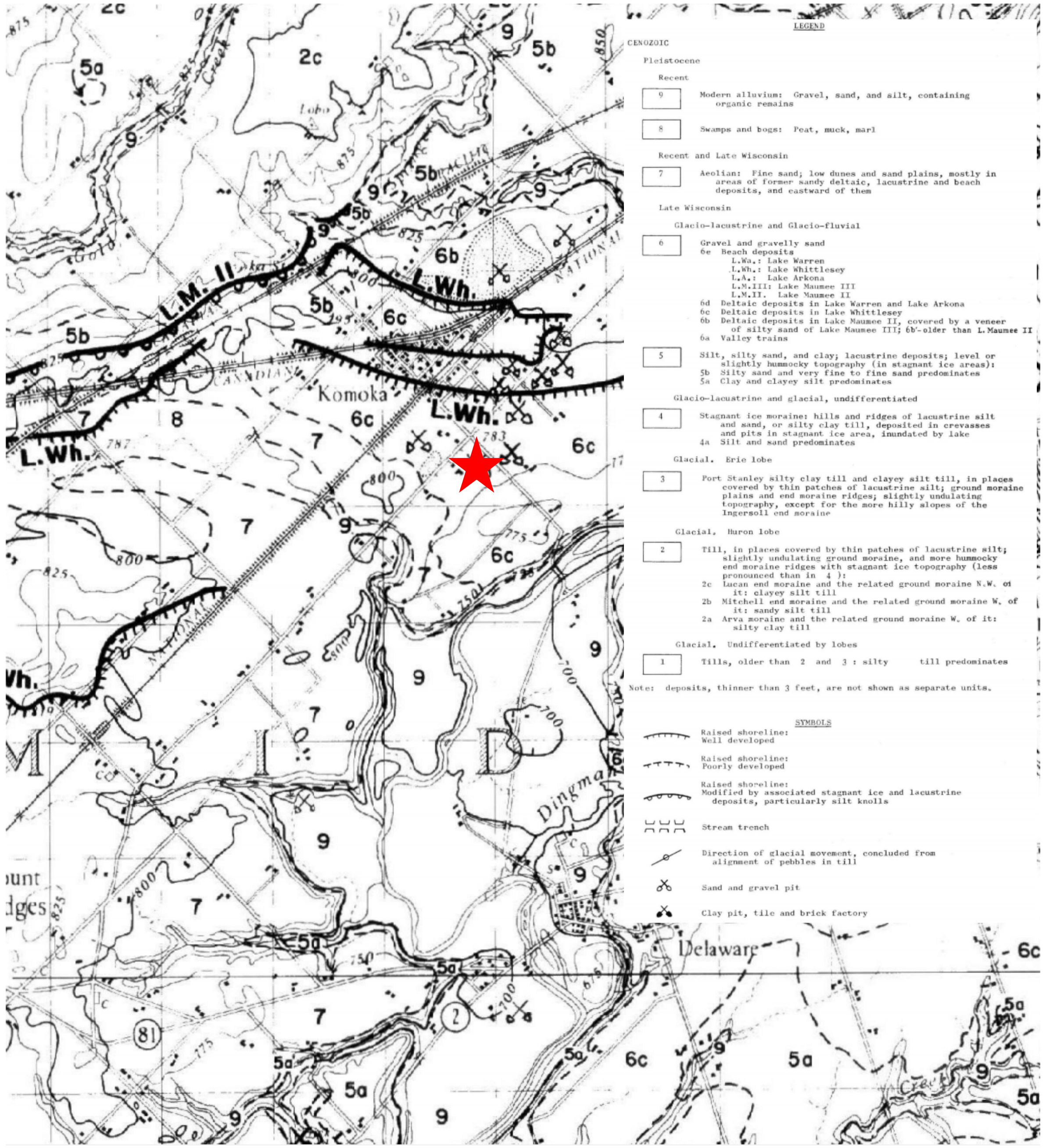
PROJECT LOCATION

22447 Komoka Road, Komoka

DRAWING NAME

Source Water Protection Mapping

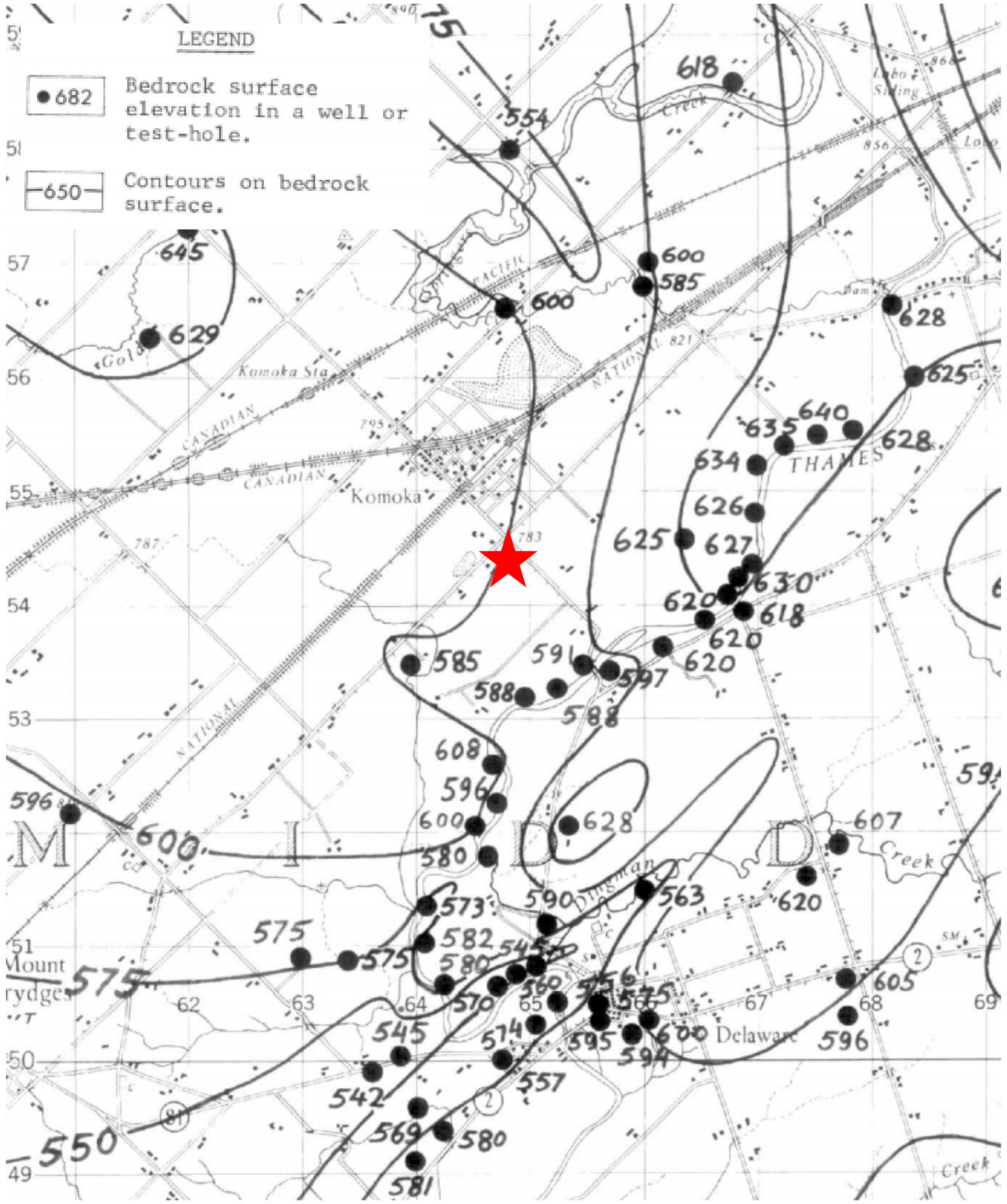
SCALE	**PROJECT NO.**
As Shown	GE-00464
DATE	**DRAWING NO.**
February 2021	**3**



SOURCE

Pleistocene Geology, St. Thomas Area (western half), Ontario Geological Survey Map P0238,

Scale 1:50,00, © 1964



SOURCE

Bedrock Topography, St. Thomas Sheet, Ontario Geological Survey Map P0482,

Scale 1:50,000, © 1968

PROJECT NAME

Proposed Residential Development

PROJECT LOCATION

22447 Komoka Road, Komoka

DRAWING NAME

Geological Mapping

SCALE	**PROJECT NO.**
1:50,000	GE-00240
DATE	**DRAWING NO.**
March 2021	4

SOURCE:
Google Earth Pro, Version 7.3.2.5776,
Coordinates 17T, 464997 m E, 4754544 m N,
Imagery date 7/2/2018

PROJECT NAME
Proposed Residential Development

PROJECT LOCATION
22447 Komoka Road, Komoka

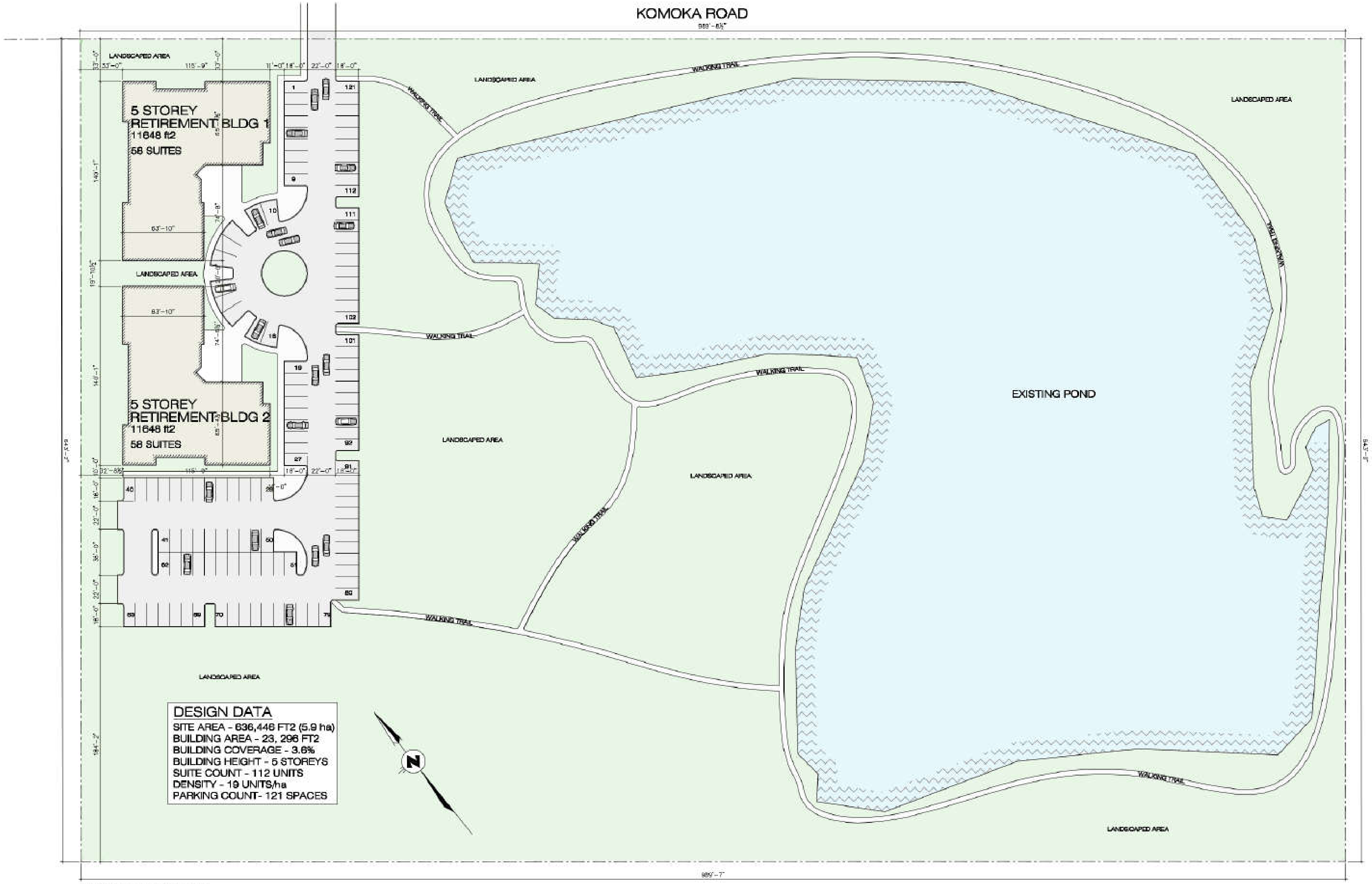
DRAWING NAME
Borehole Location Plan

SCALE
As Shown

PROJECT NO.
GE-00240

DATE
March 2021

DRAWING NO.
5



SITE LAYOUT

SOURCE:

Conceptual Site Layout Plan, prepared by others, October 2018

PROJECT NAME

Proposed Residential Development

PROJECT LOCATION

22447 Komoka Road, Komoka

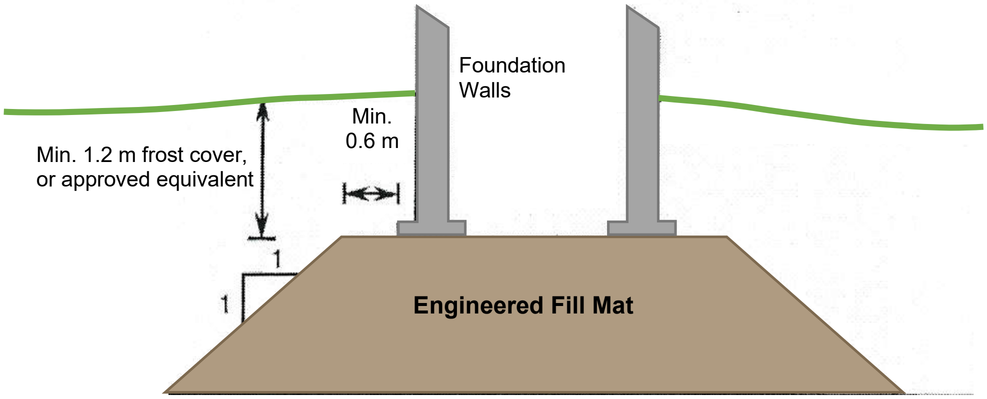
DRAWING NAME

Conceptual Site Layout Plan

SCALE As Shown	**PROJECT NO.** GE-00240
DATE March 2021	**DRAWING NO.** 6

ENGINEERED FILL PLACEMENT

SCHEMATIC DIAGRAM



NOTES:

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

LDS	**PROJECT NAME** Proposed Residential Development	**PROJECT NO.** GE-00240
	PROJECT LOCATION 22447 Komoka Road, Komoka	**DRAWING NO.** 7

APPENDIX B

BOREHOLE LOGS

&

LABORATORY TEST RESULTS

NOTES ON SAMPLE DESCRIPTIONS

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

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

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

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

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

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

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

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

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

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

Project	**Komoka Retirement Building**	Borehole ID
Project Location	**22447 Komoka Road, Komoka**	**1/MW**
Project Number	**GE-00240**	*Sheet 1 of 1*

Date Drilled	**February 18, 2021**	Ground Surface Elevation	**236.48 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	
Drilling Method	**Hollow Stem Auger**	Technician	**S. Hadden, EIT**
Drilling Contractor	**London Soil Test**	Checked By	**R. Walker**

Depth (m)	**Sample Type**	**Sample Number**	**Recovery (%)**	**SPT N-value (blows/0.3 m)**	**Graphic Log**	**Material Description**	**Remarks and Other Tests**
					▨	**SAND AND GRAVEL** - brown, trace to some silt, moist, compact, dense	
0.5					▨		
1.0	◩	1	60	38	▨		*MC - 5.7%*
					▨	- becoming compact and saturated below 1.4 m depth	
1.5					▽ Mar 2/21 WL - 1.72 m		
	◩	2	50	20	▨		
2.0					▨		
2.5	◩	3	50	25	▨		*MC - 16.1%*
3.0					▨		
	◩	4	50	17	▨		
3.5					▨		
4.0					▨		
4.5					▨		
	◩	5	50	23	*4.88 m*		*MC - 20.2%*
5.0					▥	**SILT** - brown, some sand, trace clay, very moist, compact	
5.5					▥	- becoming grey with some clay below 5.6 m depth	
6.0					▥		
6.5	◩	6	90	11	*6.55 m*		
						BH 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	2.74 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

Topsoil previously stripped by contractor

March 2, 2021 - WL, 1.72 m bgs

Project	**Komoka Retirement Building**	Borehole ID	
Project Location	**22447 Komoka Road, Komoka**	**2**	
Project Number	**GE-00240**	*Sheet 1 of 1*	

Date Drilled	**February 18, 2021**	Ground Surface Elevation	**237.63 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**Dry**
Drilling Method	**Hollow Stem Auger**	Technician	**S. Hadden, EIT**
Drilling Contractor	**London Soil Test**	Checked By	**R. Walker**

Depth (m)	**Sample Type**	**Sample Number**	**Recovery (%)**	**SPT N-value (blows/0.3 m)**	**Graphic Log**	**Material Description**	**Remarks and Other Tests**
					▨	**TOPSOIL** - brown, silty loam, 152 mm	
0.5					▨	**FILL**- brown, fine grained sand, trace organics, trace gravel, damp, dense	
1.0	◩	1	60	31	*1.37 m*		
1.5	◩	2	40	66	▨	**SAND AND GRAVEL** - brown, trace silt, damp, very dense	*MC - 2.9%*
2.0					▨	- becoming compact and saturated below 2.1 m depth *Gradation: 3.2% Silt, 70.1% Sand, 26.7% Gravel*	
2.5	◩	3	20	19	▨		
3.0	◩	4	40	24	▨		*MC - 9.4%*
3.5					▨		
4.0					▨		
4.5					▨		
5.0	◩	5	40	21	▨		
5.5					▨		
6.0					*6.25 m*		
6.5	◩	6	90	10	*6.55 m*	**SILT** - grey, some clay, very moist, compact	*MC - 26.2%*
7.0						BH Terminated at 6.55 m Water observed dry at completion Borehole observed open to 2.3 m depth at completion	
7.5							
8.0							

Legend

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

Well Construction Details

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

Additional Notes

MC - denotes moisture content

Particle Size Distribution
Results of Sieve Analysis

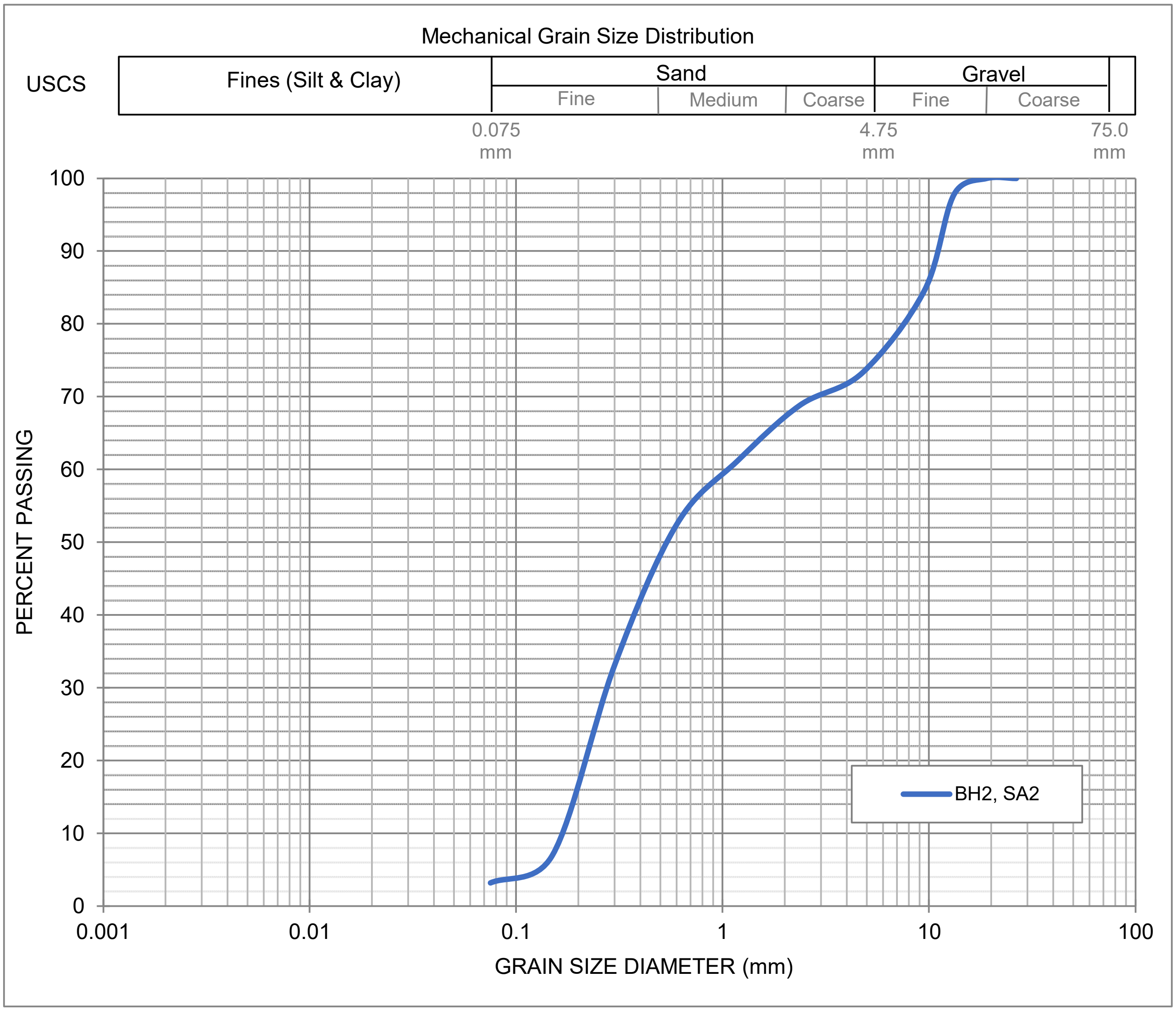
Project Name: Geotechnical Investigation -
Proposed Multi-Storey Development

Date: 8-Mar-21

Project Location: 22447 Komoka Road, Komoka

Project No.: GE-00240

Sample ID	Unified Soil Classification				Moisture Content (%)
	% Silt	% Sand	% Gravel	% Cobbles	
BH2, Sample 2	3.2%	70.1%	26.7%	0.0%	24.4
1.5 m depth					



LDS

Project	**Komoka Retirement Building**	Borehole ID
Project Location	**22447 Komoka Road, Komoka**	**3**
Project Number	**GE-00240**	*Sheet 1 of 1*

Date Drilled	**February 18, 2021**	Ground Surface Elevation	**238.98 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**Dry**
Drilling Method	**Hollow Stem Auger**	Technician	**S. Hadden, EIT**
Drilling Contractor	**London Soil Test**	Checked By	**R. Walker**

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
						TOPSOIL - brown, silty loam, 305 mm	
0.5						**FILL**- dark brown, fine grained sand, trace gravel, damp, loose	
1.0	◪	1	30	6			*MC - 11.2%*
1.5						- becoming compact below 1.4 m depth	
2.0	◪	2	40	15	*2.13 m*		
2.5	◪	3	30	65		**SAND AND GRAVEL** - brown, trace silt, moist, very dense	*MC - 3.6%*
3.0						- becoming compact and very moist below 2.9 m depth	
3.5	◪	4	40	20			
4.0							
4.5							
5.0	◪	5	40	19			*MC - 5.4%*
5.5						- becoming loose and saturated below 5.6 m depth	
6.0							
6.5	◪	6	20	9	*6.55 m*		
7.0						BH Terminated at 6.55 m Water observed dry at completion Borehole observed open to 3.0 m depth at completion	
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	**Komoka Retirement Building**	Borehole ID	**4**
Project Location	**22447 Komoka Road, Komoka**		
Project Number	**GE-00240**		*Sheet 1 of 1*

Date Drilled	**February 18, 2021**	Ground Surface Elevation	**236.89 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**1.5 m bgs**
Drilling Method	**Hollow Stem Auger**	Technician	**S. Hadden, EIT**
Drilling Contractor	**London Soil Test**	Checked By	**R. Walker**

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
						<u>SAND AND GRAVEL</u> - brown, trace to some silt, moist, compact, dense	
0.5							
1.0	◩	1	40	44			
1.5						- becoming compact and very moist below 1.4 m depth	
	◩	2	30	19		*Gradation: 14.0% Silt, 71.8% Sand, 14.2% Gravel*	*MC - 7.6%*
2.0							
2.5	◩	3	40	26			
3.0							
	◩	4	50	26			*MC - 14.3%*
3.5							
4.0							
4.5							
	◩	5	40	14	*4.88 m*		
5.0						**<u>SILT</u>** - brown, trace sand, some clay, very moist, compact	
5.5						- becoming grey below 5.6 m depth	
6.0							
	◩	6	60	14	*6.55 m*		*MC - 20.5%*
6.5							
7.0						BH Terminated at 6.55 m Water measured at 1.5 m depth at completion Borehole observed open to 2.4 m depth at completion	
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

Topsoil previously stripped by contractor

Particle Size Distribution
Results of Sieve Analysis

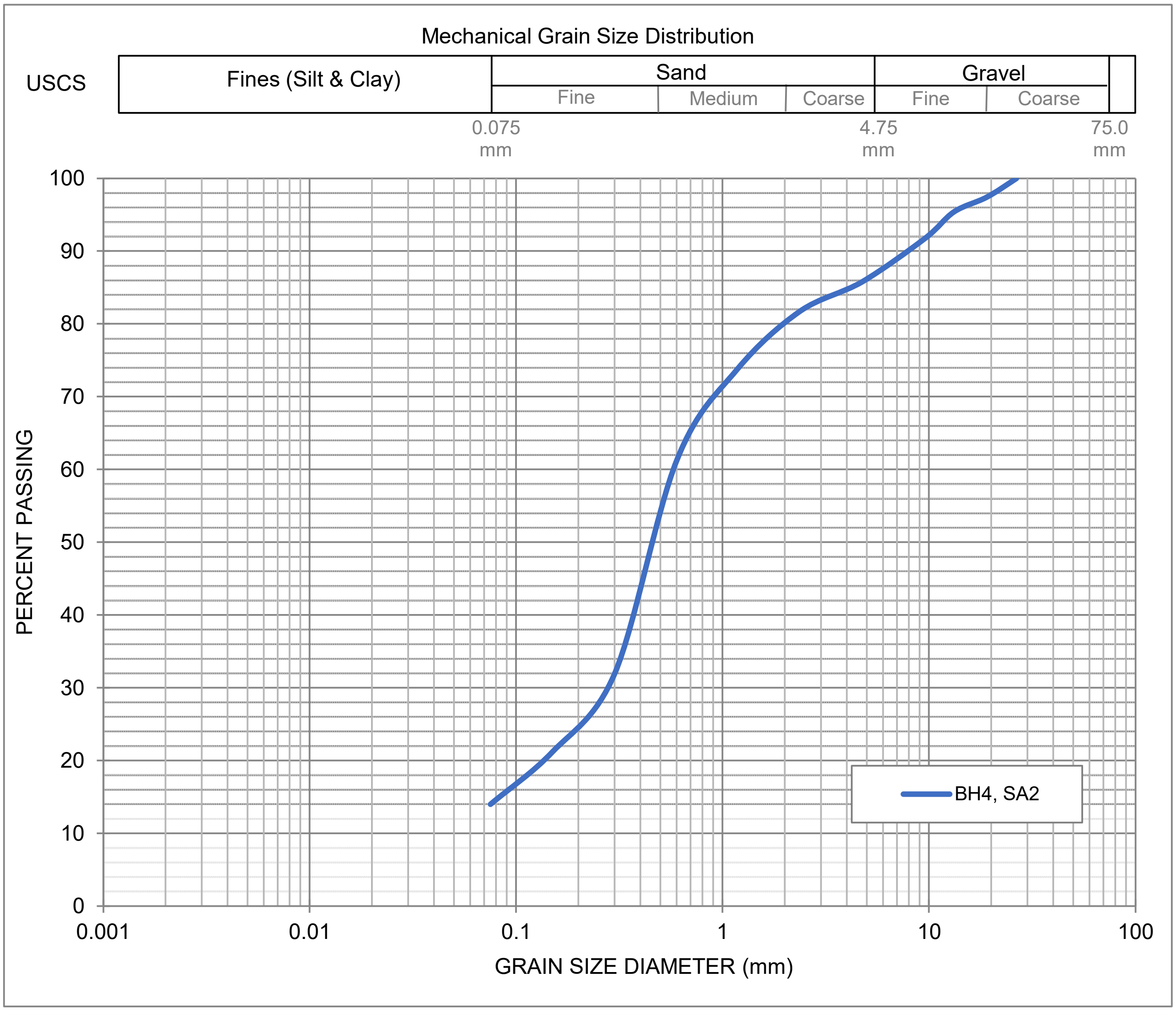
Project Name: Geotechnical Investigation -
Proposed Multi-Storey Development

Date: 8-Mar-21

Project Location: 22447 Komoka Road, Komoka

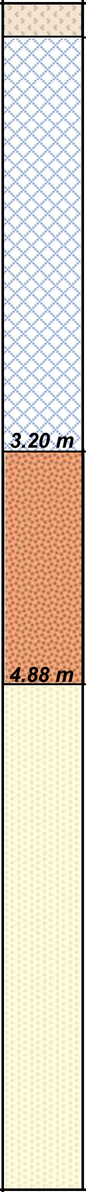
Project No.: GE-00240

Sample ID	Unified Soil Classification				Moisture Content (%)
	% Silt	% Sand	% Gravel	% Cobbles	
BH4, Sample 2	14.0%	71.8%	14.2%	0.0%	24.4
1.5 m depth					



Project	**Komoka Retirement Building**	Borehole ID
Project Location	**22447 Komoka Road, Komoka**	**5**
Project Number	**GE-00240**	*Sheet 1 of 2*

Date Drilled	**February 19, 2021**	Ground Surface Elevation	**236.84 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**2.4 m bgs**
Drilling Method	**Hollow Stem Auger**	Technician	**S. Hadden, EIT**
Drilling Contractor	**London Soil Test**	Checked By	**R. Walker**



Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
						TOPSOIL - brown, silty loam, 254 mm	
0.5						**FILL** - dark brown silt, some sand, trace gravel, moist, loose	
	◩	1	40	5			*MC - 12.1%*
1.0							
1.5							
	◩	2	50	9			
2.0							
						- dark brown silty sand mixed with organics, some gravel, moist, loose at 2.1 m depth	
2.5	◩	3	50	4			*MC - 19.7%*
3.0							
	◩	4	40	11	*3.20 m*		
						SAND AND GRAVEL - brown, trace to some silt, moist, compact, dense	
3.5							
4.0							
4.5							
	◩	5	50	8	*4.88 m*		*MC - 21.8%*
5.0						**SILTY SAND** - brown, fine grained, very moist, loose	
5.5						- becoming grey below 5.6 m depth	
6.0							
	◩	6	50	9			
6.5							
7.0						- becoming compact and saturated below 7.1 m depth	
7.5							
	◩	7	50	26			*MC - 21.0%*
8.0							

continued on the following page

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			Borehole ID
	Project	**Komoka Retirement Building**	**5**
	Project Location	**22447 Komoka Road, Komoka**	
	Project Number	**GE-00240**	*Sheet 2 of 2*

Date Drilled	**February 19, 2021**	Ground Surface Elevation	**236.84 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**2.4 m bgs**
Drilling Method	**Hollow Stem Auger**	Technician	**S. Hadden, EIT**
Drilling Contractor	**London Soil Test**	Checked By	**R. Walker**

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** - grey, trace sand, some clay, very moist, compact	
9.5	◪	8	80	14			
10.0							
10.5					*10.90 m*		
11.0						**SILTY SAND** - grey, fine grained, trace clay, very moist, very dense	
11.5							
12.0							
12.5	◪	9	80	51			*MC - 18.8%*
13.0							
13.5							
14.0						- becoming compact below 13.9 m depth	
14.5							
15.0							
15.5	◪	10	90	23	*15.70 m*		*MC - 15.4%*
16.0						BH Terminated at 15.70 m Water measured at 2.4 m depth at completion Borehole observed open to 2.4 m depth at completion	

Legend

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

Well Construction Details

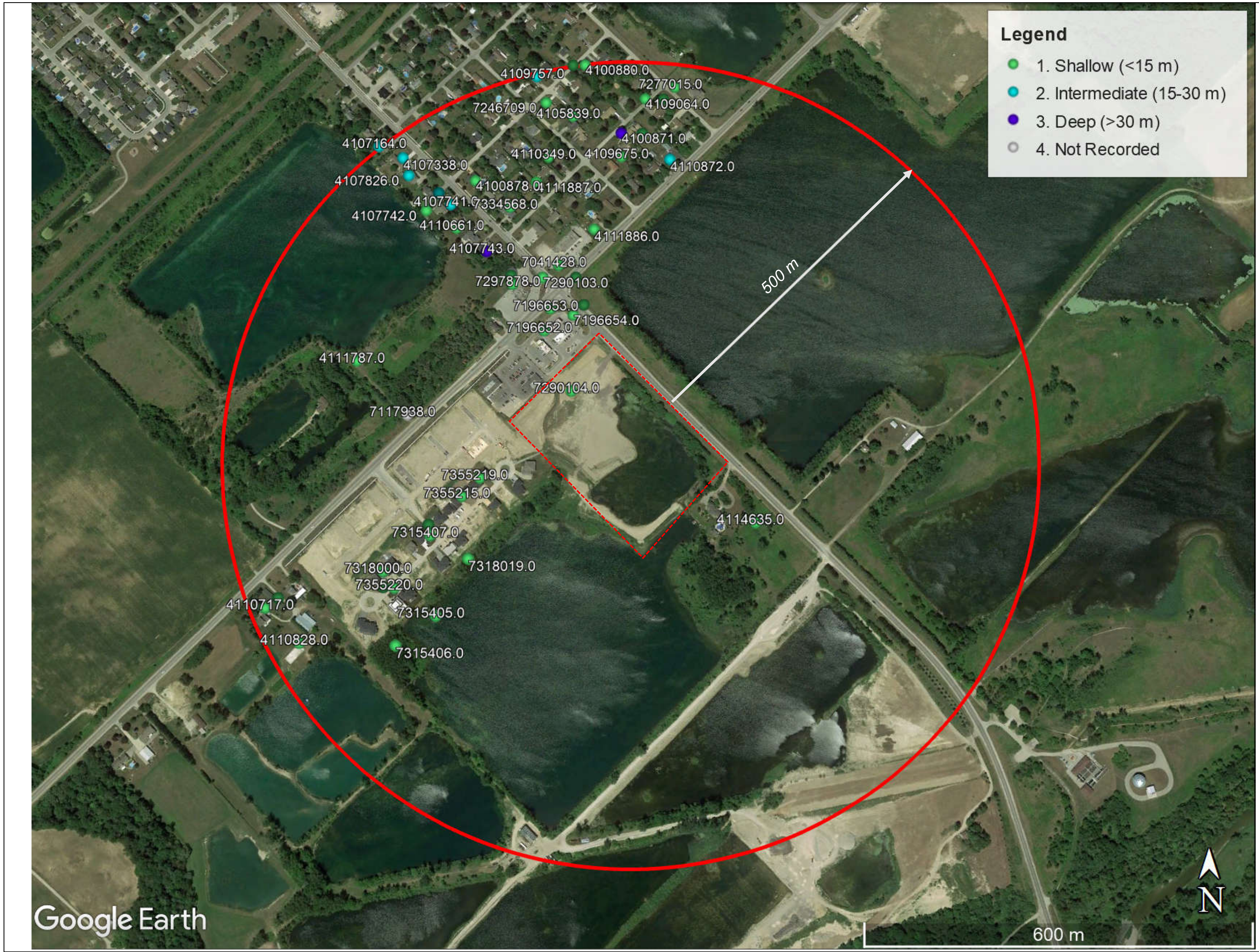
Pipe Diameter	**no well installed**
Installation Depth	
Screen Length	
Depth of Bentonite Seal	

Additional Notes

MC - denotes moisture content

APPENDIX C

MECP WELL RECORD SUMMARY



SOURCE:
MECP Well Records: www.ontario.ca/environment-and-energy/map-well-records, updated January 24, 2020

PROJECT NAME

Proposed Residential Development

PROJECT LOCATION

22447 Komoka Road, Komoka

DRAWING NAME

MECP Well Locations – All Wells

SCALE	**PROJECT NO.**
As Shown	GE-00240
DATE	**DRAWING NO.**
March 2021	C1

MECP Water Supply Wells

MECP Well ID	Registration Year	Well Type	Depth of Well (m)	Depth Water Found (m)	Static Water Level (m)	Pump Rate (L/min)
4100870	04/04/1967	Domestic	5.2	3.0	3.0	19.0
4100878	11/07/1967	Domestic	4.6	3.0	3.0	19.0
4100880	11/09/1967	Domestic	4.6	2.4	2.4	15.2
4105839	04/17/1972	Domestic	4.9	3.7	3.7	22.8
4107164	02/21/1975	Domestic	19.5	16.5	4.6	38.0
4107338	08/07/1975	Domestic	16.8	15.2	4.6	30.4
4107397	09/24/1975	Domestic	19.8	16.8	6.1	30.4
4107435	10/10/1975	Domestic	9.4	4.6	4.6	38.0
4107742	07/16/1976	Domestic	9.1	3.7	3.0	30.4
4107743	05/12/1976	Domestic	39.6	4.3	2.4	38.0
4109430	12/15/1980	Domestic	9.1	3.0	2.4	68.4
4109757	10/05/1982	Domestic	16.5	15.8	6.7	38.0
4110349	08/09/1985	Domestic	7.9	3.7	3.4	45.6
4110661	04/03/1986	Domestic	9.1	3.0	3.0	76.0
4110717	01/23/1986	Commercial	9.1	6.4	3.0	190.0
4110828	04/13/1987	Commercial	7.0	3.0	3.0	475.0
4110872	07/01/1984	Domestic	15.5	7.3	4.0	26.6
4111133	10/30/1987	Domestic	5.2	4.6	4.6	30.4
4111787	07/12/1989	Domestic	8.5	2.4	2.4	95.0
4111886	04/28/1989	Commercial	8.8	4.3	2.4	95.0
4111887	01/03/1989	Domestic	8.8	3.0	2.7	76.0
4114635	08/30/2000	Domestic	7.9	2.4	2.7	45.6
7315405	06/20/2018	Irrigation	7.8	*NR*	4.9	38.0
7315406	06/11/2018	Irrigation	5.2	*NR*	2.1	38.0
7315407	06/27/2018	Irrigation	6.2	*NR*	4.7	30.4
7318000	08/24/2018	Irrigation	7.2	*NR*	4.9	38.0
7318002	08/22/2018	Irrigation	7.2	*NR*	4.9	38.0
7318003	08/14/2018	Irrigation	6.1	*NR*	4.7	34.2
7318019	08/14/2018	Irrigation	5.9	*NR*	4.7	38.0
7334568	04/24/2019	Irrigation	5.8	*NR*	2.7	38.0
7355215	10/11/2018	Irrigation	6.2	*NR*	4.9	22.8
7355219	08/27/2018	Irrigation	6.6	*NR*	5.2	34.2
7355220	08/29/2018	Irrigation	6.9	*NR*	4.9	38.0

NR: Not recorded

MECP Observation Wells

Well	Registration Year	Well Use	Depth of Well, m	Depth Water Found, m	Static Water Level, m	Pump Rate, lpm
7041428	01/25/2007	Observation Wells	4.6	2.8	*NR*	*NR*
7196652	09/18/2012	Observation Wells	4.9	3.7	*NR*	*NR*
7196653	09/18/2012	Observation Wells	6.1	3.8	*NR*	*NR*
7196654	09/18/2012	Observation Wells	6.1	3.5	*NR*	*NR*
7277015	11/07/2016	Observation Wells	3.8	*NR*	*NR*	*NR*
7290102	05/24/2017	Observation Wells	4.6	*NR*	*NR*	*NR*
7290103	05/24/2017	Observation Wells	4.6	*NR*	*NR*	*NR*
7290104	05/24/2017	Observation Wells	4.6	*NR*	*NR*	*NR*
7304811	12/08/2017	Observation Wells	4.6	3.7	*NR*	*NR*
NR: Not Recorded						

MECP Test Holes and Abandonment Records

Well	Registration Year	Well Use	Depth of Well, m	Depth Water Found, m	Static Water Level, m	Pump Rate, lpm
4100871	04/05/1967	Monitoring & Test Hole	31.1	31.1	3.0	38.0
4107741	07/15/1976	Abandoned-Supply	24.4	*NR*	*NR*	*NR*
4107826	11/15/1976	Monitoring and Test Hole	19.5	18.0	4.3	30.4
4108139	08/01/1977	Monitoring and Test Hole	10.1	3.7	*NR*	1900.0
4109064	08/28/1979	Monitoring and Test Hole	6.4	3.7	3.7	15.2
4109675	04/05/1982	Monitoring and Test Hole	11.0	5.5	6.1	38.0
7117938	10/10/2008	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
7297878	05/24/2017	Monitoring and Test Hole	4.6	*NR*	*NR*	*NR*
7334501	04/11/2019	Abandoned-Other	*NR*	*NR*	*NR*	*NR*
NR: Not Recorded						

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