GEOTECHNICAL INVESTIGATION

PROPOSED TOWNHOUSE DEVELOPMENT
6 & 10 ELMHURST ST, KILWORTH, ONTARIO

LDS PROJECT NO. GE-00285

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Submitted to:

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1.0 INTRODUCTION

LDS Consultants Inc. (LDS) has been retained by Mr. Mohammad Abou Sweid to conduct a Geotechnical Assessment for a proposed townhouse development. The subject site is located south of the intersection of Glendon Drive and Elmhurst Street in Kilworth, municipal numbers (MN) 6 & 10 Elmhurst Street. A Key Plan showing the site location is provided on Figure 1, below.

Figure 1: Key Plan

It is understood that a townhouse development is proposed for the site, which will be accessed from Elmhurst Street. Area grading work as well as the removal of existing residential structures on site is anticipated to prepare the site for the proposed development. A concept Plan is provided on Drawing 1, appended.

The scope of work for the Geotechnical Investigation was outlined in LDS' proposal (reference G2019-046, dated September 9, 2019). Authorization to carry out this work was received from Mr. Mohammad Abou Sweid, on September 11, 2019.

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 6 & 10 Elmhurst Street, Kilworth Ontario.

This report provides a summary of the borehole findings (documenting soil and groundwater conditions at the site). The report also includes geotechnical and preliminary hydrogeological comments and recommendations for the proposed townhouse development, including: site preparation (including demolition and restoration of the former building area, and the re-use of excavated materials), excavations and excavation support, groundwater control, foundation design (including soil bearing capacity, subgrade preparation, and allowable settlements), design and construction of building foundations (including basement or slab-on-grade construction, foundation drainage, and backfilling), site servicing (including bedding and trench backfill recommendations), stormwater management considerations and pavement design.

This report is provided on the basis of the terms noted above, and on the assumption that the design will follow applicable codes and standards. The site investigation and recommendations provided in this report follow generally accepted practice for geotechnical consultants in Ontario.

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

1.2 Site Description

The subject site sits on approximately 3.1 acres of land at MN 6 & 10 Elmhurst Street, located southwest of the intersection at Glendon Drive and Elmhurst Street in Kilworth, Ontario. The lands are currently occupied by residences which front on Elmhurst Street. The rears of the lots remain vacant with a mixture of vegetation and small to medium sized trees.

Site topography is fairly flat throughout, and no surface water features are present within the site limits.

1.3 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

A portion of the site (along the north property boundary) is located within the UTRCA lands, as shown on Drawing 2, appended. Property owners must obtain permission from UTRCA before beginning any development, site alteration, construction, or placement of fill within the regulated area. Proposed development within the study area will be subject to the above referenced Regulation. Consultation with the local Conservation Authority for review of site-specific development plans is recommended in this regard.

2.0 INVESTIGATION PROGRAM

2.1 Field Program

LDS carried out a field program consisting of a series of boreholes on September 18, 2019. The boreholes were advanced at the site by a local drilling-contractor, using a track-mounted drill-rig. Six boreholes (denoted as BH1 through BH6) were advanced throughout the site and excavated to a maximum depth of 5.0 m (16.5 ft) below existing grade.

Ground surface elevations at the borehole locations were surveyed by LDS using a Trimble R10 GPS rover. The location of the boreholes are summarized below, and illustrated on Drawing 3, in Appendix A.

Location	Northing, m N	Easting, m E	Ground Surface Elevation (m asl)
BH1	4756567.99	467655.28	239.57
BH2	4756590.10	467682.55	239.54
BH3	4756636.18	467712.32	240.36
BH4	4756538.73	467685.16	240.08
BH5	4756558.35	467706.43	239.99
BH6	4756592.01	467735.19	240.24

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

Select samples were collected from the boreholes for further review and laboratory testing. Two grain size analyses were carried out on select samples of the predominant subgrade soils. Routine moisture content determinations were also carried out on select samples from each borehole. The fieldwork was supervised by members of LDS' technical staff. All samples recovered from the site were returned to LDS for detailed examination and selective testing. Collected samples will be disposed of, following the issuance of the Geotechnical Report, unless prior arrangements have been made for longer term storage.

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

3.0 SUMMARIZED CONDITIONS

3.1 Review of Available Mapping

Select geological mapping and publications were reviewed for the purposes of reviewing regional characteristics for soil conditions in the Kilworth area. Findings are summarized below, for reference. Local well records generally indicate that limestone bedrock is located below approximately 55 m of overburden soils. Bedrock was not encountered during the fieldwork for this investigation.

Source Mapping	Summarized Findings
Quaternary Geology mapping for the London area (Ontario Division of Mines, Quaternary Geology Lucan Area, Scale 1:50,000, Preliminary Map P1048, 1975).	The Quaternary Geological survey mapping indicates that the site is comprised of a glaciomarine nearshore deposits (characterized by sand, gravelly sand and gravel.)
Physiographic mapping for Southwestern Ontario (Chapman, L.J. and Putnam, D.F. 2007. Physiography of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 228).	The Physiographic mapping identifies that the site is located within the central part of the Physiographic Region known as the Cardoc Sand Plains and London Annex. The mapping indicates that the subgrade soils in the area generally consist of coarse-textured glaciolacustrine deposits. These soils are expected to be predominantly comprised of sand and gravel, with minor silt and clay.
Bedrock Geology of Ontario. Ontario Geological Survey, Miscellaneous Release Data 126, 1:250 000 scale, Revised 2006.	The map reveals that the 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.

3.2 Borehole Findings

A series of six boreholes were advanced at the site to examine soil and shallow groundwater conditions. The borehole locations are shown on Drawing 3, appended.

In general, soils observed in the boreholes consisted of topsoil overlying sand and silt till soils. General descriptions of subsurface conditions are summarized in the following sections. Borehole logs are provided in Appendix B, for reference.

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

Topsoil

Each borehole surfaced with a layer of topsoil. The topsoil consisted of brown sandy loam, with a thickness of 75 mm generally noted 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.

Sand

Sand was encountered underlying the topsoil in each borehole and, with the exception of borehole BH1, each borehole terminated within this later. In borehole BH1, the sand layer had a thickness of 3.8 m. The sand was generally described as being brown in color, with a fine- to medium-grained texture, containing trace to some gravel. The sand is in a variable loose to compact state, based on Standard Penetration Test (SPT) N-values in the range of 6 to 29 blows per 0.3 m of split-spoon sampler penetration.

Moisture content determinations conducted on recovered samples of the sand generally range between 2.4 to 10.6 percent, generally indicative of damp to moist soil conditions above the stabilized groundwater elevation. Two samples of the sand were submitted for gradation analyses, and the following table shows the grain size distribution. The results are also shown graphically in Appendix B.

Sample ID	Unified Soil Classification			
	Fines (Silt & Clay)	% Sand	% Gravel	% Cobbles
BH1 SA3 - 2.3 m depth	12.9%	72.2%	14.9%	0.0%
BH6 SA3 - 2.3 m depth	7.8%	86.9%	5.3%	0.0%

Silt Till

Underlying the sand, a layer of silt till was encountered in borehole BH1 at a depth of 4.0 m. The silt till was generally described as grey in colour, containing trace sand, and trace to some fine gravel. The silt till is generally noted to be in a dense state, based on a SPT N-value of 34 blows per 0.3 m penetration of the split-spoon sampler.

A moisture content determination conducted on a sample of the till yielded a value of 7.4 percent, generally indicative of damp soil conditions.

3.3 Soil Permeability

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

The natural subgrade soils encountered at the site are generally comprised of sand and silt till. Based on the gradation results (presented in Section 3.2) for the sand soils encountered at the site, the following values for saturated hydraulic conductivity have been calculated for the collected samples. 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).

Sample ID	% Fines (Silt & Clay)	% Sand	% Gravel	Saturated Hydraulic Conductivity (m/sec)
BH1SA3 – 2.3 m depth	11.9	73.2	14.9	1.60×10^{-5}
BH6SA3 – 2.3 m depth	7.8	86.9	5.3	1.44×10^{-4}

3.4 Shallow Groundwater Conditions

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

Borehole	Ground Surface Elevation, m asl	Groundwater Observations	Groundwater Elevation, m asl
BH1	239.57	Dry	- -
BH2	239.54	Water measured at 4.27 m	235.27
BH3	240.36	Dry	- -
BH4	240.08	Dry	- -
BH5	239.99	Dry	- -
BH6	240.24	Dry	- -

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 in wet seasons.

3.5 MECP Well Record Review

A review of local well records available through the Ministry of Environment, Conservation, and Parks (MECP) for this area was carried out to review the water levels recorded in the nearby wells. Drawing 4 in Appendix A 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 overburden aquifer at depths ranging from 3.0 to 9.1 m depth. Static water levels in water supply wells are generally reported at depths ranging from 0.6 m to 6.4 m. The remainder of the water supply wells are set into the deeper limestone bedrock aquifer.

The remaining well records are recorded as test holes or well abandonment records, and are recorded at variable depths within the well records. Given the commercial and residential development in the area, it is anticipated that a number of the wells that are identified as water supply wells are no longer in use.

4.0 GEOTECHNICAL COMMENTS AND RECOMMENDATIONS

It is understood that a townhouse is being proposed for the site, which will be accessed from Elmhurst Street, and will be serviced by the extension of municipal services from Parkland Place or Kilworth Park Drive, (see Drawing 1, appended.)

The boreholes generally revealed a layer of surficial topsoil which is underlain by a layer of sand, which is in turn underlain by silt till soils. Groundwater was encountered below 4.2 m depth (Elevation 235.2 m asl.)

This report provides a summary of the borehole findings (documenting soil and groundwater conditions at the site). The report also includes geotechnical and hydrogeological comments and recommendations for the proposed townhouse development, including: site preparation (including demolition and restoration of the former building area, and the re-use of excavated materials as engineering fill or structural fill), excavations and excavation support, groundwater control, foundation design (including soil bearing capacity, subgrade preparation, and allowable settlements), design and construction of building foundations (including basement or slab-on-grade construction, foundation drainage, and backfilling), site servicing (including bedding and trench backfill recommendations), onsite stormwater management considerations, 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. Vegetation removal and topsoil stripping is anticipated throughout the area to be developed. In general, this is expected to require the removal of about 75 mm of surficial topsoil. Thicker topsoil areas may also be present in proximity to existing wooded areas, and where local depressions are present at the site.

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

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

In areas which engineered fill is to be placed to raise grades, the exposed subgrade soils should be reviewed approved by the geotechnical consultant following topsoil stripping. In accordance with the

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

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

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

4.1.2 Restoration of Former Building Areas

Prior to demolition of the existing residential structures, a Designated Substance Survey (DSS) should be conducted. The need for a DSS is outlined in Section 30 of the Ontario Occupational Health and Safety Act, which specifies that designated substances (regulated under O.Reg. 490/09) and other potentially hazardous building materials must be identified prior to demolition work that may disturb such materials. LDS can assist with developing a scope of work for this work, when tenants and their belongings are no longer occupying the buildings.

Building demolition should include the removal of exiting footings, and septic system (tanks, field tile and tile bedding), if present. Following the demolition of the existing residential building and structures, a site review should be carried out to confirm that building foundations, concrete slabs, building debris and remnant site services are removed from the site. In the event that a water supply well is encountered, it should be decommissioned in accordance with the Regulations outlined in O.Reg. 903. This same regulation applies to the decommissioning of monitoring wells, when they are no longer required.

4.2 Excavations and Groundwater Control

Excavations for the proposed buildings and site services are generally expected to extend through the topsoil, and will terminate within the natural subgrade soils or engineered fill material.

All work associated with design and construction relative to excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). Based on the results of the geotechnical investigation and in accordance with Section 226 of Ontario Regulation 213/91, the sand encountered near ground surface is generally classified as Type 3 soil. 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.

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

4.2.1 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 sand and silt till 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 specialist shoring contractor should review the geotechnical information provided in this report. The shoring system must be designed to be internally (overturning, and sliding) and externally stable (slope stability / base heave).

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:

Soil	Angle of Internal Friction, φ	Bulk Unit Weight of Soil, γ (kN/m^3)	Active earth pressure coefficient, Ka	At-rest earth pressure coefficient, Ko	Passive east pressure coefficient, Kp
Compact Sand	30	20.0	0.33	0.47	3.05
Silt Till	28	20.5	0.30	0.50	2.38

In the event that soil conditions near the excavation vary materially from the above soils, the geotechnical consultant should review the soil conditions to confirm the design parameters. A prefabricated trench box may be used for servicing excavations (if required), provided that it is designed (by a professional engineer) to withstand the soil and hydrostatic loading (if applicable).

4.2.2 Groundwater Control

Conventional groundwater control methods are generally expected to be suitable for shallow excavations (less than 4 m deep) 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 undisturbed sand are expected to be in the range of 1.4 $x10^{-4}$ to 1.6 $x10^{-5}$ m/s, based on laboratory testing (presented in Section 4.4 below). This information is provided to assist with determining appropriate construction dewatering methods.

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

Consideration should be given to carrying out a series of pre-tender test pits for contractors to obtain a better appreciation of the behavior of excavations and to confirm dewatering requirements. 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 submission to the Environmental Activity and Sector Registry (EASR) will be required, and a Permit to Take Water (PTTW) will be required for volumes in excess of 400,000 litres per day. PTTW applications are submitted to and approved by MECP according to Sections 34 and 98 of the Ontario Water Resources Act R.S.O. 1990 and Water Taking and Transfer Regulation O. Reg. 387/04. The need for an EASR submission or PTTW should be reviewed when design depths for the building foundations and site servicing have been verified.

4.3 Building Components

4.3.1 Foundation Design

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

- Serviceability Limit States (SLS) 145 kPa (~3000 psf)
- Ultimate Limit States (ULS) 190 kPa (~4000 psf)

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

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

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

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

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

4.3.2 Slab-on-Grade Floors

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

Around the perimeter of the buildings the ground surface should be sloped on a positive grade away from the structure to promote surface water run-off and reduce groundwater infiltration adjacent to the foundations. Perimeter drains around the foundation may not be required if the floor slab is set at least 300 mm above the exterior grade and the grade is sloped away from the structure.

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 40 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.

It is recommended that the water-cement ratio and slump of concrete used for the 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.

4.3.3 Basement Construction

Basement floors can be constructed using cast slab-on-grade techniques (noted above) provided that the subgrade is stripped of unsuitable material. It is recommended that a minimum 200 mm (8 inch) thick compacted layer of 19 mm (¾ inch) clear stone be placed between the prepared subgrade and the floor slab to serve as a moisture barrier.

For basement foundations which are above the stabilized groundwater level, or where only minor groundwater seepage is observed from the excavation sidewalls, the portion of exterior basement wall 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

Foundations should be provided with damp-proofing and foundation drainage tiles, in accordance with standard Ontario Building Code (OBC) requirements.

In general, the excavated soils from the building footprints, which are free of topsoil and organics are generally expected to be suitable for re-use as foundation wall backfill. Some soil conditioning may be required in wet or winter conditions.

4.3.4 Foundation Wall Backfill

In general, the existing natural subgrade soils from the foundation areas are generally expected to be suitable for re-use as 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.

In the event that excavated materials contain topsoil, organics or otherwise unsuitable material, such materials should be stockpiled separately, and limited to re-use where settlements can be tolerated.

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.

4.4 LID Considerations

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

From a quantitative standpoint, incorporating effective at-source infiltration structures into the stormwater management design and as part of a storm water management strategy is primarily dependent on (but not limited to), native soil infiltration rates and depth to seasonal high groundwater table. Based on the gradation results (presented in Section 3.2) for the sand soils encountered at the site, the following values for saturated hydraulic conductivity and infiltration rates have been calculated for the collected samples:

Sample ID	Sample Composition			Parameter		
	% Fines (Silt & Clay)	% Sand	% Gravel	D_{10} (mm)	Saturated Hydraulic Conductivity[1] m/sec	Factored Infiltration Rate[3] mm/hr
BH1SA3 – 2.3 m depth	11.9	73.2	14.9	0.04	1.60×10^{-5}	39
BH6SA3 – 2.3 m depth	7.8	86.9	5.3	0.12	1.44×10^{-4}	69

Notes
1. Determined using Hazen Correlation of the D10 (diameter at which 10% of the sample passes)
2. 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.
3. Factor of Safety of 2.5 has been applied, in accordance with TRCA/CVC Low Impact Development Stormwater Management Planning and Design Guide protocol,

Due to the thickness of the sand near surface, shallow lateral infiltration structures such as infiltration galleries or bioretention structures may be considered at the site. Deeper infiltration structures, such as drywells set in the natural sand may also be considered for the site. The use of grassed swales and reduced lot grading can provide benefits in greenspace areas, to extend the amount of time that stormwater is detained on the surface, helping to moderate run-off and provide additional infiltration and evapotranspiration opportunities.

Where site grading activities are planned for the proposed development, onsite review of any materials imported to the site for use is recommended to identify if fill placement can be done to support possible infiltration methods, and to predict the performance of the proposed infiltration structures.

4.5 Site Services

Subgrade soils beneath new services are generally expected to consist of sand or silt till 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.

4.5.1 Pipe Bedding

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.

Consideration may be given to installation of strategically placed seepage (clay or concrete) collars, particularly along any deep sanitary sewer to minimize flow along the pipe bedding material, if servicing depths extend down into the underlying silt till soils. Otherwise, the use of seepage collars in granular soils is not considered to be effective. The use, location and frequency of collars use can be best assessed during the early stage of construction, by a geotechnical engineer. LDS can assist with technical recommendations regarding clay collar construction, configuration and location, during construction.

4.5.2 Trench Backfill

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 general consist of sand. 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

Areas to be paved should be stripped of any obviously unsuitable or unstable material to design subgrade level. The exposed subgrade must then 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. This work should be completed under the supervision of the geotechnical consultant. In general terms, compacted soils supporting site pavements should be compacted to a minimum level of 98 percent SPMDD.

Provided that the preceding recommendations are followed, pavement thickness design requirements given in the following table are recommended for the anticipated subgrade conditions and traffic loading.

Pavement Component	Minimum Design Thicknesses
	Local Road
Asphaltic Concrete	35 mm HL 3 45 mm HL 8
Granular A Base	150 mm
Granular B Subbase	300 mm

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

Samples of both Granular 'A' and Granular 'B' aggregates should be checked for conformance to OPSS 1010 prior to use on site, and during construction. Granular 'B' subbase and Granular 'A' base courses must be compacted to 100 percent SPMDD.

Asphaltic concrete paving materials should conform to requirements of OPSS 1150. Asphalt should be placed in accordance with OPSS 310 and compacted to a minimum 97 percent of the bulk relative density (BRD) as shown in the above table. Alternatively, a target compaction level of 92.0 to 96.5 percent of the

Marshall Mix design maximum relative density (MRD) may also be an appropriate measure for asphalt compaction

If frequent construction traffic is anticipated while only a portion of 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.

Good drainage provisions will optimize long term pavement performance. The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum grade of two percent) to provide effective surface drainage. The requirement for pavement subdrains will depend on the type of material used to reach design grades along the site roads. Consideration may be given to incorporating short stub drains at catch basin locations.

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.

5.0 CLOSING

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

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

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

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

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

Respectfully Submitted,

LDS CONSULTANTS INC.

Prepared By:

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Reviewed By:

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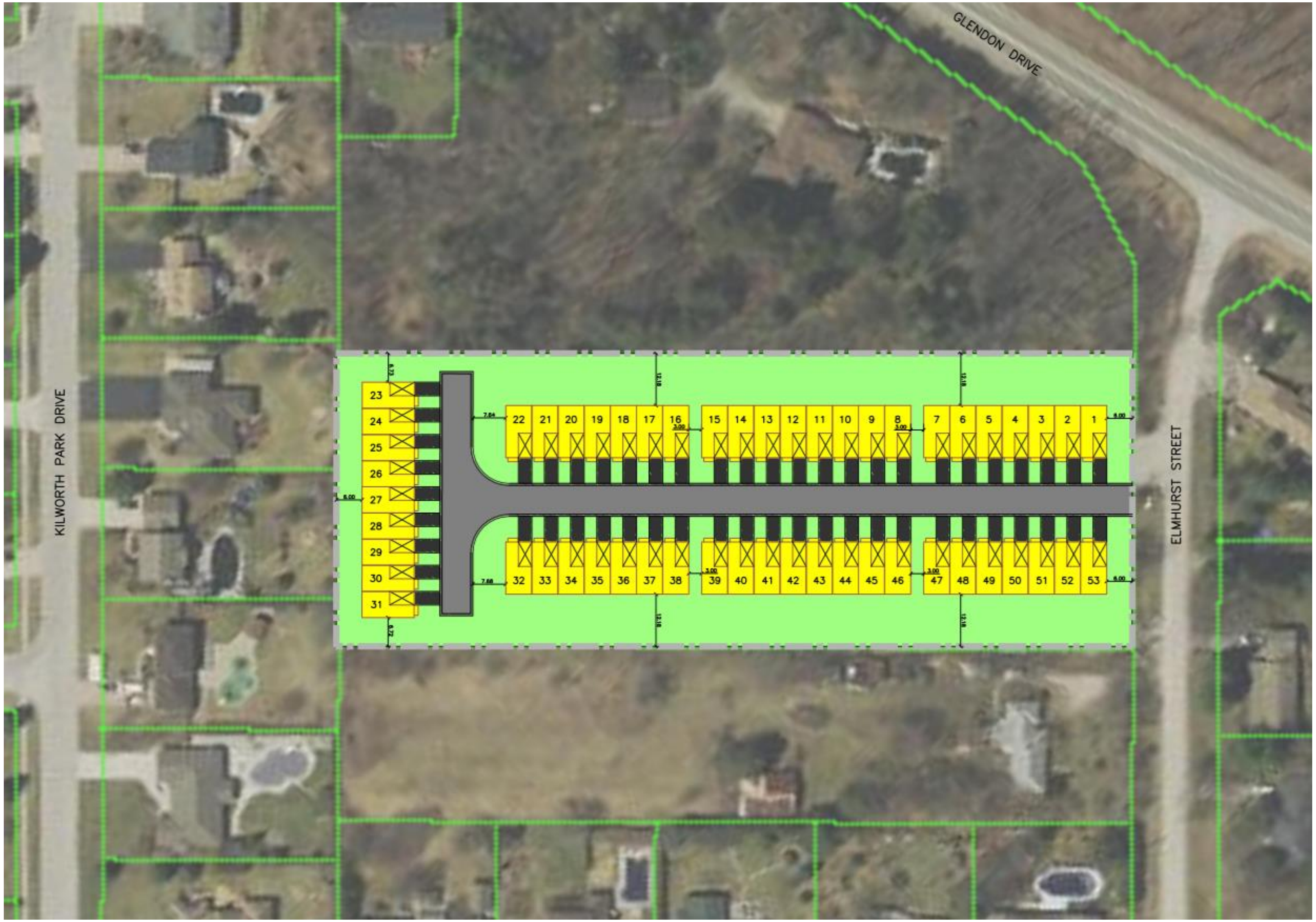
APPENDIX A

DRAWINGS AND NOTES



SOURCE:
Produced from Draft Plan of Subdivision, prepared by LDS Consultants Inc., London, Ontario, 13 September 2019

LDS

PROJECT NAME

Proposed Townhouse Development

PROJECT LOCATION

6 & 10 Elmhurst St, Kilworth

DRAWING NAME

Development Concept Plan

SCALE	PROJECT NO.
1:500	GE-00285
DATE	**DRAWING NO.**
September 2019	**1**

LEGEND

UTRCA Regulated Land

SOURCE

UTRCA Web GIS for Middlesex County, September 2019
https://www.middlesex.ca/departments/mapping

LDS

PROJECT NAME	Proposed Townhouse Development
PROJECT LOCATION	6 & 10 Elmhurst St, Kilworth
DRAWING NAME	UTRCA Regulated Lands

SCALE	PROJECT NO.
AS SHOWN	GE-00285
DATE	**DRAWING NO.**
September 2019	2

SOURCE:
Google Earth Pro, Version 7.3.2.5776,
Coordinates 17T, 532707 m E, 4759733 m N,
Imagery date 7/7/2018

LDS

PROJECT NAME

Proposed Townhouse Development

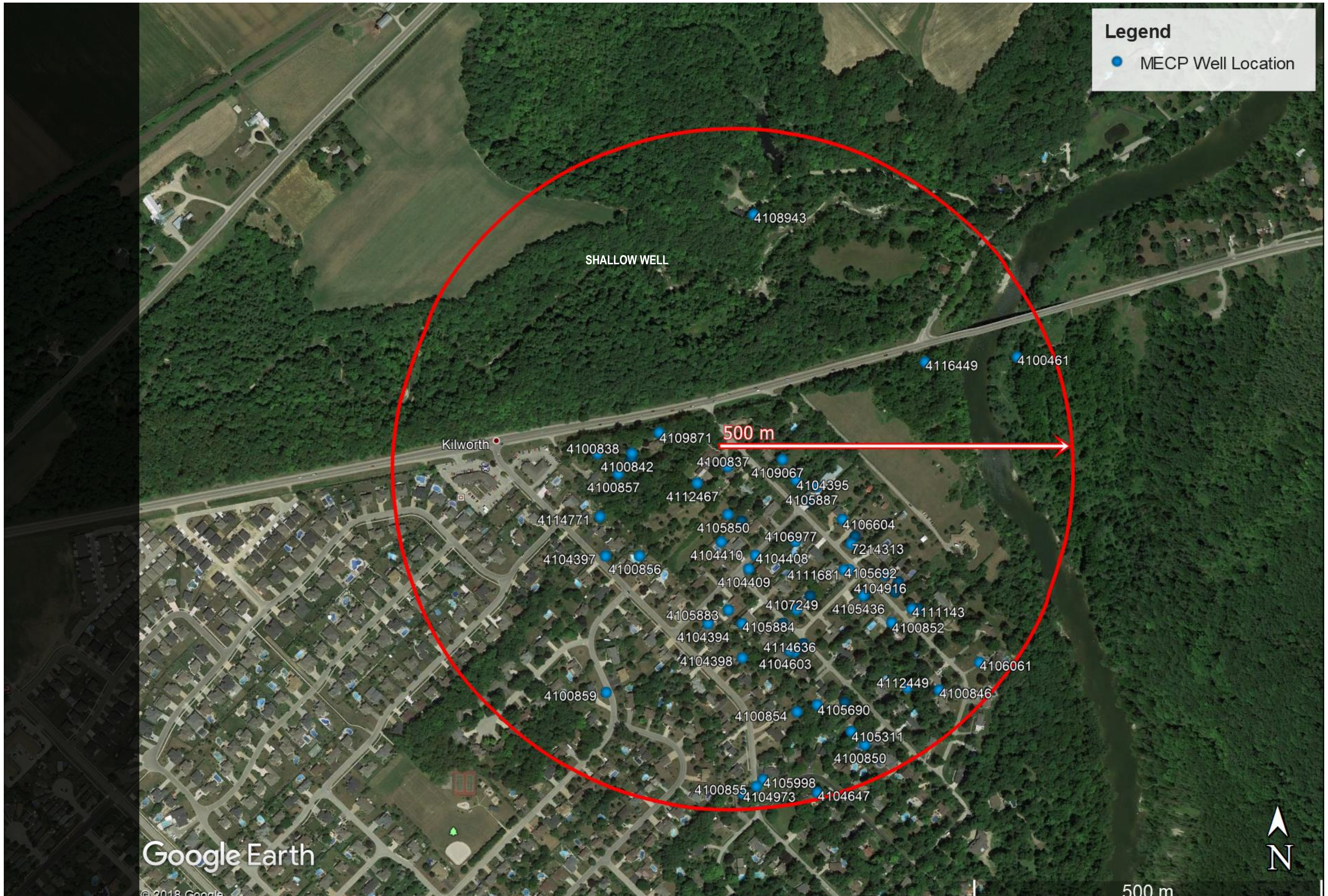
PROJECT LOCATION

6 & 10 Elmhurst St, Kilworth

DRAWING NAME

Borehole Location Plan

SCALE	PROJECT NO.
As Shown	GE-00285
DATE	**DRAWING NO.**
September 2019	**3**



SOURCE:
MECP Well Records: www.ontario.ca/environment-and-energy/map-well-records, updated March 7, 2019

LDS

PROJECT NAME
Proposed Townhouse Development

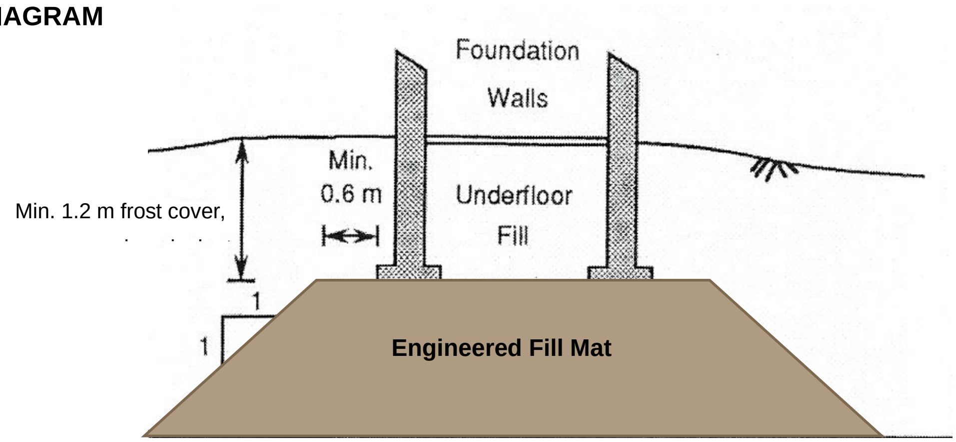
PROJECT LOCATION
6 & 10 Elmhurst St, Kilworth

DRAWING NAME
MECP Well Locations

SCALE	PROJECT NO.
As Shown	GE-00285
DATE	**DRAWING NO.**
September 2019	4

ENGINEERED FILL PLACEMENT

SCHEMATIC DIAGRAM



NOTES:

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

LDS	**PROJECT NAME** Proposed Townhouse Development	**PROJECT NO.** GE-00285
	PROJECT LOCATION 6 & 10 Elmhurst St, Kilworth	**DRAWING NO.** **5**

APPENDIX B

BOREHOLE SUMMARY &
LABORATORY TEST RESULTS

Project	6 & 10 Elmhurst St, Kilworth	Borehole ID	
Project Location	Komoka, Ontario	1	
Project Number	GE-00285	Sheet 1 of 1	

Date Drilled	**September 18, 2019**	Ground Surface Elevation	**239.57 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**Dry**
Drilling Method	**Solid Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**N Houlton, EIT**

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
0.0–0.075						**TOPSOIL** - brown silty loam, moist, 75mm	
1.0	SPT	1	70	7		**SAND** - brown, fine to medium grained, some gravel, damp, loose	
						- becoming compact below 1.37 m depth	
1.5–2.0	SPT	2	70	22			*MC - 2.4%*
2.5	SPT	3	70	19		*Gradation: 15% Gravel, 72% Sand, 13% Silt/Clay*	
3.0–3.5	SPT	4	70	17	4.04m		*MC - 2.8%*
4.04–5.03	SPT	5	60	34	5.03m	**SILT TILL** - grey, trace fine gravel, moist, dense	*MC - 7.4%*
						BH Terminated at 5.03 m Open and Dry at completion	

Legend

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

Well Construction Details

Pipe Diameter	No Well Installation
Installation Depth	
Screen Length	
Depth of Bentonite Seal	

Additional Notes

MC - denotes moisture content

	Project	6 & 10 Elmhurst St, Kilworth	Borehole ID
	Project Location	Komoka, Ontario	2
	Project Number	GE-00285	*Sheet 1 of 1*

Date Drilled	**September 18, 2019**	Ground Surface Elevation	**239.54 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**4.27 m bgs**
Drilling Method	**Solid Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**N Houlton, EIT**

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, moist, 75mm	
0.5						**SAND** - brown, fine to medium grained, trace gravel, damp, loose	
1.0	SPT	1	60	7			
1.5							
	SPT	2	70	7			MC - 2.9%
2.0							
2.5	SPT	3	60	7			
3.0							MC - 2.6%
	SPT	4	60	6			
3.5							
4.0						- saturated and very dense below 4.04 m depth	
4.5							
	SPT	5	60	50*	5.03m		
5.0							
						BH Terminated at 5.03 m Water measured at 4.27 m Open to 3.96 m	

Legend

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

Well Construction Details

Pipe Diameter	No Well Installation
Installation Depth	
Screen Length	
Depth of Bentonite Seal	

Additional Notes

MC - denotes moisture content
*50 blows for 100 mm

LDS	Project	6 & 10 Elmhurst St, Kilworth	Borehole ID
	Project Location	Komoka, Ontario	3
	Project Number	GE-00285	*Sheet 1 of 1*

Date Drilled	**September 18, 2019**	Ground Surface Elevation	**240.36 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**Dry**
Drilling Method	**Solid Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**N Houlton, EIT**

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, moist, 75mm	
0.5						**SAND** - brown, fine to medium grained, some gravel, damp, loose	
1.0	SPT	1	50	8			
1.5							
2.0	SPT	2	70	9			*MC - 3.3%*
2.5	SPT	3	70	7			
3.0							
3.5	SPT	4	70	7			*MC - 2.9%*
4.0						- compact below 4.04 m depth	
4.5							
5.0	SPT	5	70	18	*5.03m*		
						BH Terminated at 5.03 m Open and Dry at completion	

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

Well Construction Details

Pipe Diameter	No Well Installation
Installation Depth	
Screen Length	
Depth of Bentonite Seal	

Additional Notes

MC - denotes moisture content

	Project	6 & 10 Elmhurst St, Kilworth	Borehole ID
	Project Location	Komoka, Ontario	4
	Project Number	GE-00285	*Sheet 1 of 1*

Date Drilled	**September 18, 2019**	Ground Surface Elevation	**240.08 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**Dry**
Drilling Method	**Solid Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**N Houlton, EIT**

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, moist, 75mm	
0.5						**SAND** - brown, fine to medium grained, trace gravel, damp, compact	
1.0	SPT	1	70	10			
1.5	SPT	2	60	15			*MC - 2.5%*
2.0							
2.5	SPT	3	70	29			
3.0	SPT	4	80	20			*MC - 2.3%*
3.5							
4.0							
4.5	SPT	5	70	21			
5.0					*5.03m*	BH Terminated at 5.03 m Open and Dry at completion	

Legend

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

Well Construction Details

Pipe Diameter	No Well Installation
Installation Depth	
Screen Length	
Depth of Bentonite Seal	

Additional Notes

MC - denotes moisture content

LDS	Project: **6 & 10 Elmhurst St, Kilworth** Project Location: **Komoka, Ontario** Project Number: **GE-00285**	Borehole ID **5** *Sheet 1 of 1*

Date Drilled	**September 18, 2019**	Ground Surface Elevation	**239.99 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**Dry**
Drilling Method	**Solid Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**N Houlton, EIT**

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, moist, 75mm	
0.5						**SAND** - brown, fine to medium grained, trace gravel, damp, compact	
1.0	SPT	1	50	17			
1.5							MC - 3.6%
2.0	SPT	2	60	23			
2.5	SPT	3	70	27			
3.0	SPT	4	50	16		- moist below 3.03 m depth	MC - 10.6%
3.5							
4.0						- very dense below 4.04 m depth	
4.5	SPT	5	50	50*	5.03m		
5.0						BH Terminated at 5.03 m Open and Dry at completion	

Legend

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

Well Construction Details

Pipe Diameter	No Well Installation
Installation Depth	
Screen Length	
Depth of Bentonite Seal	

Additional Notes

MC - denotes moisture content

*50 blows for 125 mm

LDS	Project	6 & 10 Elmhurst St, Kilworth
	Project Location	Komoka, Ontario
	Project Number	GE-00285

Borehole ID
6
Sheet 1 of 1

Date Drilled	**September 18, 2019**	Ground Surface Elevation	**240.24 m asl**
Drill Rig	**D50 Turbo**	Groundwater Level at Completion	**Dry**
Drilling Method	**Solid Stem Auger**	Technician	**Rob Walker**
Drilling Contractor	**London Soil Test**	Checked By	**N Houlton, EIT**

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, moist, 75mm	
0.5						**SAND** - brown, fine to medium grained, trace gravel, damp, loose	
1.0	SPT	1	60	8			
1.5						- compact below 1.37 m depth	MC - 2.4%
2.0	SPT	2	60	13			
2.5	SPT	3	70	16		*Gradation: 5% Gravel, 87% Sand, 8% Silt/Clay*	
3.0	SPT	4	60	17			MC - 3.7%
3.5							
4.0						- very dense below 4.04 m depth	
4.5	SPT	5	60	50*			
5.0					5.03m	BH Terminated at 5.03 m Open and Dry at completion	

Legend

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

Well Construction Details

Pipe Diameter	No Well Installation
Installation Depth	
Screen Length	
Depth of Bentonite Seal	

Additional Notes

MC - denotes moisture content
*50 blows for 75 mm, stone in tip

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.

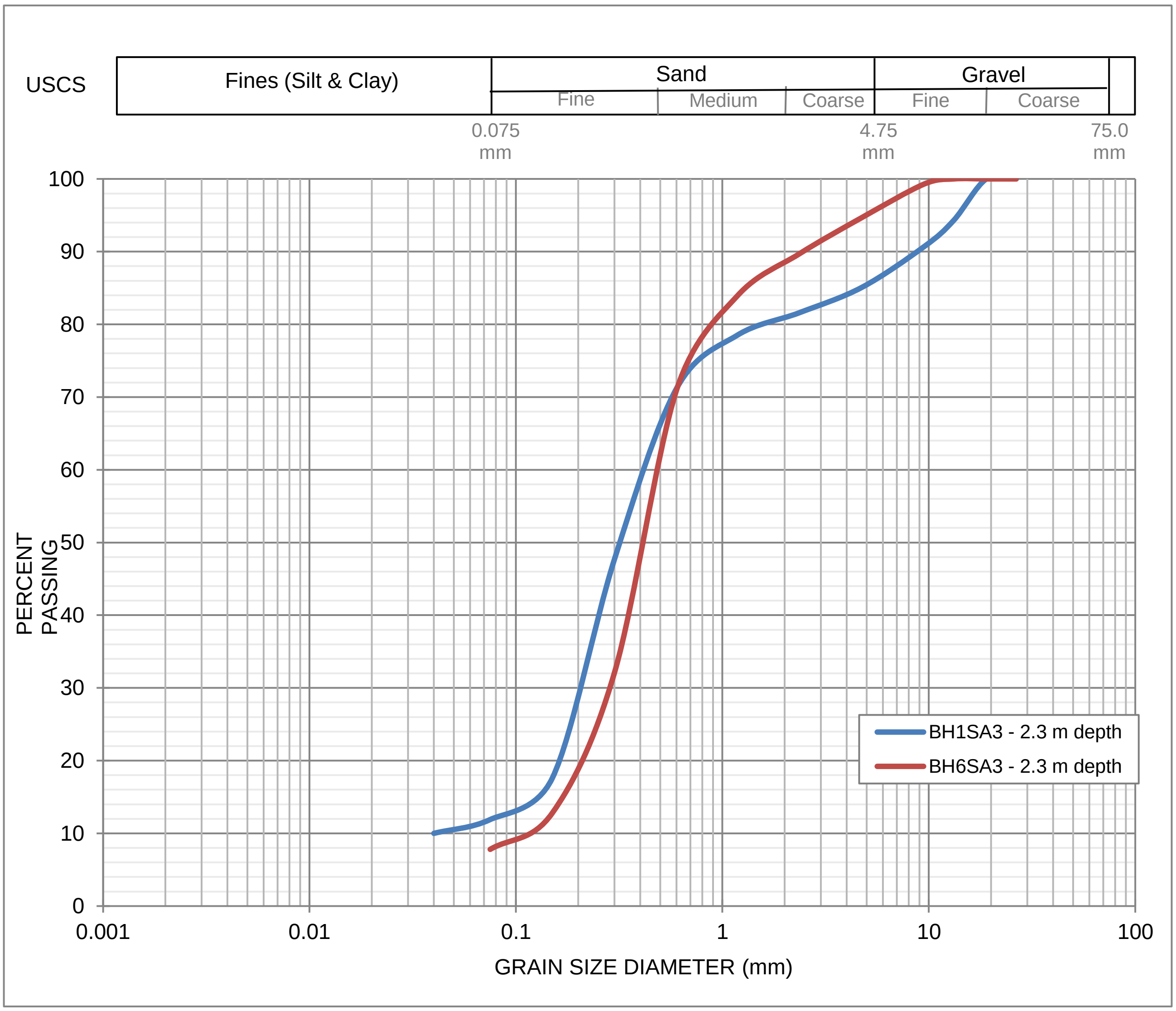
Consistency of Cohesive Soils	Field Identification	Undrained Shear Strength (kPa)
Very Soft	Easily penetrated several cm by the fist	0 – 12
Soft	Easily penetrated several cm by the thumb	12 – 25
Firm	Can be penetrated several cm by the thumb with moderate effort	25 – 50
Stiff	Readily indented by the 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+

Particle Size Distribution
Results of Sieve Analysis

Project Name: Proposed Townhouse Development **Date:** 30-Sep-19

Project Location: 6 & 10 Elmhurst St, Kilworth, Ontario **Project No.:** GE-00285

Sample ID	Unified Soil Classification			
	Fines (Silt & Clay)	% Sand	% Gravel	% Cobbles
BH1SA3 - 2.3 m depth	11.9%	73.2%	14.9%	0.0%
BH6SA3 - 2.3 m depth	7.8%	86.9%	5.3%	0.0%



APPENDIX C

MECP WELL RECORD SUMMARY

MECP Water Supply Wells

MECP Well ID	Registration Year	Well Type	Depth of Well (m)	Depth Water Found (m)	Static Water Level (m)	Pump Rate (L/min)
4100837	04/16/1957	Domestic	8.2	8.2	6.4	*NR*
4100838	06/22/1958	Domestic	13.7	13.7	11.9	*NR*
4100842	10/07/1963	Domestic	8.5	8.5	6.1	19.0
4100846	08/03/1965	Domestic	27.4	25.9	10.7	11.4
4100848	05/31/1965	Domestic	7.0	4.9	4.9	19.0
4100850	04/11/1967	Domestic	3.0	1.8	0.6	11.4
4100851	04/11/1967	Domestic	4.6	3.0	2.7	19.0
4100852	04/11/1967	Domestic	3.7	2.7	2.7	11.4
4100853	11/01/1967	Domestic	29.0	29.0	10.7	19.0
4100854	09/19/1967	Domestic	35.1	32.0	21.3	22.8
4100855	03/01/1968	Domestic	26.2	25.9	3.7	22.8
4100856	03/23/1968	Domestic	37.2	33.5	10.7	30.4
4100857	04/13/1968	Domestic	26.8	26.5	11.3	22.8
4100859	05/11/1968	Domestic	37.5	37.2	11.6	15.2
4104394	07/12/1968	Domestic	28.3	28.0	10.1	22.8
4104395	10/14/1968	Domestic	25.9	25.3	10.7	11.4
4104397	07/25/1968	Domestic	28.7	27.1	7.3	19.0
4104398	06/07/1968	Domestic	25.6	24.1	7.6	19.0
4104408	10/04/1968	Domestic	31.1	30.5	13.7	11.4
4104409	09/26/1968	Domestic	31.4	31.1	12.2	7.6
4104410	09/18/1968	Domestic	53.0	48.8	15.2	7.6
4104603	10/11/1968	Domestic	39.6	36.0	21.3	11.4
4104647	02/13/1969	Domestic	32.3	32.3	12.2	19.0
4104916	10/03/1969	Domestic	33.5	29.0	21.3	38.0
4104973	02/09/1970	Domestic	32.3	29.9	16.2	26.6
4105311	12/29/1970	Domestic	31.4	31.1	14.6	11.4
4105436	07/02/1971	Domestic	39.0	37.8	21.9	15.2
4105690	09/08/1971	Domestic	38.1	34.7	20.1	41.8
4105692	11/11/1971	Domestic	35.1	35.1	21.9	19.0
4105850	04/18/1972	Public	6.1	4.9	4.9	22.8
4105883	04/26/1972	Domestic	5.2	3.0	3.0	19.0
4105884	04/26/1972	Domestic	5.5	3.0	3.0	19.0
4105887	06/16/1972	Domestic	36.6	32.0	26.8	38.0
4105959	06/28/1972	Domestic	9.1	6.1	6.1	30.4

4105998	07/20/1972	Domestic	33.5	32.6	16.8	22.8
4106061	09/18/1972	Domestic	31.4	31.1	14.6	15.2
4106063	10/10/1972	Domestic	47.2	47.2	22.9	15.2
4106064	09/29/1972	Domestic	40.5	40.2	22.3	15.2
4106604	12/27/1973	Domestic	46.0	46.0	22.9	11.4
4106977	10/03/1974	Domestic	8.5	6.7	5.5	22.8
4107249	06/27/1975	Domestic	7.3	3.7	3.7	19.0
4107300	08/15/1975	Domestic	30.5	29.0	20.1	19.0
4108943	09/22/1979	Domestic	33.5	32.6	15.2	15.2
4109067	12/03/1979	Domestic	39.0	38.1	27.4	7.6
4109871	08/31/1983	Domestic	32.3	28.7	27.1	30.4
4111383	05/25/1988	Recharge	42.1	*NR*	*NR*	*NR*
4111681	07/11/1989	Livestock	34.4	34.4	25.6	11.4
4112449	10/03/1991	Domestic	32.9	30.5	17.1	38.0
4112467	11/07/1991	Domestic	43.3	43.0	27.7	11.4
4112940	10/20/1993	Domestic	33.2	29.9	17.1	38.0
4113302	05/12/1995	Domestic	32.6	27.7	28.0	38.0
4114630	03/08/2000	Domestic	40.5	39.6	22.6	30.4
4114636	03/01/2000	Domestic	40.5	39.0	22.6	38.0
4114771	10/07/2001	Domestic	42.7	41.5	25.3	53.2
4115207	05/20/2003	Domestic	26.8	23.8	6.1	38.0
4115547	06/19/2003	Irrigation	32.6	20.1	22.6	95.0
4116449	08/10/2005	Domestic	27.4	23.8	16.2	22.8
7214313	04/23/2013	Domestic	41.5	40.5	*NR*	38.0
7255711	10/22/2015	Domestic	*NR*	*NR*	26.8	38.0
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
4100461	10/04/1961	Test Hole	28.7	*NR*	*NR*	*NR*
4105439	06/24/1971	Abandoned-Other	2.4	*NR*	*NR*	*NR*
4111143	08/08/1987	Abandoned-Quality	43.3	43.0	*NR*	38.0
7201844	05/03/2013	Abandoned-Other	4.3	*NR*	*NR*	*NR*
7255712	10/22/2015	Abandoned-Other	*NR*	*NR*	5.8	*NR*