

STONECAIRN CONSULTING

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GEOTECHNICAL INVESTIGATION & HYDROGEOLOGICAL ASSESSMENT

**PROPOSED PLACE OF WORSHIP & ACCESSORY RESIDENCE
81 COLLEGE AVENUE WEST
GUELPH, ONTARIO**

PROJECT NO. GE-01100

JULY 3, 2025

Submitted to:

**CHABAD OF GUELPH
C/O MONTEITH BROWN PLANNING CONSULTANTS
219 OXFORD STREET WEST, UNIT #302
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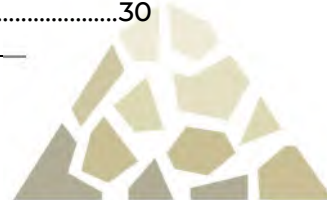
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1. INTRODUCTION

Stonecairn Consulting Inc. [Stonecairn] has been retained by Chabad of Guelph c/o Monteith Brown Planning Consultants to conduct a Geotechnical Assessment for a proposed side redevelopment. The subject property is located northeast of the intersection of College Avenue West and Edinburgh Road South, in the City of Guelph, Municipal Number (MN) 81 College Avenue West. A Key Plan showing the general site location is provided on Figure 1, below.

Figure 1: Key Plan



Based on a review of the proposed development concept plan, it is understood that the proposed redevelopment at the site will include a new Place of Worship Building, and an accessory residence, accessed from College Avenue West. The development is expected to be serviced with municipal sewers and water supply. A Conceptual Layout Plan is provided on Drawing 1, appended.

It is noted that this report has relied upon information provided by LDS Consultants Inc. [LDS], to provide an updated report which includes four seasons of groundwater monitoring data. LDS ceased operations in September 2024; however, Stonecairn is able to provide a continuation of services, under the same supervising engineer that oversaw the work by LDS. The scope of work for this investigation follows the terms outlined in LDS' proposal (reference G2023-178, dated October 31, 2023). This report contains the findings of the Geotechnical Investigation. Authorization to complete this updated report was received from Mr. Raphael Steiner in January 2025.



1.1 Terms of Reference

This report has been prepared for the purposes of providing geotechnical comments and recommendations for the design and construction of a proposed place of worship & residential development located at MN 81 College Avenue West, in Guelph.

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

- A Preliminary Site Plan, prepared Monteith Brown Planning Consultants – 610 Princess Ave, London, dated July 27, 2023; and,
- Pre-Application Consultation Checklist, provided by the City of Guelph, dated September 12, 2023.
- Phase I Environmental Site Assessment Report, prepared by LDS Consultants Inc., project reference GE-01100, November 20, 2023.
- Geotechnical Investigation & Hydrogeological Assessment, prepared by LDS Consultants Inc., project reference GE-01100, May 24, 2024.

This report provides a summary of the borehole findings (documenting soil and groundwater conditions at the site). The report provides geotechnical comments and recommendations for the proposed residential & recreational development, including:

- Site Preparation and Grading, including discussion on excess soils which may be generated from onsite excavations, and which may require disposal offsite, in addition to discussing possible regulatory exemptions, under O.Reg. 406/19.
- Temporary excavations, including maximum slope inclinations to provide stable excavation side slopes in accordance with OHSA requirements, excavation support, and lateral earth pressures;
- Groundwater Control, including the need for a Permit to Take Water (PTTW) or Environmental Activity Sector Registry (EASR) submission for construction dewatering, if required;
- Foundation design, including soil bearing capacity, subgrade preparation and allowable settlements, foundation drainage and backfilling;
- Concrete slab and basement construction;
- Seismic design considerations based on borehole data and published information for soil conditions below the depth of exploration;
- Servicing connections, including recommendations for bedding and backfill materials; and,
- Pavement design and recommendations for concrete curbs and sidewalks within the site.

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



- A summary of MECP well records in vicinity of the site. The information collected from the Well Record review will be used in supplementing the hydrogeological setting in the vicinity of the site.
- Construction dewatering discussion, including estimated construction dewatering volumes, potential zones of influence, and confirming the requirements for permitting required to carry out site servicing which may encounter shallow groundwater conditions.
- Stormwater management considerations, and factored soil infiltration rates for at-source infiltration features, including a discussion on limitations which result from soil and/or shallow groundwater conditions; and providing recommendations for best management practices during construction and the inclusion of at-source infiltration and/or LID measures (where site conditions permit) to increase post-development infiltration volumes.

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. Stonecairn has provided engineering guidelines for the geotechnical design and construction at the site.

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

1.2 Qualifications of Assessor

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

Mrs. Walker has over 25 years of direct experience in the geotechnical and hydrogeological consulting industry. Over 5,500 projects have been completed under her supervision. Mrs. Walker is also a recognized expert in the industry and has testified as an expert witness in Local Planning Appeal Tribunal (formerly Ontario Municipal Board) hearings and Municipal Councils related to groundwater hydrogeology and geotechnical matters for land development and construction. She has been retained for many projects, both directly and indirectly (as a subconsultant) by local municipalities as a hydrogeological and geotechnical consultant.



2. SITE CHARACTERIZATION

2.1 Site Description, Topography and Surface Drainage

Based on a review of aerial photographs dated from 2006 to current, the subject lands have been occupied by a residence, fronting on College Avenue West, and a mixture of vegetation and small to medium sized mature trees located along the site limits. The site is bordered by residential buildings to the north, east and west, and College Avenue West to the south. The site is irregular in shape, and comprises an area of approximately 0.8 acres.

From a topographical perspective, the ground surface at the front of the residence exhibits a gentle relief of 3 meters from northwest to southeast (towards College Avenue), and from west to east at the rear of the building (towards the east property limits.) Any minor surface flows which occur at the site under existing conditions, are generally expected to follow the topography of the site. There are no surface water features present within the site limits.

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

2.2 GRCA Generic Regulation

In April 2024, Ontario Regulation (O.Reg.) 41/24 came into effect, which replaces the former O.Reg.150/06 in the Grand River Conservation Authority (GRCA) watershed. This regulation is intended to ensure public safety, prevent property damage and social disruption, due to natural hazards such as flooding and erosion. O. Reg. 41/24 is implemented by the local Conservation Authority, by means of permit issuance for works in or near watercourses, valleys, wetlands, or shorelines, when required. The subject site is not located within the GRCA Regulated Area, therefore no action from the property owner is required at this time.

2.3 Source Water Protection Mapping

Where proposed developments are being planned, it is important to determine the presence of Significant Groundwater Recharge Areas and High Vulnerability Aquifers in the area. These areas are protected under the Clean Water Act (2006). In general, Significant Groundwater Recharge Areas are defined as areas where water seeps into an aquifer from rain and melting snow, supplying water to the underlying aquifer. A highly vulnerable aquifer occurs where the subsurface material offers limited protection from contamination resulting from surface activities.

The MECP Source Water Protection Information was reviewed to determine whether the site is in any identified areas of source water concern, as they relate to local groundwater quality (current to June 16, 2025). The subject properties are located within the Grand River Source Protection Area, and the following observations are noted for the site:



- The subject property is not located in any of the following designated areas listed in the MECP Source Protection mapping:
 - Highly Vulnerable Aquifer;
 - Significant Groundwater Recharge Area;
 - Wellhead Protection Area or Wellhead Protection Areas Q1 or Q2;
 - Intake Protection Zone or Intake Protection Zone Q;
 - Event Based Area.

- The Site is located in the following designated areas listed in the MECP Source Protection mapping:
 - Wellhead Protection Area B (score 8), and;
 - Issues Contributing Area (contaminant of concern: Trichloroethylene or another DNAPL).

The above comments are demonstrated on Drawing 3, in Appendix A.

2.4 Review of Geological Mapping

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

Physiographic mapping for Southwestern Ontario (*Chapman, L.J. and Putnam, D.F. 2007. Physiography of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 228*), identifies that Guelph is located within the south eastern extent of the physiographic region known as the Guelph Drumlin Field, and is set within a drumlinized Till Plain. Natural subgrade soil conditions are expected to be comprised of predominantly silt and silt till soils.

Quaternary geology mapping for the Guelph area (*Quaternary Geology, Ontario Geological Survey Map 1963, Guelph Area, Scale 1:50,000*) indicates that subject soils within the site limits are predominantly comprised of Wisconsinan-aged outwash gravel deposits, comprised of gravel and sand. An excerpt from the mapping is provided on Drawing 4, in Appendix A.

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

Geological publications and well records in the area indicate that the bedrock surface is below 16 - 18 m of overburden soils in the vicinity of the site. Bedrock was not encountered during the fieldwork for this investigation.



2.5 MECP Well Record Review

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

The water supply wells noted in the records are set in the shallow (<15 m depth), intermediate (15-30 m depth) and deep (> 30 m depth) overburden and limestone bedrock aquifers, with reported static water levels ranging between 4.9 m, 3.7 m and 6.4 m, and 1.5 m and 21.9 m, respectively. Several observation wells (located west, east and southeast of the site) are set in the shallow overburden aquifer, with subgrade soils described as topsoil overlying natural silty sand, gravelly sand and silt/clay till soils. Some water supply wells in the vicinity of the site have been abandoned, following access to municipal water supply/serving which is now available in the area.

3. SUMMARIZED CONDITIONS

3.1 Field Program and Laboratory Testing

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

The field program consisted of a series of four boreholes, drilled on January 15, 2024. The boreholes were advanced at the site by a local drilling-contractor, using a track-mounted drill-rig. The boreholes were advanced to a depth of 6.55 m (21.5 feet) below existing grade. The fieldwork was supervised by geotechnical field staff.

Ground surface elevations at the borehole locations were surveyed using a Trimble R12 GPS rover. The location of the boreholes is summarized below, and illustrated on Drawing 5, in Appendix A.

Table 1 – Borehole Locations

Location	Northing, m N	Easting, m E	Ground Surface Elevation (m asl)
BH1/MW	4819752.71	561549.53	326.36
BH2	4819778.44	561539.13	327.51
BH3	4819780.83	561500.53	327.91
BH4/MW	4819799.33	561487.93	327.78



Monitoring wells were installed in two of the boreholes (BH1 and BH4) to allow for monitoring the stabilized groundwater level at the site. Wells are comprised of a 50 mm diameter CPVC pipe, with a slotted and filtered screen. Details of monitoring well construction are provided on the attached borehole logs. The screens on each well are mill-slotted, with a slot spacing of 0.5 mm, and were backfilled with Type 2 Silica Sand. Above the screened depth, the annular space was backfilled with a bentonite slurry, up to ground surface. The wells have been equipped with lockable caps. The monitoring wells have been registered with the Ministry of Environment, Conservation, and Parks (MECP), in accordance with Ontario Regulation (O.Reg.) 903.

Table 2 (below) summarizes the well construction details.

Table 2 – Well Construction Details

Borehole	Ground Surface Elevation, m	Well Installation Depth, m	Screened Length, m	Screened Strata
BH1/MW	326.36	6.10	3.05	Sandy Silt and Sand and Gravel
BH4/MW	237.78	6.10	3.05	Sandy Silt and Sand and Gravel

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

All samples recovered from the site were returned to the geotechnical laboratory for detailed examination and selective testing. Four (4) grain size analyses were carried out on select samples of the predominant sandy soils, where perched groundwater conditions were identified. Routine moisture content determinations were carried out on select samples and results are presented on the borehole logs provided in Appendix B.

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

3.1.1 Soil Conditions

A series of four 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 overlying interlayered deposits of sand and sandy silt, overlying sand and gravel. General descriptions of subsurface conditions are summarized in the following sections. Borehole logs are provided in Appendix B, for reference.

It should be noted that boundaries of soil indicated in the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries reflect transition zones for



the purposes of geotechnical design and should not be interpreted as exact planes of geological change.

Topsoil

Each borehole was surfaced with a layer of topsoil. The topsoil consisted of dark brown sandy loam, and the thickness generally ranging from 150 to 200 mm across the site. The topsoil was in a damp to moist state at the time of the fieldwork, based on visual and tactile examination. It should be noted that topsoil quantities noted above are based on information provided at the borehole locations only, and may vary in areas with existing vegetation and tree cover. 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

A natural sand layer was encountered underlying the topsoil in each borehole. The sand was generally described as being brown in colour, with a fine-grained texture, and containing trace to some silt. In Boreholes BH2 and BH3, a second sand layer was encountered at depths ranging between 2.1 and 3.7 m below existing grades. The sand was in a variable loose to compact state, based on Standard Penetration Test (SPT) N-values in the range of 7 to 20 blows per 0.3 m of split-spoon sampler penetration. Moisture content determinations conducted on recovered samples of the sand generally range between 13.4 to 20.3 percent, generally indicative of moist to very moist soil conditions.

Sandy Silt / Silt

A layer of sandy silt/silt was encountered underlying the sand layer in each borehole, at depths ranging between 1.4 and 2.0 m below existing grades. In Boreholes BH2 and BH3, a second silt layer was encountered at depths ranging between 2.9 and 4.4 m below existing grades. The sandy silt is described as being brown in colour, with a noted decrease in sand content with depth. Furthermore, several very moist sand seams (~50 to 150 mm thickness) were encountered at variable depths within the silt layer across the entire site. Two samples of the silt layer were submitted for gradation analyses, and the following table shows the grain size distributions. The results are also shown graphically in Appendix B.

Table 3 – Gradation Summary, Sandy Silt

Sample ID	Unified Soil Classification			
	% Clay	% Silt	% Sand	% Gravel
BH1, Sample 4 – 3.0 m depth	7.4	90.8	1.8	0.0
BH4, Sample 6 – 4.6 m depth	7.4	91.2	1.4	0.0

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



The sandy silt was found to be in a variable loose to compact state, based on SPT N-values of 4 to 20 blows per 0.30 m penetration of the split-spoon sampler. Moisture content determinations conducted on recovered samples of the silt generally range between 17.1 to 26.9 percent, generally indicative of moist to very moist soil conditions.

Sand and Gravel

The deepest subgrade soil encountered across the site was a layer of sand and gravel, and each borehole terminated within this layer. The sand and gravel soils were described as brown in colour, with a medium to coarse grained texture, and containing trace to some silt. A sample of the sand and gravel layer was submitted for gradation analysis, and the following table shows the grain size distribution. The results are also shown graphically in Appendix B.

Table 4 – Gradation Summary, Sand and Gravel

Sample ID	Unified Soil Classification			
	% Fines (Silt & Clay)	% Sand	% Gravel	% Cobbles
BH2, Sample 6 – 6.1 m depth	24.5	33.4	42.1	0.0

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

The sand and gravel is in a compact to dense state, based on SPT N-values in the range of 14 to 47 blows per 0.3 m of split-spoon sampler penetration. Very dense (SPT N-value > 50 blows) soil conditions were encountered within the sand and gravel layer in boreholes BH2 and BH4 at depths ranging between 4.0 and 5.6 m below ground surface. Moisture content determinations conducted on recovered samples of the sand and gravel generally range between 3.9 to 11.0 percent, generally indicative of damp to moist soil conditions.

3.1.2 Soil Permeability

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

Based on the gradation results presented in Section 3.1.1, the following values for saturated hydraulic conductivity have been determined for the predominantly sandy soils encountered across the site. Hazen’s method was used to correlate the grain size analysis to the hydraulic conductivity of select soil samples. 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 and Factored Infiltration Rates from Grain Size Analyses

Sample ID	Sample Composition			D_{10} (mm)	Parameter	
	% Fines (Si & Cl)	% Sand	% Gravel		Saturated Hydraulic Conductivity (m/sec)	Factored Infiltration Rate (mm/hr)
Sand and Gravel						
BH2, Sample 6 – 6.1 m depth	24.5	33.4	42.1	0.019	3.61×10^{-6}	26
Silt, trace sand and clay						
BH1, Sample 4 – 3.0 m depth	90.8% Si 7.4% Cl	1.8	0.0	0.0026	6.76×10^{-8}	9
BH4, Sample 6 – 4.6 m depth	91.2% Si 7.4% Cl	1.4	0.0	0.0045	2.03×10^{-7}	12

The gradation analyses for the silt samples were also reviewed and factored infiltration rates were estimated based on the Puckett Method, which correlates the hydraulic conductivity of the soil to the overall clay content.

Table 6 - Hydraulic Conductivity and Factored Infiltration Rates from Puckett Method

Sample ID	Sample Composition				Parameter	
	% Clay	% Silt	% Sand	% Gravel	Saturated Hydraulic Conductivity (m/sec)	Factored Infiltration Rate (mm/hr)
Silt, trace sand and clay						
BH1, Sample 4 – 3.0 m depth	7.4	90.8	1.8	0.0	1.01×10^{-5}	34
BH4, Sample 6 – 4.6 m depth	7.4	91.2	1.4	0.0	1.01×10^{-5}	34

As noted in Table 6, a higher factored infiltration rate is determined for the silt soils using the Puckett method, compared to the Hazen’s correlation noted in Table 5. For design purposes, the more conservative values noted in Table 5 are recommended.

The natural water-bearing silt and sand and gravel soils have a saturated hydraulic conductivity in the range of 10^{-6} to 10^{-8} m/s, based on the results presented above. The infiltration rates have been calculated using correlation from TRCA/CVC Low Impact Development Stormwater Management Planning and Design Guide protocol which references Ministry of Municipal Affairs and Housing (MMAH) Supplementary Guidelines to the Ontario Building Code 1997, SG-6 Percolation Time and Soil Descriptions. A Factor of Safety of 2.5 has been applied, in accordance with TRCA/CVC Low Impact Development Stormwater Management Planning and Design Guide protocol.



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

3.1.3 Shallow Groundwater Conditions

Short term water level observations were recorded from the open boreholes at the completion of drilling. In general, each was found to be open and dry through the full depth of the borehole excavation. Short term water levels are summarized in the following table.

Table 7 - Short Term Groundwater Observations

Borehole	Ground Surface Elevation, m asl	Groundwater Observations, m bgs	Groundwater Elevation, m asl
BH2	327.51	Dry	-
BH3	327.91	Dry	-

Stabilized water level measurements were recorded in the monitoring wells installed across the site between January 2024 and April 2025, as summarized in the following table. The monitoring wells have been left in place to allow for additional seasonal monitoring, if desired.

Table 8 - Stabilized Groundwater Observations

ID	Ground Surface Elev. (m asl)	Depth to Groundwater (m bgs) Groundwater Elevation (m asl)					
		29-Jan 2024	20-Feb 2024	19-Apr 2024	17-May 2024	18-Jul 2024	14-Aug 2024
BH1	326.36	5.29 321.07	5.16 321.20	5.00 321.36	5.14 321.22	5.00 321.22	5.35 321.01
BH4	327.78	Dry to 6.1	Dry to 6.1	Dry to 6.1	Dry to 6.1	5.05 322.73	5.70 322.08

ID	Ground Surface Elev. (m asl)	Depth to Groundwater (m bgs) Groundwater Elevation (m asl)					
		29-Sep 2024	16-Oct 2024	21-Nov 2024	16-Dec 2024	03-Mar-2025	17-Apr-2025
BH1	326.36	5.27 321.09	5.25 321.11	5.13 321.23	5.22 321.14	5.11 321.25	4.91 321.45
BH4	327.78	5.74 322.04	5.69 322.09	5.62 322.16	Dry to 6.1	Dry to 6.1	Dry to 6.1

Shallow groundwater is present within the near-surface silt and sand and gravel soils, below Elevation 322.7 m. Shallow groundwater will vary in response to climatic or seasonal conditions, and, as such, may differ at the time of construction, with higher levels possible during mild weather conditions which create melting conditions, and during wet periods. A groundwater contour plan is provided on Drawing



6, in Appendix A, based on groundwater measurements recorded in July 2024, and shows a southeasterly groundwater flow direction, consistent with the surface topography.

3.1.4 Correlation of Water Levels and Precipitation Levels

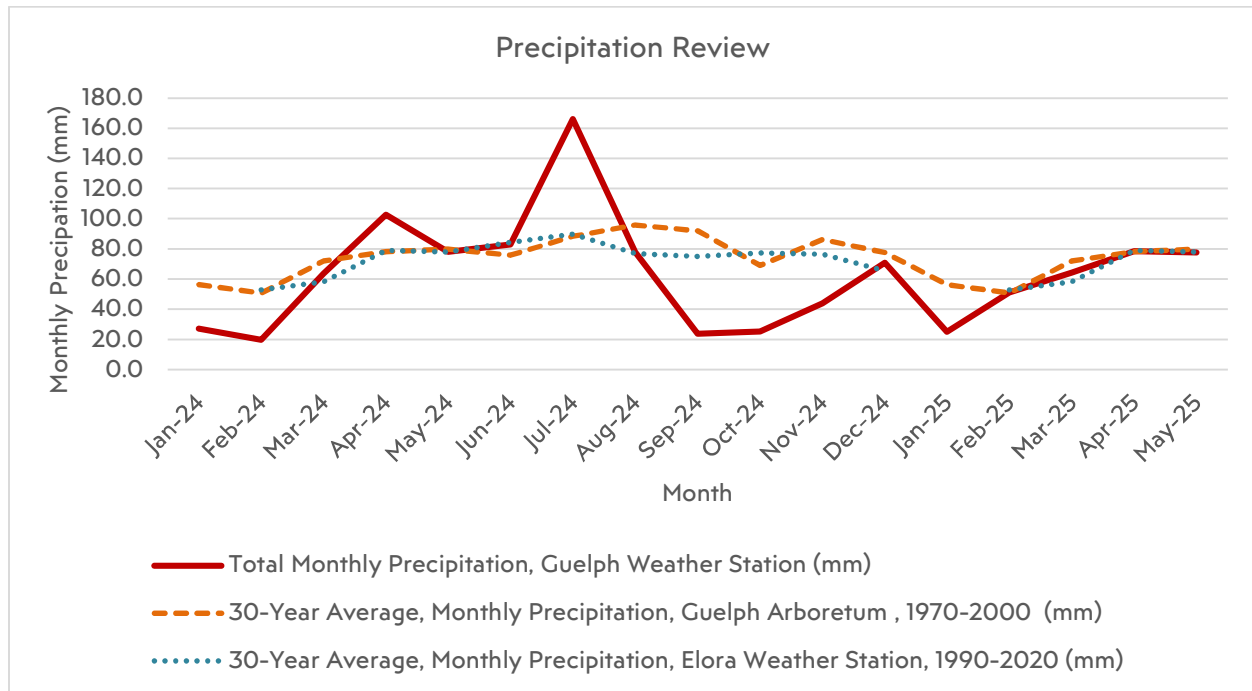
Stonecairn has reviewed the precipitation records for the City of Guelph, to determine how the precipitation records compare to the 30 year historical averages, and to see how water levels are influenced by precipitation levels recorded for the site. Monthly precipitation records for the Guelph Turfgrass Institute (Environment Canada Weather Station Climate ID 6143092) are summarized in the following table. For comparison, the 30-Year average precipitation levels recorded for the Guelph Arboretum for the period from 1970-2000 is provided for reference. A more current monitoring period is not available in the Environmental Canada records, without going to an alternate site, the closest of which is the weather station in Elora, which reports 30-year average records for the period from 1991 to 2020. The information is also shown graphically in Figure 2, below.

Table 9 – Precipitation Records

Month	Total Monthly Precipitation, Guelph Weather Station (mm)	30-Year Average Monthly Precipitation Guelph Arboretum 1970-2000 (mm)	30-Year Average Monthly Precipitation Elora Weather Station 1990-2020 (mm)
January 2024	27.2	56.4	N/R
February 2024	19.7	50.8	52.9
March 2024	63.7	72.1	58.3
April 2024	102.7	78.3	79.1
May 2024	78.1	79.9	78.1
June 2024	83.0	76.0	84.4
July 2024	166.2	88.5	89.9
August 2024	77.9	95.9	77.0
September 2024	23.8	92.1	75.1
October 2024	25.3	69.2	77.5
November 2024	43.8	86.3	76.6
December 2024	70.8	77.7	65.5
January 2025	25.1	56.4	N/R
February 2025	51.0	50.8	52.9
March 2025	64.3	72.1	58.3
April 2025	78.5	78.3	79.1
May 2025	77.7	79.9	78.1



Figure 2: Precipitation Review



Water levels recorded from January to March 2024 were recorded when precipitation levels were below typical average levels, however rain events through April to August were generally at or above typical values. This corresponds with water levels being below the depth of the northerly below in early parts of the year, and then falling within the screened strata in the monitoring well into the summer months. Water levels have been slow to drop through the fall months, although precipitation records show a distinct drop in the final quarter of the year, well below typical levels at this time of year. Based on this data, the water levels recorded in July 2024 when precipitation levels were at their highest is a fair representation of what seasonal high water levels are at the site.



4. GEOTECHNICAL COMMENTS AND DISCUSSION

It is understood that the proposed redevelopment at the site will include a new Place of Worship Building, and an accessory residence. It is understood that the development accessed from College Avenue West, and serviced with municipal sewers and water supply.

The boreholes drilled at the site generally revealed a layer of surficial topsoil which is underlain by sand, sandy silt, and sand and gravel soils. Shallow groundwater is present within the near-surface soils, below Elevation 321.2 m. Depending on the timing of construction, it is anticipated that seasonal conditions may cause variations in the stabilized groundwater level.

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

- Site Preparation and Grading, including discussion on excess soils which may be generated from onsite excavations, and which may require disposal offsite, in addition to discussing possible regulatory exemptions, under O.Reg. 406/19.
- Temporary excavations, including maximum slope inclinations to provide stable excavation side slopes in accordance with OHSA requirements, excavation support, and lateral earth pressures;
- Groundwater Control, including the need for a Permit to Take Water (PTTW) or Environmental Activity Sector Registry (EASR) submission for construction dewatering, if required;
- Foundation design, including soil bearing capacity, subgrade preparation and allowable settlements, foundation drainage and backfilling;
- Concrete slab and basement construction;
- Seismic design considerations based on borehole data and published information for soil conditions below the depth of exploration;
- Servicing connections, including recommendations for bedding and backfill materials; and,
- Pavement design and recommendations for concrete curbs and sidewalks within the site.

4.1 Site Preparation

4.1.1 Building Demolition and Removal of Existing Structures

It is understood that the existing one-story residence will be removed as part of the proposed site redevelopment.

It is possible that hazardous building materials and/or designated substances (such as lead paint, vermiculite which may contain asbestos, and other asbestos containing materials) may be present in the buildings. Owner requirements are set forth by the Ontario Ministry of Labour (MOL) under Ontario Regulation (O.Reg.) 490/09: Designated Substances; Section 30(1) of the Ontario Occupational Health



and Safety Act (OHSA); and Section 10 of O.Reg. 278/05: Designated Substance – Asbestos on Construction Projects and in Buildings and Repair Operations, as amended. It is recommended to complete a Designated Substances Survey / Hazardous Building Materials Survey (DSS/HMS) for the Site building prior to demolition to identify any hazardous substances within the premises and to provide guidance on how to handle/maintain any such products during the demolition.

During the demolition of the buildings, it is recommended that any existing concrete floor slab and building foundations be removed, to prevent future conflicts with site servicing, foundation construction and local roadways. In the event that buried and/or abandoned septic tanks or septic distribution systems are encountered, they should be removed in their entirety. Septic tanks should be pumped out by licensed contractors, prior to being removed from the site. Excavations should be restored in accordance with the engineered fill recommendations identified in the following section.

In the event that a water supply well is encountered, it should be decommissioned in accordance with O.Reg. 903, under the supervision of a licensed well technician if they are no longer being used. This same regulation applies to the decommissioning of monitoring wells, when they are no longer required.

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 150 to 200 mm of surficial topsoil. Thicker topsoil areas may also be present between the borehole locations, and where local depressions are present at the site.

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

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

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



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

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

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

The placement of the engineered fill should be monitored by the geotechnical consultant to verify that suitable materials are used, and to confirm that suitable levels of compaction are achieved. The engineered fill material should be placed in maximum 300 mm (12 inch) thick lifts and uniformly compacted to 100 percent Standard Proctor Maximum Dry Density (SPMDD). Additional notes regarding engineered fill placement are provided on Drawing 7, in Appendix A.

4.1.2 Excess Soils Management Considerations

In December of 2019, the Ministry of Environment, Conservation, and Parks (MECP) released a regulation under the Environmental Protection Act, titled *On-Site and Excess Soil Management* to support improved management of excess construction soil. The current version of Regulation 406/19 was most recently amended in December 2024.

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

Generally, these requirements include:

- Preparation of an Assessment of Past Uses Report which is similar to a Phase One Environmental Site Assessment for the source site, to evaluate the presence of potentially



contaminating activities which may have resulted in the potential for impacted soil or groundwater conditions to be present at the source site;

- Preparation and implementation of a Sampling and Analysis Plan which outlines the suggested sample locations and sampling intervals, analytical sample testing parameters, and sampling frequency;
- Preparation of a Soil Characterization Report, following the soil sampling and analytical testing;
- Preparation of an Excess Soil Destination Assessment Report which identifies where excess soils can be disposed offsite, including a review of Beneficial Reuse Sites, if the developer and/or their contractor have a potential re-use site being considered; and,
- Development and implementation of a tracking system.

The site is within a predominantly residential area. Stonecain is not currently aware of the site being considered as an “enhanced investigation project area” as defined in O.Reg. 406/19 and O.Reg. 153/04, as amended. Provided that no significant environmental concerns were identified with respect to current and/or former activities at the subject properties, the proposed development qualifies for an exemption from the regulatory requirements (preparation of planning documents, soil characterization, and tracking requirements), as noted in Section 8 of the Regulation.

Offsite disposal of excess soils may still require soil characterization and analytical testing to satisfy the receivers of excess soil. Coordination with their QP will be required to ensure that their testing requirements are satisfied prior to transporting soils offsite for beneficial reuse or disposal off-site.

In the event that the site requires imported fill material to achieve design grades, the site would be characterized as a Beneficial Re-Use Site. As such, a Qualified Person (QP) will need to be retained to prepare an Excess Soil Destination Assessment Report (ESDAR), which outlines the geotechnical requirements for beneficial reuse of imported materials onsite along with the environmental soil quality criteria (including the applicable O.Reg. 153/04 Site Condition Standards) for material which is appropriate to be accepted at the Site. In this case, material meeting the O.Reg. 406/19 Table 2.1 Site Condition Standards, Residential/Parkland/Institutional Land Use (or better) is generally considered appropriate for this site.

4.2 Methane Abatement

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

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



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

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

4.3 Excavations and Groundwater Control

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

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

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

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

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

4.3.1 Excavation Support

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



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

A prefabricated trench box may be used for service trench excavations, provided that it is designed (by a professional engineer) to withstand the soil and hydrostatic loading (if applicable). Based on the field and laboratory testing during the present geotechnical investigation and our experience with similar soils, the following soil parameters are recommended for the design of the engineered shoring system.

Table 10 - Soil Parameters for Excavation Support

Soil	ϕ	γ (kN/m ³)	K_a	K_o	K_p
Compact Sand and Silty Sand	30	19.5	0.33	0.50	3.15
Compact Silt / Silt Till	28	19.5	0.36	0.53	2.78
Compact Sand and Gravel	30	21.5	0.32	0.47	3.20
Compact Granular 'B' (OPSS 1010)	32	22.0	0.31	0.47	3.25
Notes: Φ denotes internal friction angle (degrees) K_a denotes active earth pressure coefficient (Rankine, dimensionless) γ denotes soil bulk unit weight K_o denotes at-rest earth pressure coefficient (Rankine, dimensionless) K_p denotes passive earth pressure coefficient (Rankine, dimensionless)					

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 5.1 to 5.3 m below existing ground surface. Conventional groundwater control methods are expected to be suitable for shallow excavations which remain above the groundwater table at the site, to address surface water infiltration and minor shallow groundwater seepage for excavations which do not extend below the stabilized groundwater table.

In the event that servicing excavations extend into areas where shallow groundwater is present, positive groundwater control methods may need to be utilized for construction dewatering. Soil permeability values in the natural subgrade soils are expected to be in the range of 10⁻⁵ to 10⁻⁶ m/s, based on laboratory testing (presented in Section 3.1.2). This information is provided to assist with determining appropriate construction dewatering methods. The use of sump pits and pumps and/or interceptor trenches to reroute groundwater seepage which can accumulate in open excavations is expected to be sufficient for groundwater control of typical excavations.



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

Although not anticipated based on the current soil and groundwater information for the site, it should be noted that for projects requiring positive groundwater control with a removal rate in excess of 50,000 litres per day, a submission to the Environmental Activity and Sector Registry (EASR) will be required, and a Permit to Take Water (PTTW) will be required for volumes in excess of 400,000 litres per day.

4.4 Building Design and Construction

4.4.1 Foundation Design

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

- Serviceability Limit States (SLS) 125 kPa (~2500 psf)
- Ultimate Limit States (ULS) 145 kPa (~3000 psf)

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

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

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

The natural subgrade soils may be susceptible to disturbance by construction activities, especially during adverse weather conditions or when water seepage from excavation sidewalls is present. Considering the presence of shallow groundwater encountered at the site, and variable loose sandy



soils at shallow depth, after the founding surfaces have been exposed, the soils should be thoroughly compacted to provide a uniform base, suitable to provide the bearing capacity noted above. Due to capillary rise which can occur in fine grained soils, the proof-roll should be carried out without vibration or excess disturbance to the subgrade soils. Consideration should be given to placing concrete foundations as soon as possible following excavation and subgrade inspection.

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

- For strip footings, the transition zones should be adequately reinforced with additional reinforced steel lap lengths or widened footings;
- Steel reinforced poured concrete foundation walls; and
- Control joints throughout the transition zone(s).

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

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

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

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



4.4.2 Concrete Slab Construction

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

In preparation for the construction of the floor slab, any unstable (loose) fill material should be removed and recompacted (as noted previously) where founding soils will support the floor slab. In the event that the exposed subgrade soils are wet they will exhibit a greater sensitivity to disturbance. Structural fill placed below the concrete floor slab should be comprised of inorganic soils, placed and compacted in uniform lifts, to a minimum of 98 percent SPMDD.

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

A moisture barrier, consisting of a minimum 200 mm thick of uniformly compacted 19 mm clear stone should be placed over the approved subgrade. For design purposes, the modulus of subgrade reaction (k) can be taken as 30 MPa/m, for the compacted stone over approved subgrade soils. An alternate configuration of compacted granular material such as OPSS 1010 Granular A may also be considered for the moisture barrier. If alternative materials are proposed for use onsite, the minimum level of compaction and overall design thickness of the moisture barrier layer should be reviewed by the geotechnical consultant.

The water-to-cement ratio of the concrete utilized in the floor slab should be strictly controlled to minimize shrinkage of the slab. Adequate joints and / or the use of fibre reinforcement may be considered by the designer to help control cracking. The sawcut depth for control joints should be $\frac{1}{4}$ of the slab thickness. The use of super plasticizers should be considered to reduce shrinkage and increase workability of the concrete.

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

4.4.3 Basement Construction

The proposed accessory residence may be constructed with full depth foundations with basements. The basement floors can be constructed using cast slab-on-grade techniques provided that the subgrade is stripped of unsuitable material. It is recommended that a minimum 200 mm (8 inch) thick compacted layer of 19 mm ($\frac{3}{4}$ inch) clear stone be placed between the prepared subgrade and the floor slab to serve as a moisture barrier.



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

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

where, P = lateral earth pressure in kPa acting at depth h;

γ = natural unit weight, a value of 20.0 kN/m³ may be assumed;

h = depth of point of interest in m;

q = equivalent value of any surcharge on the ground surface in kPa.

K = earth pressure coefficient, assumed to be 0.4

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

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

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

4.4.4 Seismic Design Considerations

Subsoil and groundwater information at the Site have been examined in relation to Section 4.1.8.4 of the Ontario Building Code (OBC) 2012. The subsoils expected below the buildings will generally consist of compact sand, silt and sand and gravel soils.

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 Site Classification recommendation is based on the available information as well as our interpretation of conditions at and below the boreholes, and based on a review of geological mapping, and our knowledge of the soil conditions in the area.

Based on the above assumptions, interpretations in combination with the known local geological conditions, the Site Class for the proposed development is classified as "D" as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012. In the event that a higher Site Classification is



being sought by the structural design engineer, additional deep boreholes and / or multichannel analysis of surface waves (MASW) testing would be required to determine if the soil conditions below the current depth of exploration can support a higher Site Classification.

4.4.5 Concrete Recommendations

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

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

4.5 Site Services

Subgrade soils beneath new services are generally expected to consist of sand, silt and 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, in consultation with the 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 sand, silty sand, and sand and gravel. Select portions of this inorganic material may be used for construction backfill provided that reasonable care is exercised in handling the material. In this regard, material should be within 3 percent of the optimum moisture as determined by the Standard Proctor density test. Stockpiling of material for prolonged periods of time should be avoided. This is



particularly important if construction is carried out in wet, adverse weather. Backfilling operations during cold weather should avoid inclusions of frozen lumps of material, snow and ice.

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

Table 11 - Trench Backfill Compaction Requirements

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

Soils excavated from below the stabilized groundwater table may be too wet for re-use as backfill, unless adequate time is allowed for drying, or if material is blended with approved dry fill; otherwise, it may be stockpiled onsite for re-use as landscape fill, or disposed of off-site, testing of the material for transport should conform to MECP Guidelines and requirements. Backfill above bedding aggregate can consist of excavated (inorganic) soils, compacted in maximum 300 mm thick lifts to a minimum of 95 percent SPMDD. A program of in situ density testing should be set up to ensure that satisfactory levels of compaction are achieved.

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

4.6 Pavement Design

The development will be accessed with an internal road network, accessing College Avenue to the southeast. The exposed subgrade soils within the roadways are expected to be comprised of re-compact soils comprised of silt, sand, and sand and gravel. The road subgrade should be thoroughly proof-rolled and reviewed by the geotechnical consultant. In the event that loose or soft areas are noted, additional work may be required to sub excavate and replace unstable soils with suitable compactable material. In general terms, subgrade soils supporting site pavements should be compacted to a minimum level of 98 percent SPMDD.



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

Table 12 – Pavement Design Recommendations

Pavement Component for Local Roads	Pavement Component Thicknesses		Compaction Requirements
	Site Access & Surface Parking	Restoration of College Avenue (if required)	
Asphaltic Concrete	40 mm HL 3 45 mm HL 8	Match existing asphalt thicknesses (est. 2 lifts)	97% Bulk Relative Density (BRD)
Granular A Base	150 mm	150 mm	100% SPMDD
Granular B Subbase	250 mm	300 mm	100% SPMDD

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

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

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

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



Consideration may be given to utilizing permeable pavements within a portion of the site. If this option is selected, the manufacturer specifications for granular support and drainage properties of granular base materials should be reviewed to ensure conformance with materials used during construction.

Good drainage provisions will optimize pavement performance. The finished pavement surface should be free of depressions and should be sloped (preferably at a minimum grade of two percent) to provide effective surface drainage. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas.

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

4.7 Curbs and Sidewalks

Concrete for any new exterior curbs and sidewalks should be proportioned, mixed placed and cured in accordance with the requirements of OPSS 353, and OPSS 1350. Field sampling and testing of concrete should be in accordance with OPSS 904.

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

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

4.8 Erosion and Sediment Control

Sediment and erosion control measures will be required during construction, particularly around the zone of construction, to contain and filter sediment-laden stormwater run-off, before reaching existing stormwater infrastructure. The design of the Sediment and Erosion Control Plan for the site should 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 general mitigation measures are suggested as best management practices:

- Delineate work areas to limit construction activities.
- Monitoring of discharge water (for water quality – turbidity) from stormwater run-off and construction dewatering activities.
- Maintain perimeter silt fence (and other perimeter ESC measures) in place until disturbed areas are stabilized.
- Build-up boulevard areas to help limit sediment-laden stormwater run-off from discharging into catchbasins and stormwater infrastructure, and regular inspection and maintenance of silt bags / geotextile filters installed in catchbasins.

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 conditions.

4.9 Geotechnical Inspection and Testing

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

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

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



5. HYDROGEOLOGICAL DISCUSSION

5.1 Hydrogeologic Setting

As discussed in Section 3.1.2, stabilized water levels were recorded during two return visits to the site, prior to issuance of this report. The results indicate that the shallow groundwater contained within the near surface soils, below Elevation 321.2 m. Where encountered, it is anticipated that the groundwater flow direction will follow site topography, towards the south.

The shallow groundwater encountered in the monitoring wells installed at the site contact the shallow (< 15 m depth) unconfined overburden aquifer. This type of aquifer can be interconnected with surface water features, and is generally fed by infiltrated surface water. Shallow overburden aquifers tend to be heavily influenced by site topography. It is important to note that shallow groundwater will vary in response to climatic or seasonal conditions, and, as such, may differ depending on the seasonal conditions. Shallow groundwater in unconfined aquifers can be significantly influenced by exceptional and/or sustained rainfall events.

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

As shown on Drawing 4 in Appendix A, bedrock is estimated at more than 16 to 18 m below ground surface in the vicinity of the site. As such, the potential impact to the bedrock aquifer from the proposed residential development at the site is not anticipated to be significant, and no further discussion is provided regarding the bedrock aquifer.

5.2 Water Quality Considerations

Baseline groundwater conditions (including general chemistry parameters) have not been established under the current scope of work for this investigation. Prior to construction, consideration may be given to carrying out baseline water quality sampling to establish the general chemistry and characteristics of the shallow groundwater, if encountered. Stonecairn is not aware of any contaminant plumes or existing environmental contamination in the vicinity of the site.

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



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

5.2.1 Potential Impact from Construction Equipment

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

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

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

5.2.2 Potential Impact from Uncontrolled Erosion / Sediment Discharge

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

Sediment and erosion control measures will be required to limit sediment discharge towards the natural features. It is important to ensure that the sediment control measures are installed properly, and in accordance with the design drawings. If deficiencies are identified in its performance through regular inspection, enhancements beyond the recommended design may be required. Refer to additional discussion in Section 4.8.



5.3 Impact Assessment

5.3.1 Construction Dewatering

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

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

Based on the available information, construction dewatering for trench excavations are not expected to require an Environmental Activity and Sector Registry (EASR) submission or Permit to Take Water (PTTW). However, if localized areas require active construction dewatering, a program which includes turbidity monitoring is recommended, to confirm that the quality of discharge water will not have adverse impacts to sensitive receptors. In the event that water discharged from the site is considered to have an elevated turbidity level, associated construction activities should be halted until remedial measures can be implemented. Such measures may include enhanced or more robust sediment and erosion control measures, incorporating pooling areas and measures that will reduce suspended solids, temporary storage measures to prevent off-site discharge.

5.3.2 Local Water Supply Wells

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

As noted in the MECP well records, one water supply wells in the general vicinity of the site is set within the shallow overburden aquifer, however the well is located ~450 m southwest of the site, and is therefore unlikely to be affected by construction dewatering activities.

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

5.3.3 Well Decommissioning

Monitoring wells associated with the preparation of this report have been installed at the site, to document stabilized groundwater conditions. When the monitoring wells are determined to be no longer required, the wells should be properly decommissioned in accordance with Ontario Regulation



903. This regulation identifies that only certified and qualified well drilling technicians are permitted to direct the decommissioning work for existing wells.

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

5.4 Low Impact Development Considerations

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

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

Based on the permeability results presented in Section 3.1.2, the natural water-bearing subgrade soils have a saturated hydraulic conductivity in the range of 10^{-5} to 10^{-6} m/s, corresponding to factored infiltration rates in the range of 26 to 34 mm/hr. In addition, consideration must be given to the depth of shallow groundwater, and the potential drawdown effects of site servicing on perched groundwater levels.

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

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



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



6. CLOSING

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

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

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

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

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

Respectfully Submitted,

STONECAIRN CONSULTING INC.

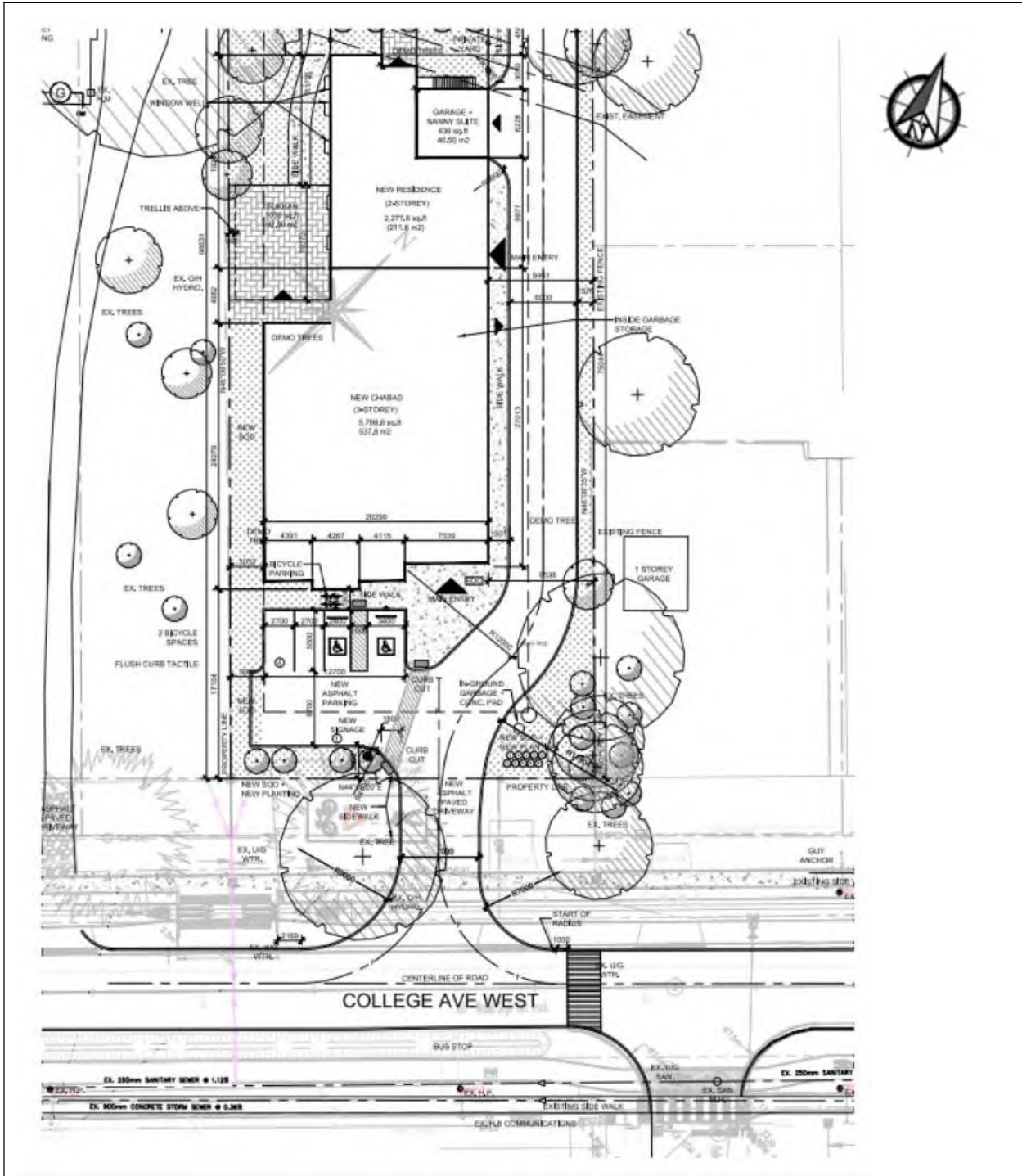


Rebecca A. Walker, P. Eng., QP_{ESA}
Principal, Geotechnical Services
rebecca.walker@stonecairn.ca



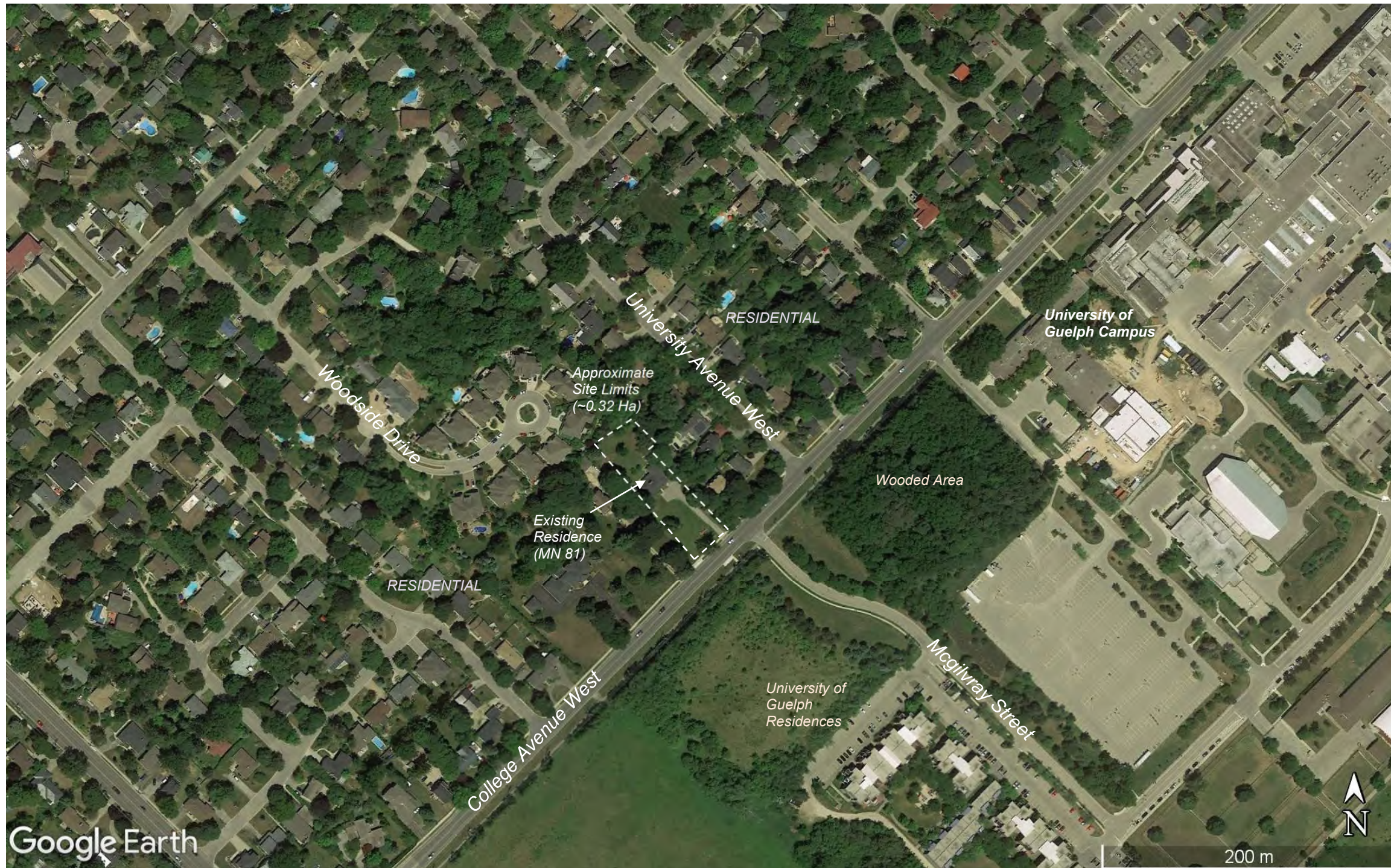
APPENDIX A

DRAWINGS AND NOTES



SOURCE: Proposed Site Plan, prepared by a+LINK Architecture Inc., January 24, 2025

<p>PROJECT NAME PROPOSED PLACE OF WORSHIP & ACCESSORY RESIDENCE</p>	<p>PROJECT LOCATION 81 COLLEGE AVENUE WEST, GUELPH, ON</p>	<p>SCALE NTS</p>	<p>PROJ. NO. GE-01100</p>
<p>STONECAIRN CONSULTING</p>	<p>DRAWING NAME PROPOSED SITE PLAN</p>	<p>DATE JUL 2025</p>	<p>DWG. NO. 1</p>



SOURCE

Google Earth Pro, Version 7.3.6.9345,
Coordinates 17T, 467453 m E, 4750875 m N,
Imagery date 7/2/2018



PROJECT NAME

PROPOSED PLACE OF WORSHIP & ACCESSORY
RESIDENCE

PROJECT LOCATION

81 COLLEGE AVENUE WEST, GUELPH, ON

DRAWING NAME

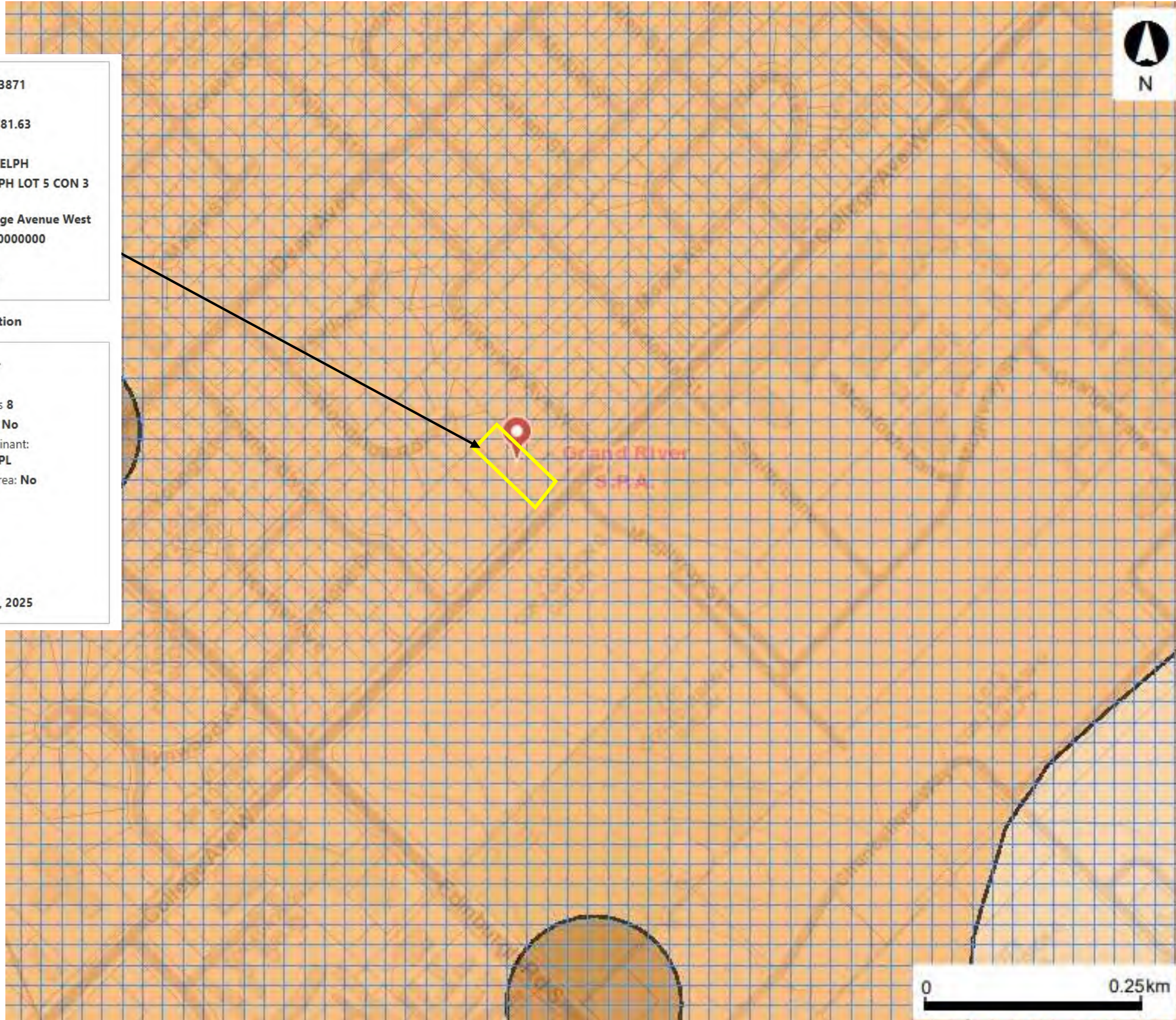
SITE FEATURES

SCALE AS SHOWN	PROJECT NO. GE-01100
DATE JULY 2025	DRAWING NO. 2

Latitude: **43.52844** Longitude: **-80.23871**
 UTM Zone: **17**
 Easting: **561517.04** Northing: **4819781.63**
 Upper Tier Municipality: **N/A**
 Lower Tier Municipality: **CITY OF GUELPH**
 Township Concession and Lot: **GUELPH LOT 5 CON 3 DIVISION G**
 Assessment Parcel Address: **81 College Avenue West**
 Assessment Roll #: **2308060006021000000**
 MECP District: **Guelph**
 MECP Region: **West Central Region**

Source Protection Details for Location

Source Protection Area: **Grand River**
[View Source Protection Plan](#)
 Wellhead Protection Area: **B** ; score is **8**
 Wellhead Protection Area (WHPA-E): **No**
 Issue Contributing Area: **Yes** Contaminant: **Trichloroethylene or another DNAPL**
 Significant Groundwater Recharge Area: **No**
 Highly Vulnerable Aquifer: **No**
 Event Based Area: **No**
 Wellhead Protection Area Q1: **No**
 Wellhead Protection Area Q2: **No**
 Intake Protection Zone Q: **No**
 Information is current as of: **June 16, 2025**



LEGEND

- Source Protection Areas
- Issue Contributing Areas
- Highly Vulnerable Aquifers
- Significant Groundwater Recharge Area
- 0
- 2
- 4
- 6
- 8
- 10
- WHPA Groundwater Under Direct Influence (WHPA-E)
- Wellhead Protection Area
- A
- B
- C
- G1
- D
- F
- Intake Protection Zone 1
- Event Based Areas
- Intake Protection Zone 2
- Vulnerable Scoring Area - Groundwater
- 2
- 4
- 6
- 8
- 10

Source

Ministry of Environment, Conservation and Parks – Source Water Protection Atlas, current to June 2025



PROJECT NAME

PROPOSED PLACE OF WORSHIP & ACCESSORY RESIDENCE

PROJECT LOCATION

81 COLLEGE AVENUE WEST, GUELPH, ON

DRAWING NAME

SOURCE WATER PROTECTION MAPPING

SCALE	PROJECT NO.
AS SHOWN	GE-01100
DATE	DRAWING NO.
JULY 2025	3

CENOZOIC

Pleistocene

Recent

- 9 Modern alluvium: Gravel, sand, silt.
- 8 Swamps and bogs: Peat, muck, marl.

Wisconsinan

- 7 Lacustrine, kame, and outwash sand.
- 6 Flood deposits: Silt and clay.
- 5 Outwash gravel.
- 4 Kames and eskers: Sand and gravel.
- 3 WESTWORTH TILL: Buff or pink sandy till.
- 2 Clay till, Silty sand till.

PALEOZOIC

Silurian

- 1 ALBERMARLE GROUP: Dolomite.

SYMBOLS

- Glacial striae.
- Sand and gravel pit.
- Rock quarry.

For other conventional signs refer to 1:50,000 National Topographic Map Series.



SOURCE
 Pleistocene Geology, Guelph Area, Ontario Geological Survey Map P0189,
 Scale 1:50,000, © 1963



SOURCE
 Bedrock Topography, Guelph Area, Ontario Geological Survey Map P2224
 Scale 1:50,000, © 1979



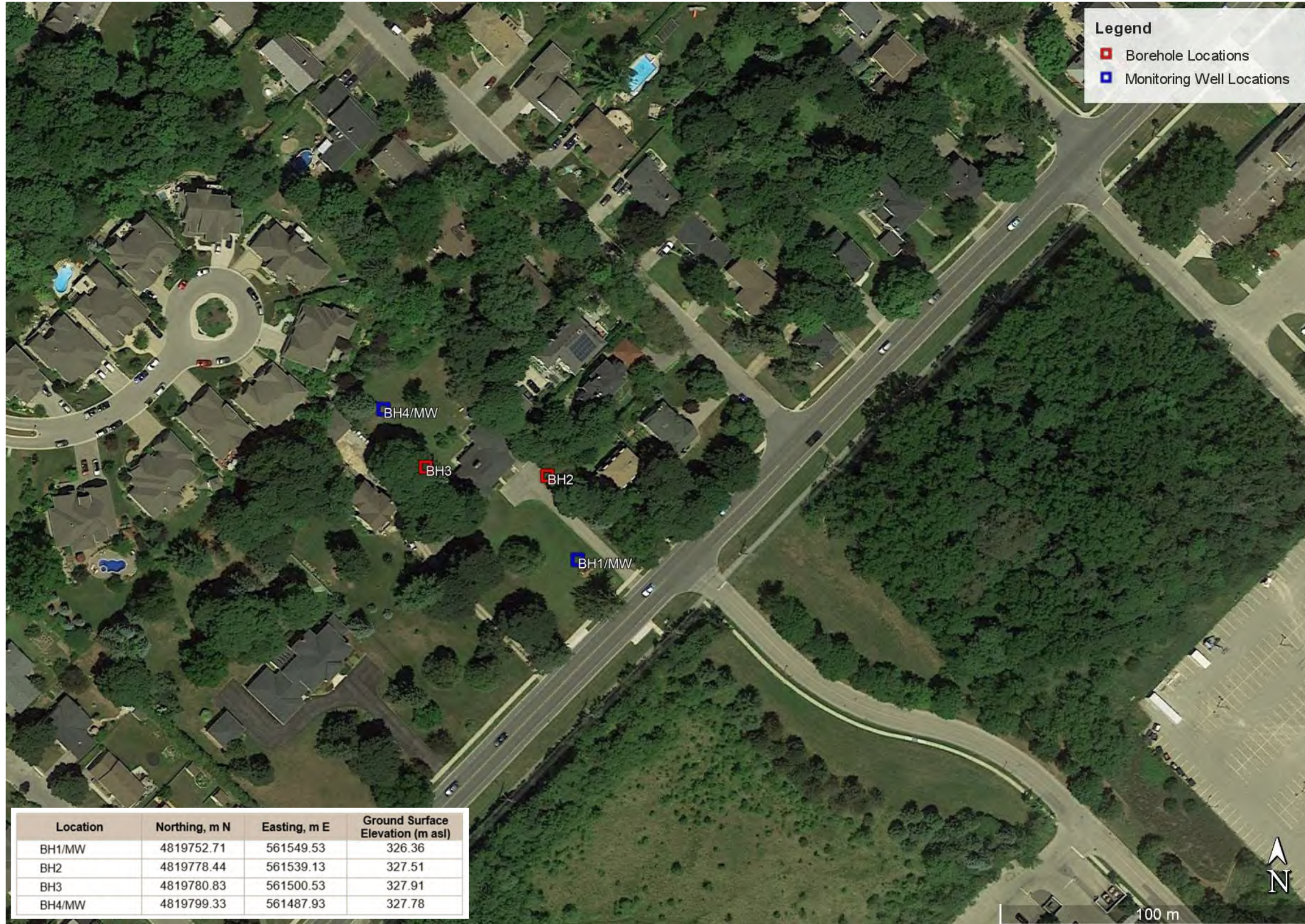
STONECAIRN CONSULTING

PROJECT NAME
 PROPOSED PLACE OF WORSHIP &
 ACCESSORY RESIDENCE

PROJECT LOCATION
 81 COLLEGE AVENUE WEST, GUELPH, ON

DRAWING NAME
 GEOLOGICAL MAPPING

SCALE 1:50,000	PROJECT NO. GE-01100
DATE JULY 2025	DRAWING NO. 4



Legend

- Borehole Locations
- Monitoring Well Locations

SOURCE:
 Google Earth Pro, Version 7.3.6.9345,
 Coordinates 17T, 561548 m E, 4819782 m N,
 Imagery date 7/7/2018

NOTE:
 BH Locations surveyed using a Trimble R12
 GPS Rover.



PROJECT NAME
 PROPOSED PLACE OF WORSHIP &
 ACCESSORY RESIDENCE

PROJECT LOCATION
 81 COLLEGE AVENUE, GUELPH, ON

DRAWING NAME
 BOREHOLE LOCATION PLAN

SCALE	PROJECT NO.
AS SHOWN	GE-01100

DATE	DRAWING NO.
JULY 2025	5



Legend
 ■ Borehole Locations
 ■ Monitoring Well Locations

Location	Northing, m N	Easting, m E	Ground Surface Elevation (m asl)
BH1/MW	4819752.71	561549.53	326.36
BH2	4819778.44	561539.13	327.51
BH3	4819780.83	561500.53	327.91
BH4/MW	4819799.33	561487.93	327.78



LEGEND

WL-322.73 Groundwater Elevation (m asl)
 Inferred Groundwater Contour

SOURCE:

Google Earth Pro, Version 7.3.6.9345,
 Coordinates 17T, 561548 m E, 4819782 m N,
 Imagery date 7/7/2018

NOTE:

Water Levels recorded in the monitoring wells
 on July 18, 2024.



PROJECT NAME

PROPOSED PLACE OF WORSHIP &
 ACCESSORY RESIDENCE

PROJECT LOCATION

81 COLLEGE AVENUE, GUELPH, ON

DRAWING NAME

GROUNDWATER CONTOUR PLAN

SCALE

AS SHOWN

PROJECT NO.

GE-01100

DATE

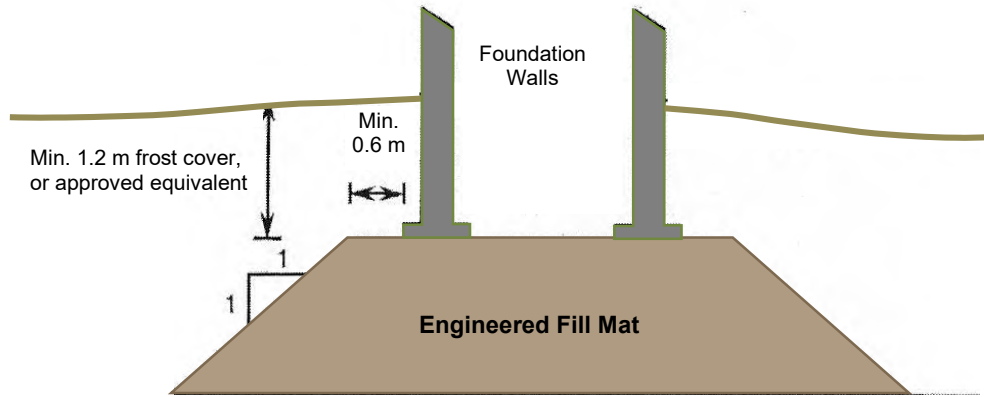
JULY 2025

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.
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.

	PROJECT NAME	PROJECT NO.
	PROPOSED PLACE OF WORSHIP & ACCESSORY RESIDENCE	GE-01100
	PROJECT LOCATION	DRAWING NO.
	81 COLLEGE AVENUE, GUELPH, ON	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 **81 College Ave, Guelph**
 Project Location **81 College Ave, Guelph**
 Project Number **GE-01100**

Borehole ID

1/MW

Sheet 1 of 1

Date Drilled	January 15, 2024	Ground Surface Elevation	326.36 m asl
Drill Rig	D50 Turbo	Groundwater Level at Completion	
Drilling Method	Solid Stem Auger	Technician	Rob Walker
Drilling Contractor	London Soil Test	Checked By	J. May, 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 - 0.5						<u>TOPSOIL</u> - brown, sandy loam, moist, 200 mm	
0.5 - 1.0		1	40	9		<u>SAND</u> - brown, fine grained, some silt, very moist, loose	MC - 20.3%
1.0 - 1.5					1.37 m		
1.5 - 2.0		2	70	4		<u>SANDY SILT</u> - brown, very moist, loose	MC - 19.3%
2.0 - 2.5							
2.5 - 3.0		3	70	6			MC - 23.2%
3.0 - 3.5						- becoming saturated with trace sand observed below 2.9 m depth	
3.5 - 4.0		4	60	4		Gradation: 7% Clay, 91% Silt, 2% Sand, 0% Gravel	MC - 26.9%
4.0 - 4.5							
4.5 - 5.0		5	60	47	4.88 m		MC - 10.0%
5.0 - 5.5					 Feb 24'	<u>SAND AND GRAVEL</u> - brown, moist, dense	
5.5 - 6.0						- becoming compact below 5.6 m depth	
6.0 - 6.5		6	30	17	6.55 m		MC - 8.6%
6.5 - 7.0						Borehole terminated at 6.55 m MW installed to 6.10 m - refer to details below	
7.0 - 7.5							
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
 Depth of Bentonite Seal 2.44 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content

 Water Levels:
 January 29, 2024 - 5.29 m bgs
 February 20, 2024 - 5.16 m bgs



Particle Size Distribution Results of Sieve Analysis

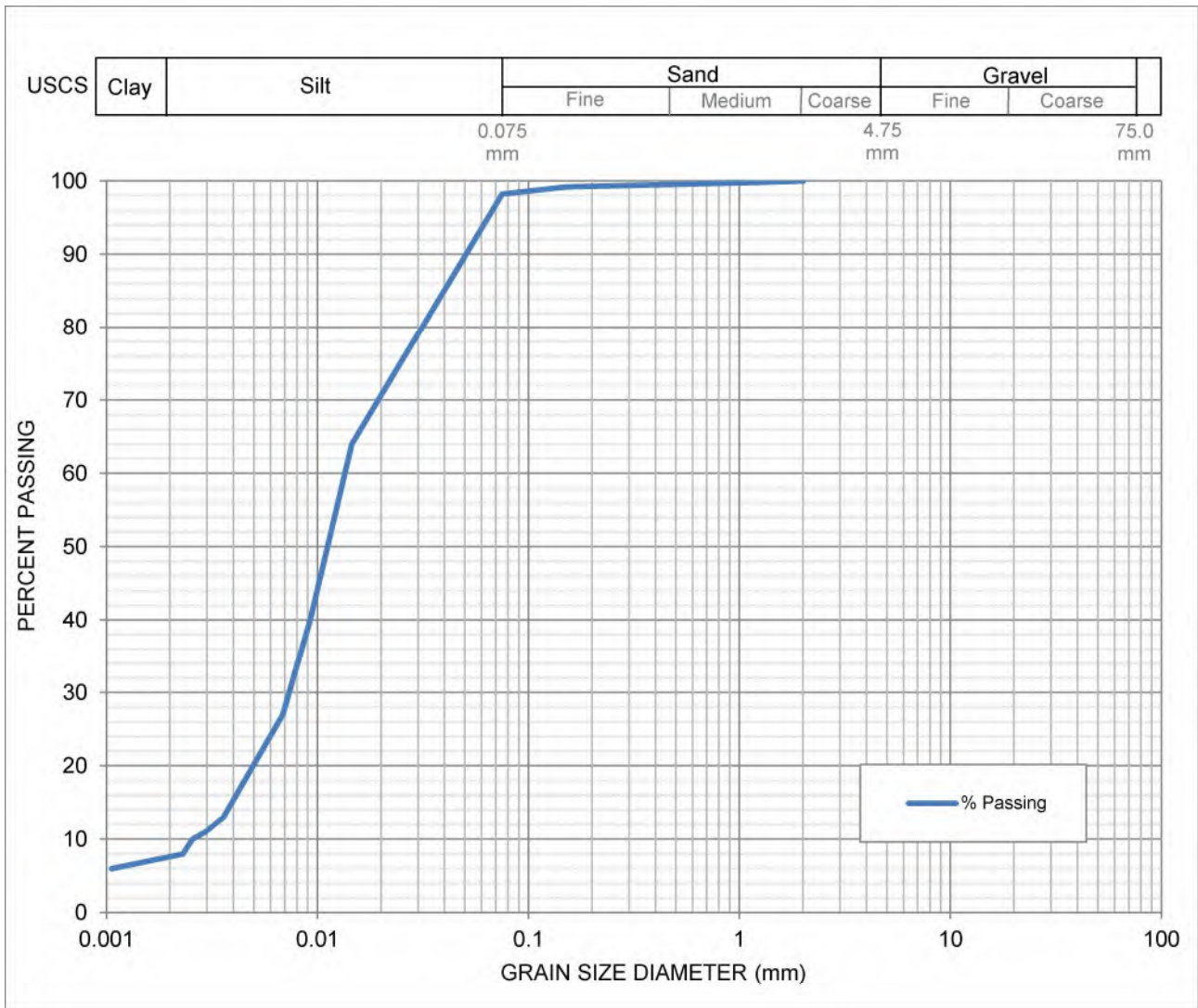
Project Name: Proposed Place of Worship & Accessory Residence

Date: 22-Jan-24

Project Location: 81 College Avenue West, Guelph

Project No.: GE-01100

Sample ID	Unified Soil Classification				Moisture Content (%)
	% Clay	% Silt	% Sand	% Gravel	
BH1SA4 - 3.0 m depth	7.4	90.8	1.8	0.0	26.9





Project	81 College Ave, Guelph	Borehole ID
Project Location	81 College Ave, Guelph	2
Project Number	GE-01100	Sheet 1 of 1

Date Drilled	January 15, 2024	Ground Surface Elevation	327.51 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	J. May, 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 - 0.5						TOPSOIL - brown, sandy loam, moist, 200 mm	
0.5 - 1.0						SAND - brown, fine grained, very moist, compact	
1.0 - 1.5	▲	1	60	11			MC - 14.9%
1.5 - 2.0	▲	2	70	7	1.83 m		MC - 22.6%
2.0 - 2.5					2.13 m	SANDY SILT - brown, wet seams throughout, very moist, loose	
2.5 - 3.0	▲	3	70	20		SAND - brown, fine grained, very moist, compact	MC - 13.4%
3.0 - 3.5					2.90 m		
3.5 - 4.0	▲	4	60	12		SANDY SILT - brown, fine sand layering throughout, very moist, compact	MC - 21.1%
4.0 - 4.5					4.04 m		
4.5 - 5.0	▲	5	30	50**		SAND AND GRAVEL - brown, moist, very dense	MC - 7.6%
5.0 - 5.5						- becoming compact below 5.6 m depth	
5.5 - 6.0							
6.0 - 6.5	▲	6	30	26*	6.55 m	Gradation: 25% Fines (Silt & Clay), 33% Sand, 42% Gravel	MC - 6.3%
6.5 - 7.0						Borehole terminated at 6.55 m Borehole was observed open to 5.18 m at time of completion Borehole was observed dry at time of completion	
7.0 - 7.5							
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
 * - Stone in tip
 ** - 50 blows for 26 mm



Particle Size Distribution Results of Sieve Analysis

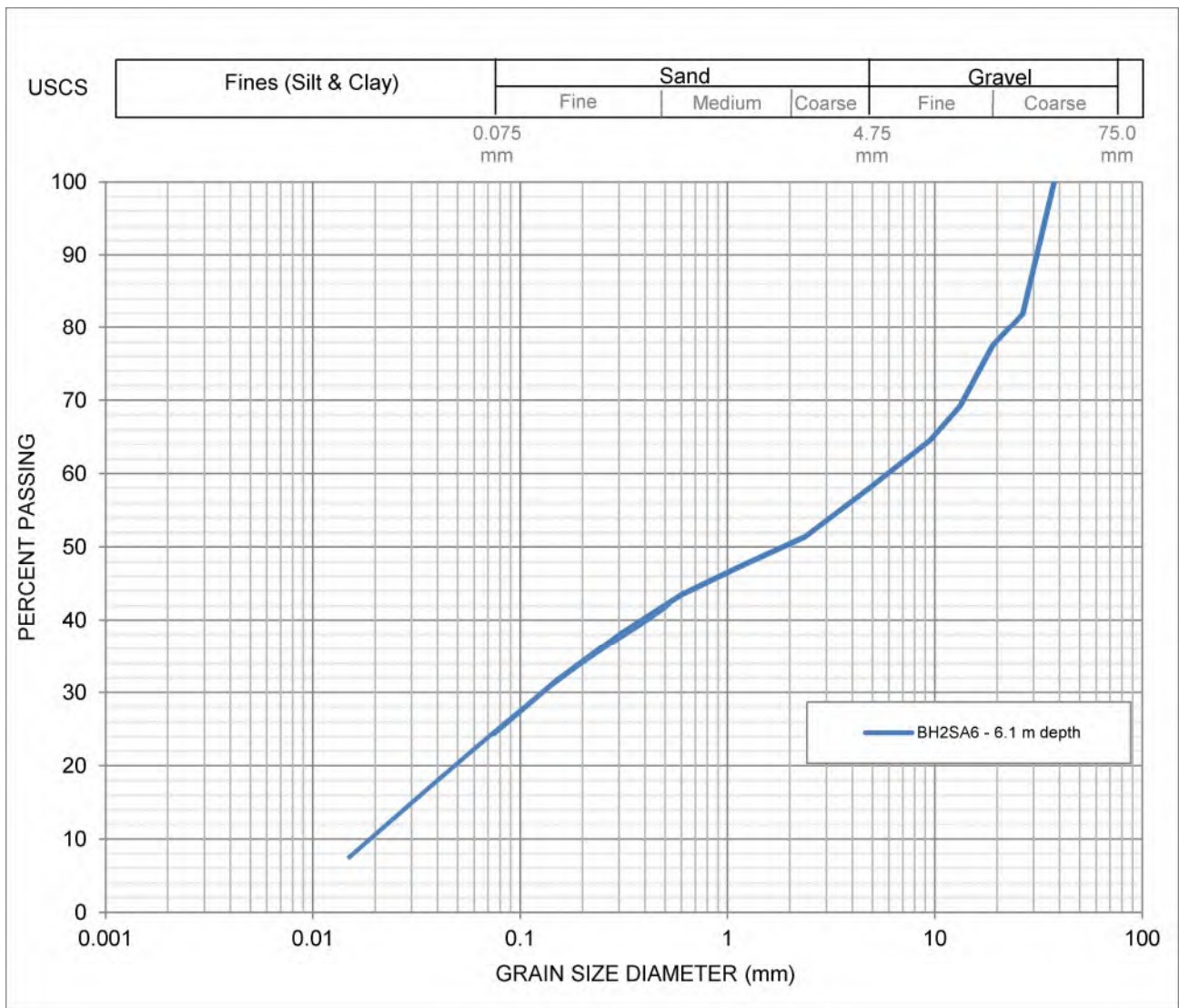
Project Name: Proposed Place of Worship & Accessory Residence

Date: 22-Jan-24

Project Location: 81 College Avenue West, Guelph

Project No.: GE-01100

Sample ID	Unified Soil Classification				Moisture Content (%)
	Fines (Silt & Clay)	% Sand	% Gravel	% Cobbles	
BH2SA6 - 6.1 m depth	24.5%	33.4%	42.1%	0.0%	6.3





Project	81 College Ave, Guelph	Borehole ID
Project Location	81 College Ave, Guelph	3
Project Number	GE-01100	Sheet 1 of 1

Date Drilled	January 15, 2024	Ground Surface Elevation	327.91 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	J. May, EIT

Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT N-value (blows/0.3 m)	Graphic Log	Material Description	Remarks and Other Tests
0.5						TOPSOIL - brown, sandy loam, moist, 150 mm	
1.0		1	60	17		SAND - brown, fine grained, some silt, very moist, compact - becoming loose below 1.4 m depth	MC - 17.3%
1.5		2	70	8	1.98 m		MC - 19.1%
2.0						SANDY SILT - brown, moist, loose - becoming very moist and compact below 2.1 m depth	MC - 17.1%
2.5		3	60	14			MC - 17.2%
3.0		4	60	20		- fine grained, very moist sand layer encountered at 3.4 m	
3.5					3.66 m		
4.0		5	60	19*	4.41 m	SAND - brown, fine grained, some silt, very moist, compact	MC - 14.7%
4.5						SANDY SILT - brown, moist, compact	MC - 6.0%
5.0		6	60	29	5.03 m		
5.5						SAND AND GRAVEL - brown, moist, compact - becoming silty and very moist below 2.1 m depth	MC - 11.0%
6.0		7	70	14	6.55 m		
6.5						Borehole terminated at 6.55 m Borehole was observed open to 5.79 m Borehole was observed dry at time of completion	
7.0							
7.5							
8.0							

Legend SPT Sample Bulk Sample Shelby Tube Stabilized Groundwater Inferred Groundwater	Well Construction Details Pipe Diameter no well installed Installation Depth Screen Length Depth of Bentonite Seal	Additional Notes MC - denotes moisture content * - Stone in tip
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Project **81 College Ave, Guelph**
 Project Location **81 College Ave, Guelph**
 Project Number **GE-01100**

Borehole ID

4/MW

Sheet 1 of 1

Date Drilled	January 15, 2024	Ground Surface Elevation	327.78 m asl
Drill Rig	D50 Turbo	Groundwater Level at Completion	
Drilling Method	Solid Stem Auger	Technician	Rob Walker
Drilling Contractor	London Soil Test	Checked By	J. May, 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 - 0.5						TOPSOIL - brown, sandy loam, moist, 150 mm	
0.5 - 1.0	▲	1	70	9		SAND - brown, fine grained, some silt, moist, loose	MC - 13.4%
1.0 - 1.5						- becoming very moist below 1.4 m depth	
1.5 - 2.0	▲	2	70	8			MC - 18.8%
2.0 - 2.5						SANDY SILT - brown, very moist, loose	
2.5 - 3.0	▲	3	70	9		- becoming compact below 2.9 m depth	MC - 17.5%
3.0 - 3.5							MC - 21.7%
3.5 - 4.0	▲	4	80	15			
4.0 - 4.5							MC - 17.7%
4.5 - 5.0	▲	5	80	18		- becoming saturated with trace sand observed below 4.4 m depth	
5.0 - 5.5						Gradation: 7% Clay, 91% Silt, 1% Sand, 0% Gravel	MC - 19.4%
5.5 - 6.0							
6.0 - 6.5	▲	7	50	50*		SAND AND GRAVEL - brown, moist, very dense	MC - 3.9%
6.5 - 7.0						Borehole terminated at 6.55 m MW installed to 6.10 m - refer to details below	
7.0 - 7.5							
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
 Depth of Bentonite Seal 2.44 m

Well equipped with locking J-Plug cap.

Additional Notes

MC - denotes moisture content
 * - 50 blows for 102 mm
 Water Levels:
 January 29, 2024 - dry
 February 20, 2024 - dry



Particle Size Distribution Results of Sieve Analysis

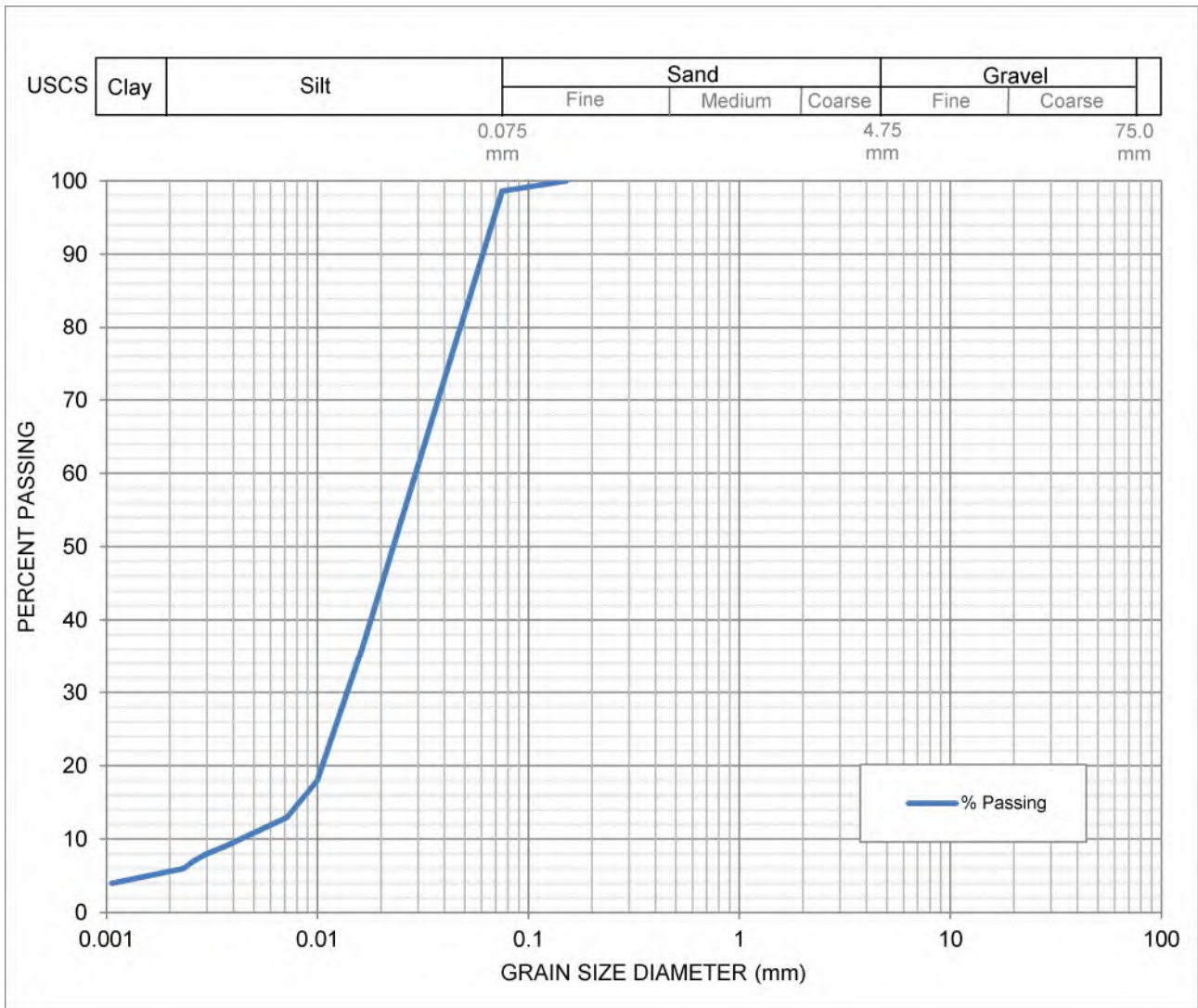
Project Name: Proposed Place of Worship & Accessory Residence

Date: 22-Jan-24

Project Location: 81 College Avenue West, Guelph

Project No.: GE-01100

Sample ID	Unified Soil Classification				Moisture Content (%)
	% Clay	% Silt	% Sand	% Gravel	
BH4SA6 - 4.6 m depth	7.4	91.2	1.4	0.0	19.4



APPENDIX C

MECP WELL RECORD SUMMARY



MECP WATER SUPPLY WELLS

Well	Registration Year	Well Use	Depth of Well (m)	Depth Water Found (m)	Static Water Level (m)	Pump Rate (L/min)
6700857	1947-09-12	Domestic	39.6	39.6	1.5	18.9
6701430	1953-12-11	Domestic	32.9	24.4	8.5	NR
6701431	1950-11-07	Domestic	27.4	10.7	4.6	NR
6701432	1954-01-12	Domestic	26.2	16.8	6.4	NR
6701445	1953-08-11	Domestic	27.4	9.8	3.7	NR
6701446	1953-08-15	Domestic	28.0	26.8	4.6	NR
6701449	1955-07-16	Domestic	26.2	26.2	4.0	NR
6701450	1955-10-31	Domestic	25.9	10.7	4.6	NR
6701451	1956-01-25	Domestic	26.5	26.5	5.2	NR
6701452	1959-09-29	Domestic	14.6	12.2	4.9	56.8
6701453	1960-10-18	Domestic	19.8	15.2	3.7	37.9
6701455	1951-11-30	Domestic	33.5	33.5	3.7	NR
6701457	1952-05-10	Domestic	31.4	31.4	4.3	NR
6701429	1958-09-10	Municipal	57.3	51.2	12.8	NR
6701433	1957-06-18	Municipal	51.8	51.8	17.4	NR
6701435	1963-05-04	Public	75.0	22.9	7.9	NR
6701436	1964-04-18	Public	76.8	22.9	21.9	NR
NR: Not Recorded						

MECP OBSERVATION AND MONITORING WELLS

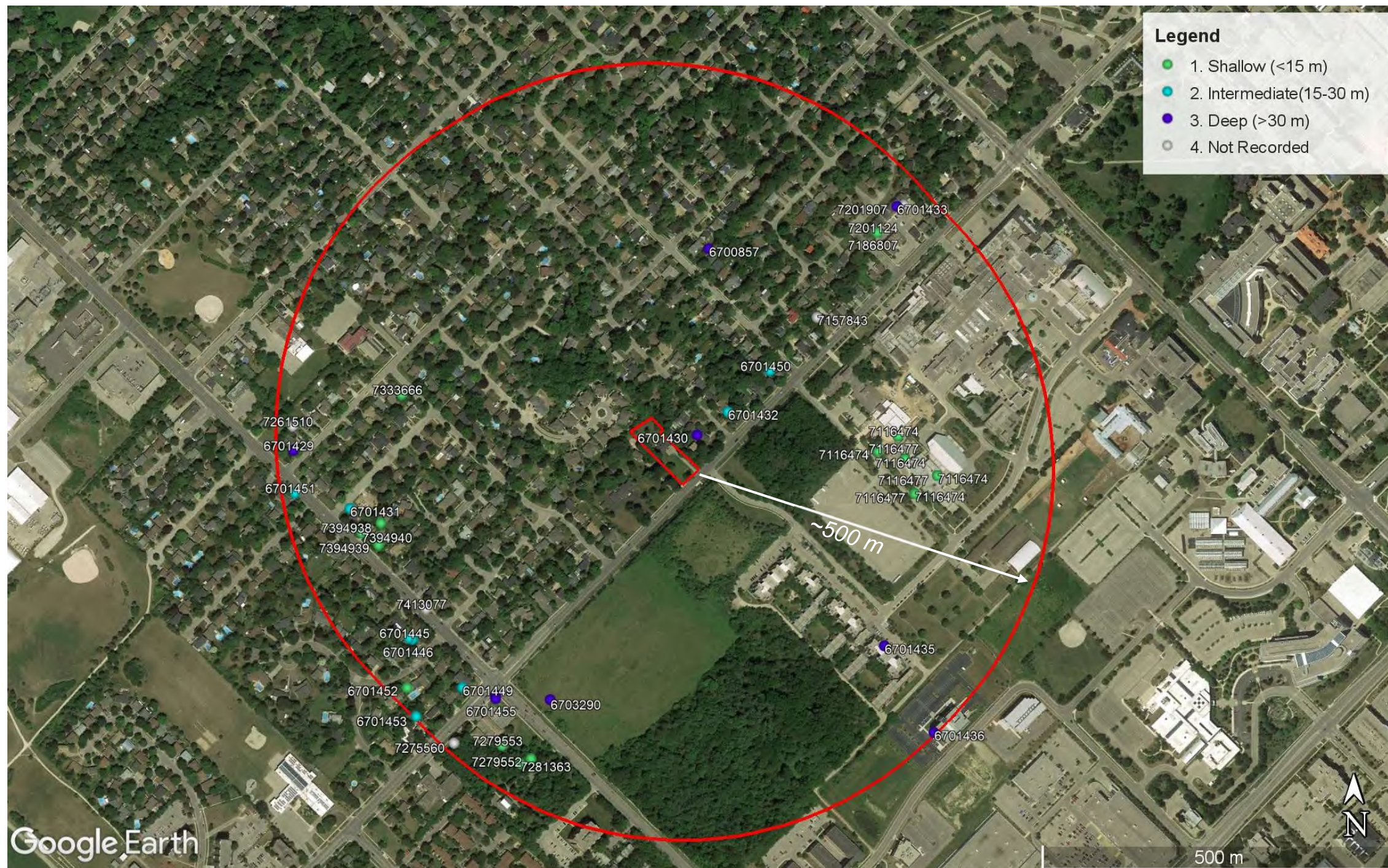
MECP Well ID	Registration Year	Well Type	Depth of Well (m)	Depth Water Found (m)	Static Water Level (m)	Pump Rate (L/min)
7186806	2012-08-07	Monitoring	4.5	NR	NR	NR
7186807	2012-08-07	Monitoring	5.1	NR	NR	NR
7394938	2021-07-13	Monitoring	7.3	NR	NR	NR
7394939	2021-07-13	Monitoring	6.4	NR	NR	NR
7394940	2021-07-13	Monitoring	6.1	NR	NR	NR
NR: Not recorded						



MECP TEST HOLES AND ABANDONMENT RECORDS

MECP Well ID	Registration Year	Well Type	Depth of Well (m)	Depth Water Found (m)	Static Water Level (m)	Pump Rate (L/min)
7157843	2010-10-08	Domestic	NR	NR	NR	NR
7116477	2008-09-04	Not Used	NR	4.0	NR	NR
7116477	2008-09-04	Not Used	NR	4.0	NR	NR
7116477	2008-09-04	Not Used	NR	4.0	NR	NR
7116477	2008-09-04	Not Used	NR	4.0	NR	NR
7116477	2008-09-04	Not Used	NR	4.0	NR	NR
7275560	2016-10-11	NR	NR	NR	NR	NR
7201907	2013-05-01	Municipal	NR	NR	NR	NR
7333666	2019-03-11	Monitoring	6.0	NR	NR	NR
7279552	2017-01-05	Monitoring and Test Hole	10.4	NR	NR	NR
7279553	2017-01-05	Monitoring and Test Hole	8.8	NR	NR	NR
7281363	2017-01-03	Test Hole	9.1	NR	NR	NR
7261510	2016-04-06	Municipal	NR	NR	NR	NR
7201123	2013-04-26	NR	NR	NR	NR	NR
7201124	2013-04-26	NR	NR	NR	NR	NR
7413077	2022-01-24	NR	NR	NR	NR	NR
7116474	2008-08-25	Monitoring	6.0	4.0	4.0	NR
7116474	2008-08-26	Monitoring	6.0	4.0	4.0	NR
7116474	2008-08-26	Monitoring	6.0	4.0	4.0	NR
7116474	2008-08-26	Monitoring	6.0	4.0	4.0	NR
7116474	2008-08-25	Monitoring	6.0	4.0	4.0	NR
6703290	1968-06-04	Not Used	80.8	39.6	10.7	NR
NR: Not recorded						





SOURCE:
Google Earth Pro, Version 7.3.6.9345,
Coordinates 17T, 561522 m E, 4819773 m N,
Imagery date 7/7/2018



PROJECT NAME
PROPOSED PLACE OF WORSHIP & ACCESSORY
RESIDENCE

PROJECT LOCATION
81 COLLEGE AVENUE WEST, GUELPH, ON

DRAWING NAME
MECP WELL LOCATIONS – ALL WELLS

SCALE AS SHOWN	PROJECT NO. GE-01098
DATE JULY 2025	DRAWING NO. C1



SOURCE:
Google Earth Pro, Version 7.3.6.9345,
Coordinates 17T, 561522 m E, 4819773 m N,
Imagery date 7/7/2018



PROJECT NAME
PROPOSED PLACE OF WORSHIP & ACCESSORY
RESIDENCE

PROJECT LOCATION
81 COLLEGE AVENUE WEST, GUELPH, ON

DRAWING NAME
MECP WELL LOCATIONS – WATER SUPPLY WELLS

SCALE AS SHOWN	PROJECT NO. GE-01100
DATE JULY 2025	DRAWING NO. C2

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