



Geotechnical Report Proposed Residential Development

51-55 Hawkesville Road, St. Jacobs, Ontario

Submitted to:

Polocorp Inc.
379 Queen Street South
Kitchener, ON
N2G 1W6

Submitted by:

GEI Consultants Canada Ltd.
650 Woodlawn Road West, Block C, Unit 2
Guelph, ON N1K 1B8
<https://canada.geiconsultants.com/>

March 31, 2026
Project No. 2403070

Table of Contents

1.	Introduction	1
2.	Procedures and Methodology	2
3.	Subsurface Conditions	3
3.1	General Overview	3
3.2	Stratigraphy	3
3.3	Groundwater Levels	5
3.4	Corrosivity	5
4.	Engineering Design Parameters & Analysis	7
4.1	Site Grading	7
4.2	Existing Fill	8
4.3	Foundations	8
4.4	Basement Wall Earth Pressure Design Parameters	11
4.5	Slab-on-Grade Design	12
4.6	Basement Drainage	13
4.7	Site Servicing	13
4.8	Pavement Design	14
5.	Constructability Considerations	16
5.1	Excavations	16
5.2	Temporary Construction Groundwater Control	17
5.3	Stormwater Management Pond	18
5.4	Compaction Specifications	19
5.5	Quality Verification Services	19
6.	Limitations and Conclusions	21
6.1	Limitations	21
6.2	Conclusion	22

Figures

1. Site Location Plan
2. Borehole Location Plans
 - A. Aerial
 - B. Site Plan



Table of Contents (cont.)

Appendices

- A. Borehole Logs and Geotechnical Laboratory Data - GEI
- B. Borehole Log - CVD
- C. Typical Details
- D. Site Photographs
- E. Site Plan



1. Introduction

GEI Consultants Canada Ltd. (GEI) was retained by Polocorp Inc. (Client) to complete a geotechnical investigation and report for the proposed residential development at 51-55 Hawkesville Road, St. Jacobs, Ontario (site).

The site is bounded by Hawkesville Road to the north, existing residential properties to the east, Conestogo River to the south, and a Waterloo-Spur Line rail corridor to the west. There are currently three (3) buildings at the site and an existing pond near the northwest corner of the site which is mapped as a wetland. The southern portion of the site includes a forested area and steep slope and is located within the floodplain of the Conestogo River. Portions of the site are located within an area regulated by the Grand River Conservation Authority (GRCA) under O.Reg. 150/06. Photographs of the site are provided in Appendix D. An aerial photograph of the site is provided Figure 2A.

Based on the site plan dated December 11, 2024, the proposed development will consist of 151 residential dwelling units comprising 17 single detached dwelling units and 134 townhouse type dwelling units. The proposed development will also feature amenity areas, a stormwater management facility, and a sewage pumping station. Internal roadways, watermain and sewers will be private as the proposed development will be a condominium development. The site plan is included in Appendix E for reference.

The purpose of the geotechnical investigation is to assess the subsurface soil conditions at the site, and based on this information, provide borehole logs, a borehole location plan, a written description of the subsurface conditions, and geotechnical engineering recommendations to support the design of the proposed development. This report only addresses the geotechnical design of the proposed development. Hydrogeological recommendations are provided under separate cover. A stability assessment of the slope near the south side of the property is also provided separately.

If there are any changes to the proposed development, GEI should be contacted to review that the recommendations provided in this report are still valid.



2. Procedures and Methodology

It is noted that all elevations in this report are metric/geodetic and expressed in metres (m). All measurements are also in metric and expressed in millimetres (mm), metres (m) or kilometres (km).

Prior to the commencement of drilling activities, the borehole locations were staked in the field by GEI. Ground surface elevations of the boreholes and horizontal coordinates (referencing NAD 83 geodetic datum) were surveyed by GEI. The boreholes were cleared of underground utilities prior to drilling.

The fieldwork for the drilling program was carried out on April 23, 2024, at which time six (6) boreholes were drilled at the site. The boreholes were advanced to a depth of 5.2 m below existing grade except for Boreholes 24-01 and 24-02, which were advanced to 9.8 m and 12.8 m depth, respectively. Borehole logs are provided in Appendix A and the borehole locations are shown on the Borehole Locations Plans in Figures 2A and 2B. Additionally, Polocorp provided GEI with a borehole log for a borehole previously at the site by Chung Vander Doelen (CVD) in 2021. The CVD borehole log is included in Appendix B.

The boreholes were advanced by a drilling subcontractor retained and supervised by GEI using a track-mounted drill rig, hollow stem augers, and standard soil sampling equipment. Sampling was conducted using a 51 mm O.D. Split Spoon (SS) sampler. Standard Penetration Test (SPT) “N” Values (N values) were recorded for the sampled intervals as the number of blows required to drive an SS sampler 305 mm into the soil using a 63.5 kg drop hammer falling 750 mm, in accordance with ASTM D1586. In each borehole soil sampling was conducted at 0.75 m intervals for the upper 3.0 m and at 1.5 m intervals thereafter.

Six (6) monitoring wells were installed (i.e. one in each borehole) to permit long-term groundwater monitoring and hydrogeological testing. Each monitoring well consisted of a 50 mm diameter PVC pipe with a 3.0 m long screen. The screen was surrounded with a sand filter pack and the annulus surrounding the pipe was backfilled to surface with bentonite pellets. Monitoring well construction is shown on the borehole logs in Appendix A.

The GEI field staff examined, and classified characteristics of the soils encountered in the boreholes, including the presence of fill materials, groundwater observations during and upon completion of the drilling, recorded observations of borehole construction, and processed the recovered samples. All recovered soil samples were logged in the field, carefully packaged, and transported to GEI’s laboratory for more detailed examination and classification.

In GEI’s laboratory, the samples were classified based on their visual and textural characteristics. Five (5) soil samples were selected from the boreholes and submitted to our laboratory for grain size analysis. Two (2) soil samples were subjected to Atterberg Limits



testing to assess plasticity. The grain size and Atterberg limits testing results are provided in Appendix A.

Three (3) samples were also submitted to an external laboratory for corrosivity parameters.

3. Subsurface Conditions

3.1 General Overview

Borehole logs and the geotechnical laboratory results from the GEI investigation are provided in Appendix A. The borehole locations are shown on Figures 2A and 2B. The borehole log from the CVD investigation is included in Appendix B.

It should be noted that the conditions indicated on the borehole logs are for specific locations only and can vary between and beyond the locations. It should be noted that the soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones and should not be interpreted as exact planes of geological change.

In addition, the descriptions provided in the borehole logs are inferred from a variety of factors, including visual observations of the soil samples retrieved, laboratory testing, measurements prior to and after drilling, and the drilling process itself (speed of drilling, shaking/grinding of the augers, etc.). The passage of time also may result in changes in conditions interpreted to exist at locations where sampling was conducted.

3.2 Stratigraphy

The following sections provide a description of the stratigraphy encountered in boreholes completed as part of the GEI investigation.

3.2.1 Topsoil

Topsoil was encountered at the ground surface in all boreholes. The thickness of the topsoil ranged from 305 to 455 mm. Topsoil thicknesses measured at the borehole locations may not be representative of the topsoil depth across the site.

3.2.2 Fill

Fill was encountered in Borehole 24-01 underlying the topsoil. The fill was generally comprised of sand with varying amounts of silt and gravel and cobbles. Some trace organics and/or rootlets were noted in the upper portion of the fill.



In Borehole 24-01, the fill was 8.7 m thick and extended to a depth of 9.1 m below existing ground surface (Elev. 329.1). The fill was typically moist however some recovered samples of the fill were wet. Measured moisture contents ranged from 10% to 27%. SPT “N” values in the fill ranged from 3 to 50+ blows per 0.3 m of penetration, indicating a variable relative density (very loose to very dense).

3.2.3 Silty Sand to Sand and Silt

A deposit of silty sand was encountered in Borehole 24-02. The silty sand generally contained trace amounts of clay and gravel. This layer was 1.9 m thick and extended to a depth of 2.3 m below existing ground surface (Elev. 336.9). Moisture contents ranged from 13% to 17%. SPT “N” values were 5 to 6 blows per 0.3 m of penetration, indicating a loose condition.

3.2.4 Silty Clay

A layer of silty clay was encountered in Boreholes 24-03 and 24-05. The silty clay contained some sand and trace gravel. In Borehole 24-03, the silty clay was 2.0 m thick and extended to a depth of 2.3 (Elev. 337.8). In Borehole 24-05, the silty clay was 0.4 m thick and extended to 0.7 m depth (Elev. 337.8). The silty clay was moist to dry in Borehole 23-03 and wet in Borehole 24-05. Measured moisture contents ranged from 9% to 34%. SPT “N” values measured in the silty clay ranged from 3 to 14 blows per 0.3 m of penetration, indicating a soft to stiff consistency.

One (1) sample of the silty clay was submitted for grain size analysis and the results are included in Figure A1 in Appendix A. One (1) sample of the silty clay was submitted for Atterberg testing and the results are provided in Figure A3 in Appendix A. The results of the Atterberg limits testing indicate the silty clay to have high plasticity (CH).

3.2.5 Glacial Till

A deposit of sandy silt to silty sand glacial till was encountered underlying the fill in Borehole 24-01, underlying the silty sand in Borehole 24-02, below the silty clay in Borehole 24-03 and Borehole 24-05, and below the topsoil in Boreholes 24-04 and 24-06. The glacial till generally contained trace to some gravel and trace amounts of clay, although some plastic zones were encountered locally within the till. Glacial till inherently contains cobbles and boulders.

All of the boreholes were terminated in the glacial till at depths ranging from 5.2 m to 12.8 m below ground surface (Elev. 341.2 to 326.4). Moisture contents ranged from 7% to 20% and SPT “N” values ranged from 2 to greater than 50 blows per 0.3 m of penetration. However, the glacial till was typically compact to very dense.

Four (4) samples of the glacial till were collected and submitted for grain size analysis. The results of the testing are presented in Figure A2 in Appendix A. Two (2) samples of the glacial



till exhibiting plasticity were submitted for Atterberg testing and the results are provided in Figure A4 in Appendix A. The results of the Atterberg limits testing indicate the plastic zones within the glacial till to have slight to low plasticity (ML to CL-ML).

3.3 Groundwater Levels

Unstabilized groundwater level measurements were taken upon the completion of drilling and six (6) boreholes were outfitted with a monitoring well to permit measurement of stabilized groundwater level. The monitoring wells consisted of 50 mm diameter PVC standpipes and 3.0 m long screens. Monitoring well configuration and groundwater observations are noted on the borehole logs in Appendix A. A summary of the groundwater level measurements is provided in the following table.

Borehole	Ground Surface Elevation (m)	Depth of Cave (m)	Un-Stabilized Groundwater Level in Open Borehole	Stabilized Groundwater Level in Monitoring Well on May 3, 2024	Stabilized Groundwater Level in Monitoring Well on May 13, 2024
			Depth / Elevation (m)		
24-01	338.2	Open	Dry	3.6 / 334.6	4.3 / 333.9
24-02	339.2	Open	8.5 / 330.7	4.2 / 335.0	4.2 / 335.0
24-03	340.1	Open	Dry	0.9 / 339.2	1.1 / 339.0
24-04	346.4	Open	Dry	4.0 / 342.4	3.6 / 342.8
24-05	338.6	Open	Dry	0.2 / 338.4	0.3 / 338.3
24-06	345.1	Open	Dry	3.2 / 341.9	2.3 / 342.7

The stabilized groundwater levels measured to date in the monitoring wells were 0.2 to 4.3 m below existing ground surface (Elev. 342.8 to 333.9). It should be noted that GEI is currently carrying out a long-term groundwater monitoring program at the site as part of the hydrogeological investigation.

Groundwater levels are expected to show seasonal fluctuations and vary in response to prevailing climate conditions.

3.4 Corrosivity

Select soil samples from Boreholes 24-02, 24-03, and 24-04 were submitted for analytical testing of corrosivity parameters and sulphate. The laboratory certificates of analysis are provided in Appendix A and summarized below.



Parameter	Units	Test Results		
		24-04, SS2 Depth = 0.8 to 1.4 m	24-02, SS2 Depth = 0.8 to 1.4 m	24-03, SS2 Depth = 0.8 to 1.4 m
		Glacial Till	Silty Sand	Silty Clay
Sulphate	%	<0.01	<0.01	<0.01
Electrical Conductivity	mS/cm	0.12	0.14	0.13
pH	-	7.96	7.93	8.02
Resistivity	ohm-cm	8333	7143	7692
Chloride	%	0.003	<0.002	<0.002

The American Water Works Association (AWWA) has developed a scoring system in their C-105 Standard to determine the potential for soil to corrode cast iron alloys. Based on the AWWA scoring system, all samples subjected to corrosivity testing scored less than 10 points which indicates a low potential for the soil corrosivity to affect cast iron alloys and corrosion protection measures are therefore not recommended for the site.

It should be noted that this ranking system provides a general indication of soil corrosivity potential only. Other external factors may increase corrosivity potential, such as stray electrical currents or the application of de-icing salts in the winter that can leach into the ground.

The samples were also tested for water-soluble sulphate content and compared to *Table 3: Additional Requirements for Concrete Subjected to Sulphate Attack* as found in CSA A23.1 *Concrete Materials and Methods of Construction*. Table 3 is used to determine the cementing materials required in concrete based on degree of exposure to sulphate attack. The testing shows the samples have less than 0.1% water-soluble sulphate, and therefore there will have negligible exposure to sulphate attack, as such no additives are required.

4. Engineering Design Parameters & Analysis

Based on the site plan (Appendix E), it is understood that the existing buildings at the site will be demolished, the pond will be filled in, and a total of 151 residential dwelling units comprising 17 single detached dwelling units and 134 townhouse type dwelling units will be constructed. A private pumping station and stormwater management (SWM) facility will also be constructed near the southwest corner of the site.

If there are any changes to the proposed development, GEI should be contacted to review that the recommendations provided in this report are still valid.

4.1 Site Grading

Detailed grading plans were not available for review at the time of this report. However, based on the Sanitary Sewer Servicing Strategy letter prepared by GM BluePlan, dated February 5, 2024, it is envisaged that the grades at the site will likely be raised as part of the proposed development. When final grading is established, GEI should review the drawings for geotechnical requirements.

The existing topsoil, fill, loose silty sand, and soft to firm silty clay are not suitable to support the foundations. These materials should be removed and replaced with engineered fill. Footings could then be placed at nominal depth and be founded on engineered fill or native soil at depth and the floor slab could be supported by engineered fill. It is envisioned that the engineered fill/earthworks take place first then excavation for building addition footings would occur.

4.1.1 Engineered Fill

GEI defines “engineered fill” as material that will support foundations, and which is placed and compacted in a specified and controlled manner under full-time supervision of geotechnical engineering staff.

In any location where engineered fill will be placed to raise grades or replace poor/weak soil, any topsoil, fill, vegetation, wet/saturated/weathered soil must be fully removed down to competent native soil. The exposed subgrade soil must be proof-rolled and inspected by the geotechnical engineer to ensure all unsuitable material (e.g., organics, weak or loose/soft soil, weathered / disturbed soil, deleterious materials) is removed from the engineered fill footprint. Particular attention should be given to existing pond area where the soils may be particularly soft/wet when the removal of sediment is taking place after pond draining. Also, the existing fill associated with the building foundations must also be removed both vertically and laterally when the buildings are demolished. Any unsuitable areas must be further sub-excavated and replaced with approved material and compacted to a targeted 100% Standard Proctor



maximum dry density (SPMDD) in building areas and 95% SPMDD in pavement and servicing areas.

Once the subgrade is approved, engineered fill can be placed. Engineered fill must be placed under the full-time supervision of a geotechnical engineer as required in the Ontario Building Code. The engineered fill may consist of excavated on-site cohesionless soils provided they have been moisture conditioned to a moisture content within 2% of optimum moisture content (OMC) and do not contain organics, topsoil or deleterious material. It should be noted that it may be impractical to reuse existing native soils excavated from below the groundwater table as they may be too wet to achieve compaction. Moisture can be reduced by tilling or spreading out the soil to dry or blending it with drier material however if compaction cannot be achieved it will need to be replaced with approved imported soil. It is recommended that any imported soil consist of Granular B (OPSS.MUNI 1010) and be first used in building areas, with suitable on-site soil used in landscaped or road areas. Engineered fill must be placed in loose lifts of 200 mm or less and compacted as noted above.

The engineered fill must extend a minimum of 1 m out from all sides of the foundations and extend at a 1 horizontal to 1 vertical slope (1H:1V) down to the exposed subgrade. A typical detail for engineered fill pad dimensioning is included in Appendix C.

4.2 Existing Fill

Borehole 24-01, advanced near the southwest corner of the site, encountered a thick layer of sand fill containing varying amounts of silt and gravel, cobbles, and organics. The fill extended to a depth of 9.1 m (Elev. 329.1) in the borehole and was loose to very dense. It is envisaged that the fill was placed as part of previous grading activities at the site. The thickness and composition of the fill will vary across the site.

A SWM facility, pumping station, and townhouse buildings are currently proposed to be constructed within the area of existing fill. It must be noted that no structures, ponds, etc. may be founded in the existing fill given that this fill was placed in an uncontrolled manner and structures founded within the fill would be at risk of settlement and/or instability. Consequently, this fill will need to be removed entirely and replaced with engineered fill prior to construction of any structures or ponds within this area. Additional geotechnical investigation is recommended in the area of the existing fill.

4.3 Foundations

4.3.1 Footings on Native Soil

Foundations at this site can be constructed as conventional shallow spread and strip footing foundations that bear on the undisturbed native compact to very dense sandy silt to silty sand



glacial till. Foundations should not be supported on the topsoil, fill, loose silty sand, and soft to firm silty clay.

The depth and elevation where compact to dense glacial till was encountered in the boreholes is provided in the table below. Footings founded on undisturbed compact to dense glacial till may be designed using a geotechnical reaction at Serviceability Limit State (SLS) of 150 kPa for 25 mm of settlement and a factored geotechnical resistance at Ultimate Limit State (ULS) of 225 kPa.

Borehole	Level of <i>Compact to Dense Glacial Till in Borehole</i>	
	Depth (m)	Elevation (m)
24-01	N/A	N/A
24-02	2.3	336.9
24-03	2.3	337.8
24-04	0.5	345.9
24-05	2.3	336.6
24-06	1.5	343.6

Final footing elevations must be reviewed by geotechnical personnel from GEI to confirm bearing capacity values. The final site configuration must also be reviewed by GEI to assess the potential for footings to be founded on different soil subgrades, and to assess the potential for differential settlement.

4.3.2 Footings on Engineered Fill

As an alternative to foundations made on undisturbed native glacial till, spread and strip footings can be supported on engineered fill supported on the undisturbed native compact to very dense sandy silt to silty sand glacial till.

Engineered fill must be placed and compacted in accordance with Section 4.1. Foundations supported on engineered fill may be designed using a geotechnical reaction at SLS of 100 kPa for 25 mm of settlement and a factored geotechnical resistance at ULS of 150 kPa. The engineered fill must extend a minimum of 1 m out from all sides of the foundations and extended at a 1H:1V slope down to the exposed subgrade. A typical detail for engineered fill pad dimensioning is included in Appendix C.

It is recommended that nominal reinforcing steel for stiffening of the foundation walls made on engineered fill be provided to help mitigate minor cracking due to minor differential settlement. The reinforcing steel in the poured concrete foundation walls may consist of 2-15M

bars continuous at the top of the foundation wall, and 2-15M bars continuous at the bottom of the foundation walls. Typically, these bars are placed 100 to 200 mm from the top or bottom of the foundation wall, respectively. The reinforcing steel should extend a minimum of 3 m past any transition zones between engineered fill and native soil. A typical reinforcing steel detail for foundation walls placed on engineered fill is provided within Appendix C. The recommended nominal reinforcing steel should not be considered a structural design. The need for different or additional reinforcement should be reviewed by a structural engineer to ensure the original structural design intent of the structure is maintained.

4.3.3 Pumping Station

The site plan indicates that a pumping station is proposed near the southwest corner of the site. Few details were available for the proposed pumping station at the time of this report preparation. However, based on Figure 6 included in the Sanitary Sewer Servicing Strategy letter prepared by GM BluePlan, dated February 5, 2024, it is understood that the proposed grade at the pumping station will be at Elevation 341.48 m and that the invert at the pumping station will be at Elevation 338.36 (i.e. pumping station will be founded approximately 3.1 m below final grade).

The location of the pumping station coincides with a thick layer of fill. It must be noted that no structures may be founded in the existing fill given that this fill was placed in an uncontrolled manner and structures founded within the fill would be at risk of settlement and/or instability. Consequently, this fill will need to be removed entirely and replaced with engineered fill prior to construction of the pumping station within this area.

Once the existing fill is removed and replaced with engineered fill then the pumping station may be supported on conventional spread footings founded within the engineered fill. Additional geotechnical investigation should be carried out to support design of the pumping station once the final location of the pumping station is known.

4.3.4 General Foundation Considerations

The recommended geotechnical resistances provided in Sections 4.3.1 and 4.3.2 are for vertical, concentric loading. Where eccentric or inclined loads are applied, the effects of load inclination and eccentricity must be considered.

The minimum strip and spread footing widths to be used shall be dictated as per the Ontario Building Code, regardless of loading considerations. Footings stepped from one level to another must be at a slope not exceeding 7 vertical to 10 horizontal. This concept should also be applied to excavations for new foundations in relation to existing footings or underground services unless rigid shoring is provided. All footings exposed to ambient air temperature throughout the year must be provided with a minimum of 1.2 metres of earth cover or equivalent insulation for frost protection. A 25 mm thick polystyrene insulation board is equivalent to 300 mm of soil cover.



The foundation design parameters provided above are predicated on the assumption that the foundation subgrade surface is undisturbed, and that all deleterious, softened, disturbed, organic, and caved material is removed. The foundation excavation must be done in such a way that groundwater is controlled to prevent any disturbance to the foundation base. Temporary groundwater control during construction is discussed in Section 5.2.

The foundation subgrade must be inspected and approved by a geotechnical engineer prior to the placement of any fill or concrete to verify the foundation design parameters provided are applicable, and to provide remedial recommendations if necessary. If foundation excavations will be open for a prolonged period of time, the foundation subgrade should be protected with a skim coat of lean mix concrete (after inspection and approval by the geotechnical engineer), to ensure that no deterioration will occur due to weather effects.

Should construction extend to the winter season, particular attention should be given to prevent freezing of the exposed subgrade and ensure frozen material is not used as backfill. The building foundations must not be placed on frozen ground.

4.4 Basement Wall Earth Pressure Design Parameters

Backfill to the structure walls may consist of the on-site glacial till provided it is free of organics and deleterious materials. Oversize particles (cobbles and boulders) larger than 100 mm should be removed and discarded from the backfill. Alternatively, imported granular material (e.g. Granular 'B' Type I) may be used to backfill the basement walls.

Basement walls must be designed to resist unbalanced lateral earth pressures imparted from the weight of adjacent soils. For a fully drained condition, lateral earth pressures are calculated using the following equation:

$$P = K[\gamma h + q]$$

- where, **P** = the horizontal pressure at depth, **h** (m)
K = the earth pressure coefficient (dimensionless)
h = depth below surface in metres
γ = the bulk unit weight of soil, (kN/m³)
q = surcharge loading (kPa)

The above equation assumes that a drainage system is present which prevents the build up of any hydrostatic pressure behind the structure subjected to the unbalanced lateral earth pressures. If this is not the case, the equation must be revised to also incorporate the submerged unit weight of the soil multiplied by the earth pressure coefficient, in addition to the water pressure itself.

The values for use in the design of basements subjected to unbalanced lateral earth pressures are as follows:



Soil Type	γ – Bulk Unit Weight (kN/m ³)	ϕ – Friction Angle (degrees)	Earth Pressure Coefficient (dimensionless)		
			K_a – Active	K_o – At-Rest	K_p – Passive
Granular 'B' (OPSS.MUNI 1010)	21.0	32	0.31	0.47	3.25
Compacted Glacial Till	21.0	30	0.33	0.50	3.00

The calculation of the earth pressure coefficients is based on Rankine theory, which provides a conservative estimate as no friction between the soil and the structure is accounted for. The earth pressure coefficients provided above are only applicable for flat ground surfaces beyond the structure and will change for sloping ground surfaces.

The earth pressure coefficients referenced within the above table are a function of the friction angle of the adjacent soil, and both the degree and direction of movement of the structure subjected to unbalanced lateral earth pressures. For structures that are restrained at the top (such as basement walls), the at-rest earth pressure coefficient will apply.

4.5 Slab-on-Grade Design

Inorganic native soils and engineered fill are suitable to support floor slabs. Topsoil, vegetation, organics, peat, organic sands, in-situ earth fill and other soil containing organics, excessive moisture, or deleterious materials are not suitable to support floor slabs.

The exposed subgrade must be proof-rolled and inspected by the geotechnical engineer. If any soft or weak subgrade areas are identified, or if there are areas containing excessive amounts of deleterious/organic material, they must be locally sub-excavated and backfilled with approved clean earth fill or imported granular material and compacted to a minimum of 98% SPMDD within 2% of OMC.

All building floor slabs must be provided with a capillary moisture barrier and drainage layer. This is made by placing the concrete slab on a minimum 200 mm layer of 19 mm clear stone (OPSS.MUNI 1004) compacted by vibration to a dense state. The upper 50 mm of clear stone can be replaced with 19 mm crusher run limestone for a working surface. Granular A compacted to 100% SPMDD can also be utilized.

For new structures that will be slab-on-grade with no basement levels, perimeter and under-slab drainage at the foundation level is not required, provided that the underside of concrete slab is at least 200 mm above the prevailing grade of the site and the surrounding surfaces slope away from the building at a gradient of at least 2% to promote surface water run-off and to reduce groundwater infiltration adjacent to foundations. To minimize infiltration of surface water, the upper 150 mm of backfill could consist of less permeable, compacted clayey soil.

4.6 Basement Drainage

Where basements are constructed, all basement foundation walls must be provided with damp-proofing provisions in conformance to the Ontario Building Code. Backfill to the basement walls must consist of Granular 'B' Type I (OPSS.MUNI 1010) for a minimum lateral distance of 600 mm out from the foundation wall. Alternatively, if a filtered cellular drainage media is provided adjacent to the foundation wall, the backfill may consist of common earth fill.

For buildings with basements, a perimeter drainage system must be installed that will remove any water that infiltrates into the building backfill, to ensure that any water does not infiltrate into the basement. The perimeter drains must consist of minimum 100 mm diameter perforated pipes wrapped in filter socks, sufficiently covered on all sides by 19 mm clear stone. Perimeter drains should be directed to the sump underneath the basement floor in solid pipes so as not to surcharge the underfloor drainage layer with water. Two runs of perforated subfloor drainage pipe, surrounded by 19 mm clear stone, surrounded by a geotextile such as Terrafix 270R (or approved equivalent) to prevent the migration of fines, and trenched into the subgrade should also be installed beneath the slab-on-grade of each building along the long axis of the building. A typical basement drainage detail is included in Appendix C.

It is recommended that basements be established a minimum of 0.6 m above the seasonal high groundwater table. If the basement level is set near or within the prevailing groundwater level, it is possible that perimeter drainage issues may occur in the future (e.g. sump pump failure, blockage of drainage pipes, etc.), which may lead to basement flooding.

Once basement floor elevations are known for the townhouses, they should be compared to seasonal high groundwater levels across the site.

4.7 Site Servicing

4.7.1 Subgrade and Bedding

Following subexcavation to the piping subgrade level, any topsoil/organics, loose/soft or wet, and disturbed materials should be subexcavated and replaced. These materials should be removed and replaced with engineered fill. The entire base of the excavation should be inspected and approved by a geotechnical engineer. Should construction extend to the winter season, particular attention should be given to prevent freezing of the exposed subgrade and ensure frozen material is not used as backfill.

The pipes must be placed on a uniformly competent subgrade. Any water within the excavation must be removed prior to pipe placement. Pipe bedding materials, compaction and cover should follow OPSD 802.010 or 802.013 for flexible pipes, and OPSD 802.030 to 802.033 for rigid pipes. Oversize particles (cobbles and boulders) larger than 100 mm should be removed and discarded.



Where disturbance of the trench base has occurred from ground water seepage, construction traffic, etc., the disturbed soils should be sub excavated and replaced with suitably compacted granular material.

Regardless of whether flexible or rigid pipes are implemented, granular bedding and cover material should consist of a well graded, free draining material, such as Granular “A” (OPSS.MUNI 1010). All granular bedding must be compacted to a minimum of 95% SPMDD.

4.7.2 Backfill

Trench backfill materials should be placed and compacted as per OPSS.MUNI 401. Excavated soil is considered geotechnically suitable for use as backfill in trenches provided it is unfrozen, free of organics (topsoil, roots, peat, etc.) and deleterious materials. Backfill must be compacted to the following limits:

- Utility trenches located beneath a roadway: 98% SPMDD
- Utility trenches not located beneath a roadway: 95% SPMDD

All trench backfill must be placed in maximum 200 mm lifts and within 2% of optimum moisture content. The maximum cobble or boulder size should not exceed half of the loose lift thickness (i.e. all particles with a diameter greater than 100 mm should be removed). In confined areas the layer thickness will have to be reduced to utilize smaller compaction equipment efficiently or by using granular material or unshrinkable fill.

Moisture conditioning of the excavated soils may be required to achieve adequate compaction. Moisture can be increased by adding water and mixing the soil prior to re-use, blending the soil with wetter material, or by importing soil to the site that is at optimum and can be readily compacted. Moisture can be reduced by tilling or spreading out the soil to dry or blending it with drier material. In-situ moisture contents can change based on the season and local groundwater levels and can also change for stockpiled material due to precipitation.

Where trenches are located within the traveled portions of a roadway/pavement areas, backfill within the frost penetration depth of 1.2 metres should consist of on-site, non-organic, excavated material consistent with the soils surrounding the trench. If this technique is not undertaken, then frequently problems arise with yearly differential frost heave movements between the trench backfill and the adjacent native soil. This would occur, for example, if imported granular fill was used to backfill the trenches which is less susceptible to frost compared to most of the existing soils at the site with higher silt content.

4.8 Pavement Design

The site plan shows that five roads will be constructed as part of the new development (i.e. Condo Road A to E).



The pavement subgrade must be inspected and approved by the geotechnical engineer at the time of construction. The exposed pavement subgrade should be compacted to a minimum 98% SPMDD. If any soft or weak subgrade areas are identified, or if there are areas containing excessive amounts of moisture or deleterious/organic material, they must be locally sub-excavated and backfilled with approved clean earth fill or imported granular material and compacted to a minimum of 98% SPMDD.

The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures must be maintained to ensure that uniform subgrade moisture and density conditions are achieved as much as possible when fill is placed, and the subgrade is not disturbed or weakened after it is exposed.

Control of surface water is an important factor in achieving a good pavement life. The need for adequate subgrade drainage cannot be over-emphasized. The subgrade must be free of depressions and sloped (at a minimum grade of 2 percent) to provide effective drainage toward subgrade drains. Grading adjacent to pavement areas should be designed to ensure that water is not allowed to pond adjacent to the outside edges of the pavement.

Continuous pavement subdrains should be provided along the edges of the pavement and drained into respective catchbasins to facilitate drainage of the subgrade and the granular materials. The subdrain invert should be maintained at least 0.3 m below subgrade level. To minimize the problems of differential movement between the pavement and catchbasins/manhole due to frost action, the backfill around the structures should consist of free-draining OPSS Granular B. Typical pavement drainage details are provided in Appendix C.

The industry pavement design methods are based on a design life of 15 to 20 years for typical weather conditions depending on actual traffic volumes. For preliminary design the following pavement design is recommended. The pavement design will need to be reviewed once the grading/subgrade has been established.

Pavement Layer	Compaction Requirements	Thickness
<u>Surface Course Asphaltic Concrete:</u> HL3 (OPSS 1150) with PG 58-28 Asphalt Cement (OPSS.MUNI 1101)	OPSS 310	40 mm
<u>Binder Course Asphaltic Concrete:</u> HL8 (OPSS 1150) with PG 58-28 Asphalt Cement (OPSS.MUNI 1101)	OPSS 310	50 mm



Pavement Layer	Compaction Requirements	Thickness
<u>Base Course:</u> Granular A (OPSS.MUNI 1010)	100% SPMDD (ASTM-D698)	150 mm
<u>Subbase Course:</u> Granular B Type I (OPSS.MUNI 1010)	100% SPMDD (ASTM-D698)	300 mm

The granular materials should be placed in lifts 200 mm thick or less and be compacted to a minimum of 100% SPMDD for both granular base and subbase. Asphalt materials should be rolled and compacted as per OPSS 310. The granular and asphalt pavement materials and their placement should conform to OPSS 310, 501, 1010 and 1150.

If the pavement construction occurs in wet, winter or inclement weather, it may be necessary to provide additional subgrade support for heavy construction traffic by increasing the thickness of the granular subbase, base or both. Further, traffic areas for construction equipment may experience unstable subgrade conditions. These areas may be stabilized utilizing additional thickness of granular materials or the use of geogrid.

It should be noted that in addition to adherence to the above pavement design recommendations, a close control on the pavement construction process will also be required in order to obtain the desired pavement life. Therefore, it is recommended that regular inspection and testing should be conducted during the pavement construction to confirm material quality, thickness, and to ensure adequate compaction.

Frost tapers of 10H:1V should be implemented between areas of differing pavement thickness and tie-in areas to existing pavement.

Smooth transitions are required in all areas where the new pavement meets the existing asphalt surface. Asphalt joints shall follow OPSS.MUNI 310. Longitudinal asphalt joints should be milled into the existing asphalt a minimum 0.5 m for each lift. Transverse joints shall be milled into the existing asphalt a minimum 0.5 m for each lift. Successive joints should be staggered.

5. Constructability Considerations

5.1 Excavations

Excavations are generally expected to extend through surficial topsoil and fill/sand layers and into compact to very dense cohesionless glacial till. Where engineered fill has been



constructed, excavation will be carried out through the engineered fill material prior to contacting the native soil. Glacial till inherently contains cobbles and boulders. The Contractor should anticipate slow digging in the glacial till. Provision should be made for handling and removal of cobbles/boulders during excavation.

Excavations must be carried out in accordance with the Occupational Health and Safety Act, Ontario Regulation 213/91 (as amended), Construction Projects, Part III - Excavations, Section 222 through 242. Where workers must enter a trench or excavation, the soil must be suitably sloped and/or braced in accordance with the OHSA. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. If more than one soil type is encountered in an excavation, the most conservative soil type must be followed for sloping the sidewalls of the excavation. Excavations for the site should be completed as follows:

- Type 3 Soil above the water table: 1 Horizontal to 1 Vertical from base of the excavation.
- Type 4 Soil below the water table: 3 Horizontal to 1 Vertical from base of the excavation.

Excavation sidewalls will need to be continuously reviewed for evidence of instability and ground water seepage, particularly following periods of heavy rain or thawing. When required, remedial action must be taken to ensure the continued stability of excavation slopes and the safety of the workers.

Minimum support system requirements for steeper excavations are stipulated in Sections 235 through 238 and 241 of the OHSA and include provisions for timbering, shoring and moveable trench boxes. To reduce the potential for instability of the trench excavations, materials excavated from the service trenches and/or other fill materials or heavy equipment should not be placed near the crest of the trench excavations.

5.2 Temporary Construction Groundwater Control

The water levels measured during the site visits in May 2024 indicated groundwater levels within 0.23 to 4.29 m below ground surface. Consideration should be given to raising the grades at the site to minimize the depth of excavation below the groundwater table and amount of dewatering required.

The exact scenario where certain groundwater control techniques will work are directly correlated to how coarse/fine the native soils are in an excavation, and both the lateral and vertical extent of the wet cohesionless deposits. If the groundwater table is not controlled during construction, the base of the excavations will be unstable, leading to difficulties in excavating and placement of pipes, footings or engineered fill, and providing safety for the workers. Further recommendations regarding dewatering are provided in GEI's Hydrogeological Study Report.



It is recommended to carry out the work during the dry time of the year when the groundwater table is lowest, to mitigate groundwater control measures. Also reducing the size of the excavation that is open at any one time will aid in reducing groundwater control requirements.

5.3 Stormwater Management Pond

A SWM facility is currently proposed near the southwest corner of the site. The SWM facility coincides with a thick fill layer. This fill layer will need to be removed prior to SWM facility construction.

Preliminary recommendations are provided below. These recommendations will need to be revisited once further details are available including grading plans and pond design drawings.

5.3.1 General Construction Considerations

The steepest recommended pond slope inclination is 5H:1V (inside the pond) and should follow the design guidelines of the local municipality. It is recommended that any piping or trenching in the area of the pond should be provided with seepage cut-off collars (clay plugs, concrete plugs, or other barriers) to protect against water seepage through the pipe bedding and backfill. Reference should be made to OPSD 802.095. Pond berms above grade will have to be constructed as engineered fill, constructed as described earlier in the report.

5.3.2 Pond Slope Surface Treatment

The final slope surface and all bare or exposed areas (where applicable) should be provided with suitable vegetation cover or erosion protection. The sloped surface should be provided with a layer of topsoil (minimum 100 mm thick) and should be hydro-seeded with a grass mixture and mulch. If seeded, during the first 2 to 3 years, the surface cover of topsoil and seeding may require periodic maintenance until the vegetation becomes well established. It is recommended that erosion netting/erosion control blankets be staked on the slope surface for erosion protection (including the inside slope above the water level).

5.3.3 Liner Considerations

A liner will be required if a permanent pool is proposed. The liner should be placed along the entire pond bottom and extend a minimum of 1.0 m above the permanent pool elevation. The liner may consist of a natural soil material (such as clay), a synthetic membrane liner (such as a High-Density Polyethylene, Geo-synthetic Clay Liner, or PVC), a concrete liner, or a combination thereof. Details can be provided when the design has progressed.

The liner system must be designed to withstand uplift pressure due to hydrostatic head at the base of the liner for the worse-case condition when the pond is emptied for cleaning and maintenance activities. Uplift pressure can be assessed and reviewed when design details are

established. A gravel/rip rap protection layer should also be considered for the liner when the pond is cleaned out in the future.

5.4 Compaction Specifications

Standard Proctor Maximum Dry Density the specification to indicate the degree to which soil or aggregate is compacted. To achieve the specified SPMDD as indicated in this report, all soils or aggregates must be placed in lift thicknesses no greater than 200 mm. If this is not the case, only the upper portion of the lift will be adequately compacted, and the lower portion of the lift has a high probability of not meeting compaction specifications. In addition, industry standard equipment used to determine the degree of compaction consists of nuclear densometers. These devices have an inherent limitation in that they cannot test beyond 300 mm in depth, and so the degree of compaction beyond this depth cannot be quantitatively determined.

Along with lift thickness, ensuring that the soil or aggregate is within 2% of its OMC ensures that the specified compaction can be reached. If the soil or aggregate is too dry/wet, it is either very difficult or impossible to reach the specified compaction. This is especially true for when higher compaction specifications such as 98% and 100% SPMDD are required.

Moisture can be increased by adding water and mixing the soil prior to re-use, blending the soil with wetter material, or by importing soil to the site that is at optimum and can be readily compacted.

Moisture can be reduced by tilling or spreading out the soil to dry or blending it with drier material. In-situ moisture contents can change based on the season and local groundwater levels and can also change for stockpiled material due to precipitation. Zones of the fine-grained soil with very high moisture contents may find moisture conditioning to be difficult to accomplish.

In addition to the above compaction specifications, in any areas where compacted fill will be placed over the exposed native soil subgrade, any loose, soft, wet, organic or unstable areas should be sub-excavated, and backfilled with clean earth fill or Granular 'B' (OPSS.MUNI 1010) compacted to a minimum of 95% SPMDD. This recommendation applies to site servicing and pavement subgrades. Where structures/buildings require upfilling beneath the structure the fill should be compacted to 100% SPMDD.

5.5 Quality Verification Services

On-site quality verification services are an integral part of the geotechnical design function, and for foundations, engineered fill and retaining walls, are required under the Ontario Building Code. Quality verification services are used to confirm that construction is being conducted in general conformance with the requirements as outlined in the drawings, reports and specifications prepared for the proposed development.



GEI Consultants can provide all the on-site quality verification services outlined below:

- The subgrade for shallow foundations for the proposed buildings will need to be field reviewed by the geotechnical engineer.
- Installation of retaining structures over 1.0 m high and related backfilling operations must be field reviewed on a continuous basis by the geotechnical engineer as required in the OBC.
- Full-time monitoring, testing and inspection of engineered fill placement is required by the geotechnical engineer per the OBC.
- Part-time monitoring of the subgrade support capabilities, material quality, lift thickness, moisture content, degree of compaction, etc. is recommended for the following areas to ensure the recommendations within this report are followed and they perform adequately in the long-term;
 - Slab-on-grades;
 - Pavement structure (granular and asphalt); and
 - Bedding/backfilling of site servicing.
- Testing of the concrete (compressive strength, slump, air content, etc.) and testing of the asphalt (asphalt content and gradation) are recommended to ensure that the quality of the materials being brought to site meet the requirements of the project.



6. Limitations and Conclusions

6.1 Limitations

The recommendations and comments provided are necessarily on-going as new information of underground conditions becomes available. More specific information with respect to the conditions between samples, or the lateral and vertical extent of materials may become apparent during excavation operations. The interpretation of the borehole information must, therefore, be validated during excavation operations. Consequently, conditions not observed during this investigation may become apparent. Should this occur, GEI should be contacted to assess the situation and additional testing and reporting may be required.

GEI should be retained for a general review of the final design drawings and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, GEI will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the guidance of the design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. could be greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

This report was authorized by, and prepared by GEI for Polocorp Inc. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. GEI accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this project.



6.2 Conclusion

It is recognized that municipal/regional governing bodies, in their capacity as the planning and building authority under Provincial statutes, will make use of and rely upon this report, cognizant of the limitations thereof, both as are expressed and implied.

We trust this report is complete within our terms of reference, and the information presented is sufficient for your present purposes. If you have any questions, or when we may be of further assistance, please do not hesitate to contact our office.

Yours Truly,

GEI Consultants

Prepared By:



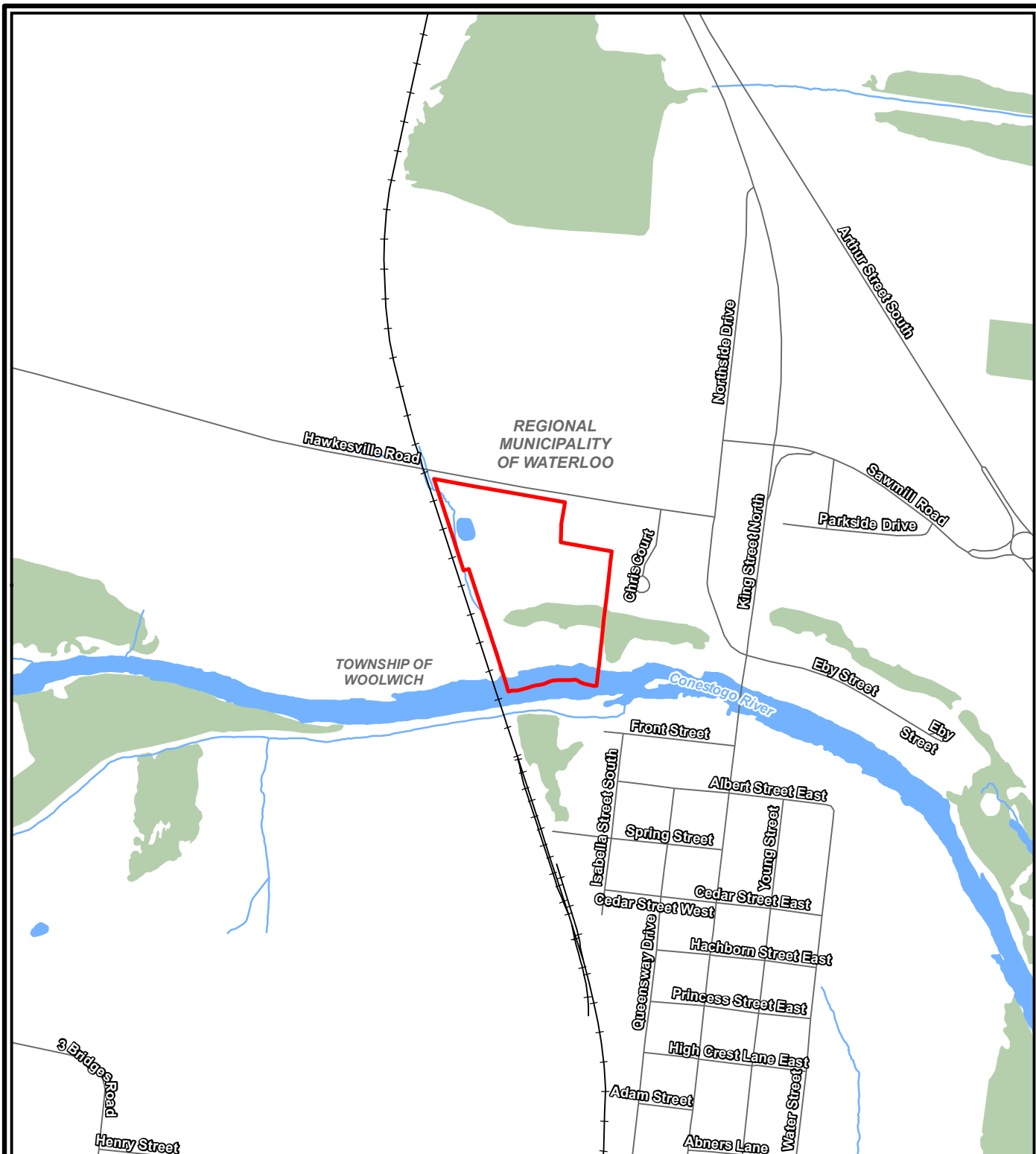
Isaac Asonya, P.Eng.
Senior Geotechnical Engineer

Figures

Site Location Plan

Borehole Location Plans



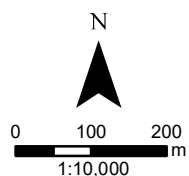


NOTES:

1. Coordinate System: NAD 1983 UTM Zone 17N.
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © King's Printer for Ontario, 2024.

Legend

- Subject Lands
- Waterbody
- Road
- Wooded Area
- Railway
- Watercourse



51-55 Hawksville Road,
St. Jacobs, ON

Polocorp Inc.

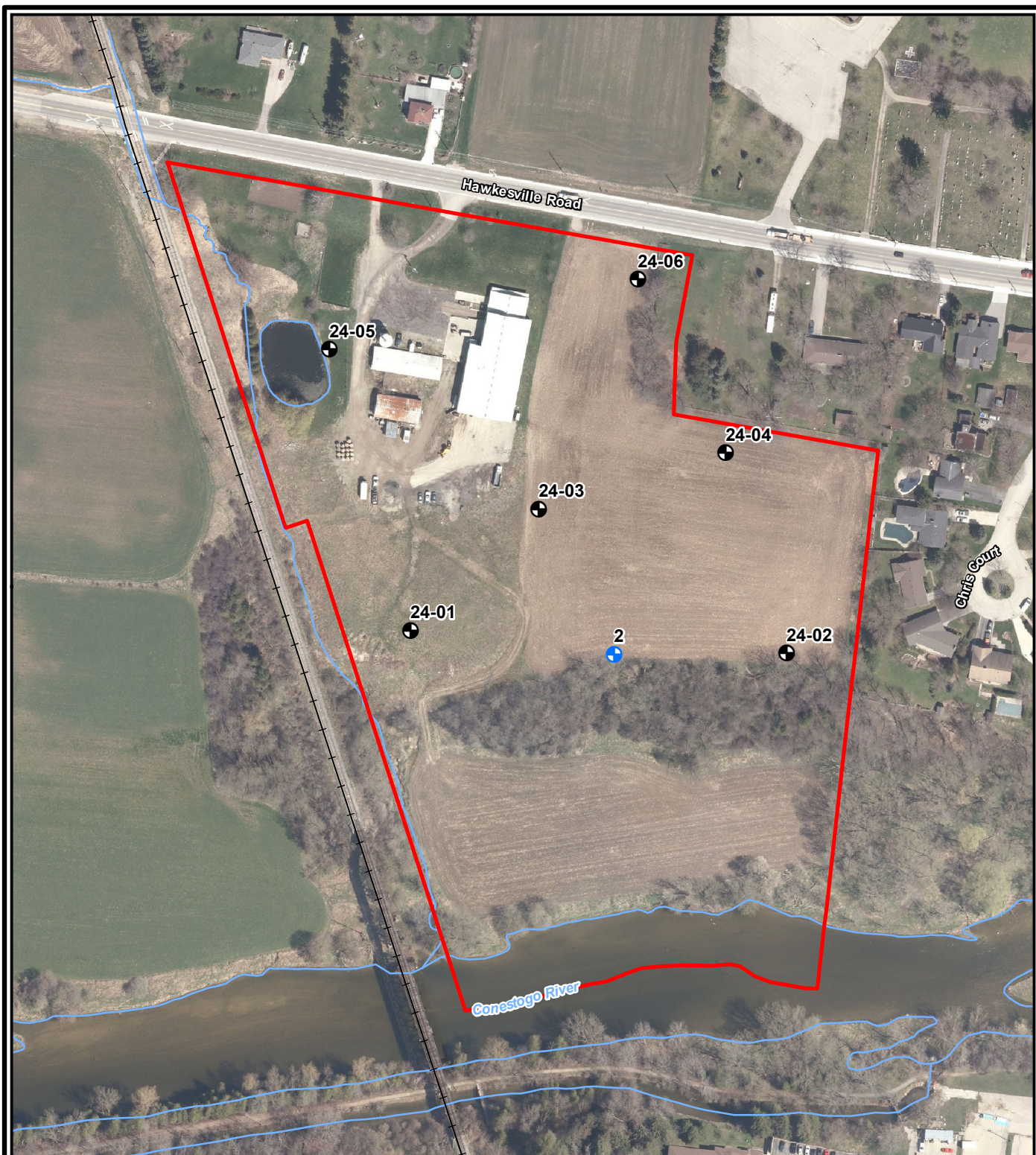


Project 2403070

SITE LOCATION PLAN

June 2024

Fig. 1

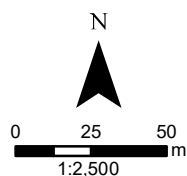


NOTES:

1. Coordinate System: NAD 1983 UTM Zone 17N.
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © King's Printer for Ontario, 2024.
3. Orthoimagery © First Base Solutions, 2024. Imagery taken in 2022.

Legend

- | | |
|--|-------------|
| Subject Lands | Railway |
| GEI Borehole (2024) | Watercourse |
| CVD Borehole (2024) | Waterbody |



51-55 Hawkesville Road,
St. Jacobs, ON

Polocorp Inc.

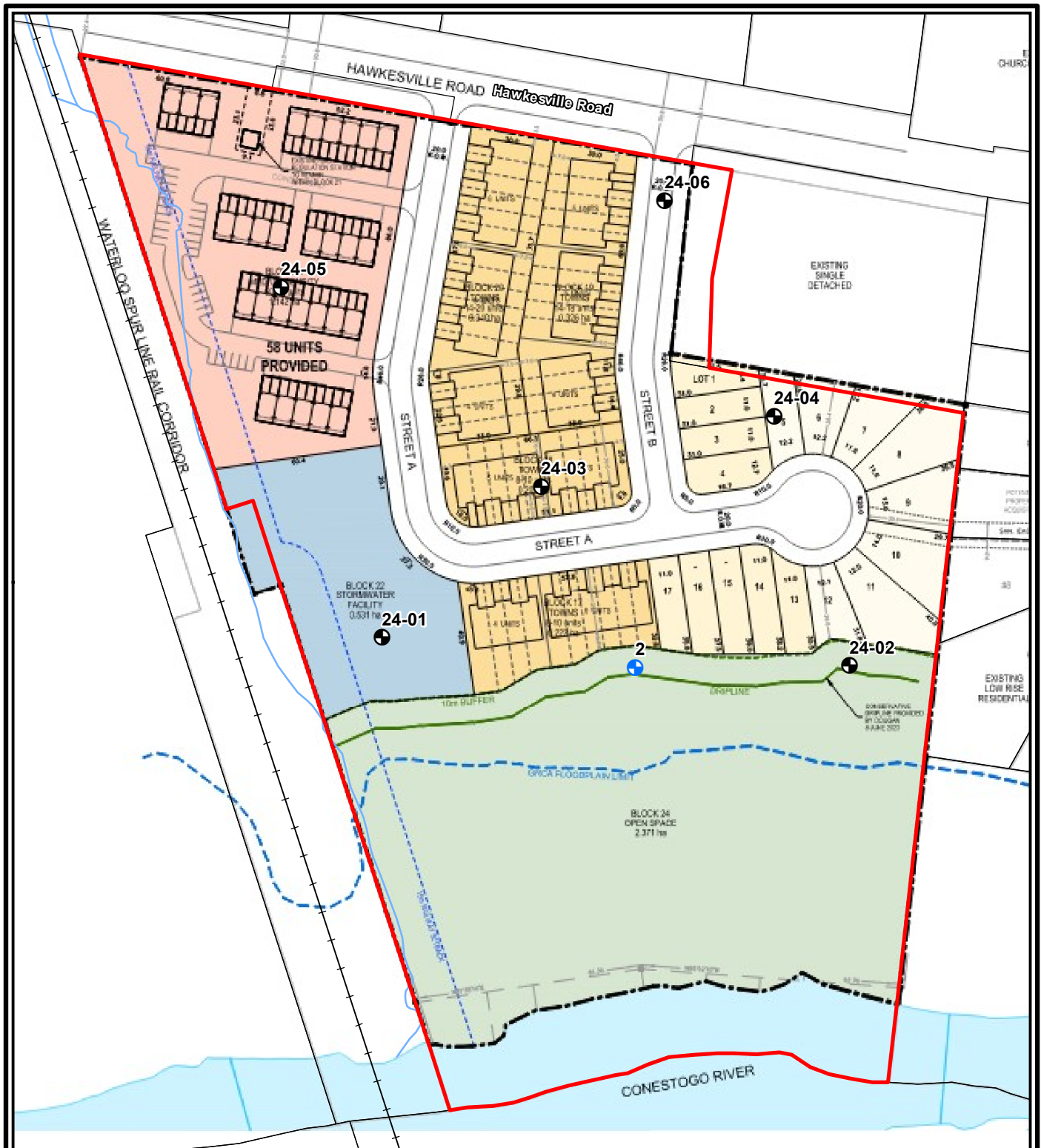


Project 2403070

BOREHOLE LOCATION PLAN
(AERIAL)

June 2024

Fig. 2A

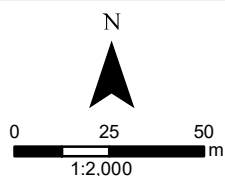


NOTES:

1. Coordinate System: NAD 1983 UTM Zone 17N.
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © King's Printer for Ontario, 2026.

Legend

- Subject Lands
- GEI Borehole (2024)
- CVD Borehole (2024)
- Railway
- Watercourse
- Property Parcel



51-55 Hawkesville Road,
St. Jacobs, ON

Polocorp Inc.



Project 2403070

BOREHOLE LOCATION PLAN
(SITE PLAN)

June 2024

Fig. 2B

Appendix A

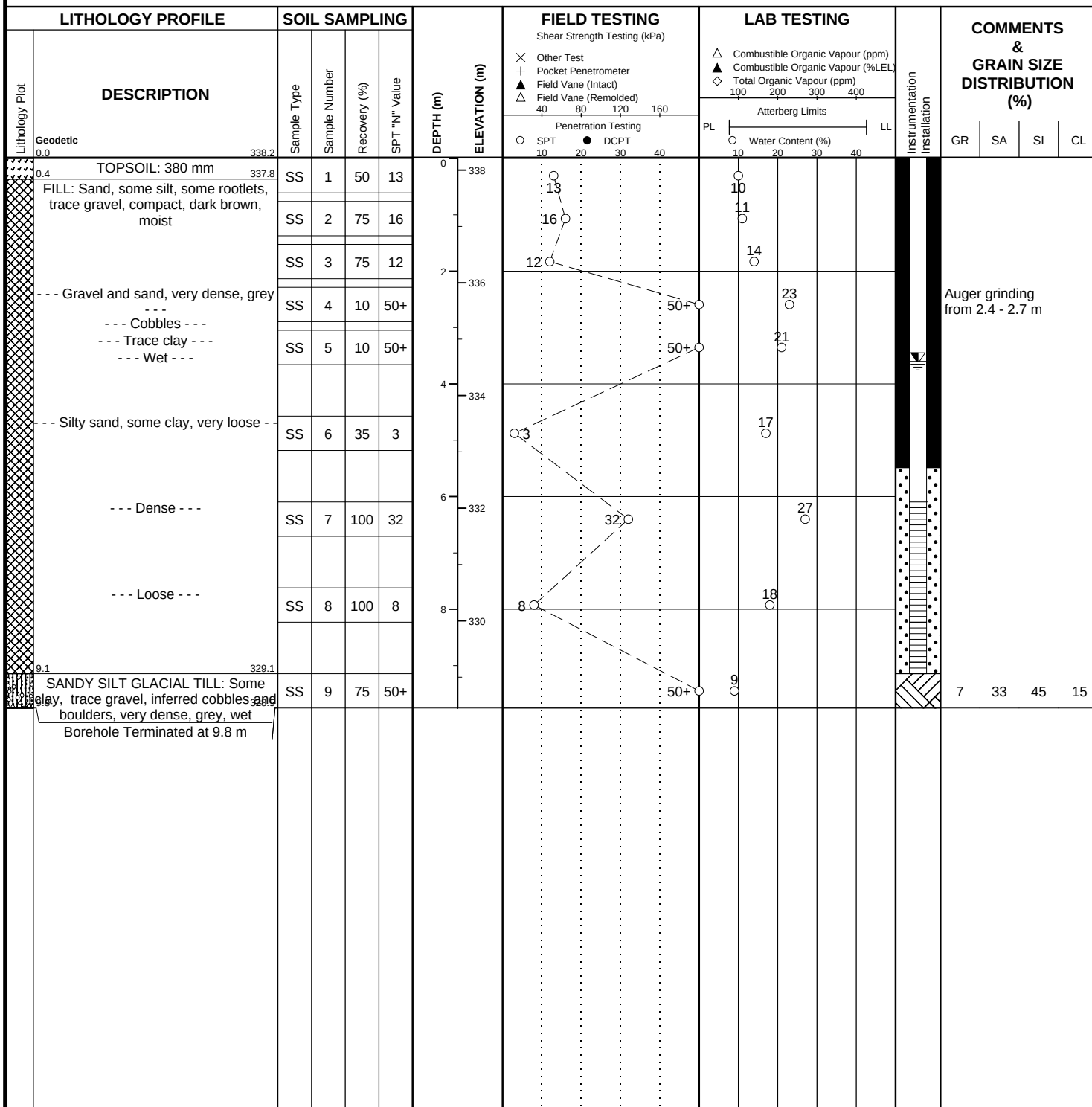
Borehole Logs and Geotechnical Laboratory Data - GEI



RECORD OF BOREHOLE No. 24-01

Project Number: **2403070**
 Project Client: **Polocorp**
 Project Name: _____
 Project Location: **51-55 Hawkesville Road, St. Jacobs, ON**
 Drilling Location: **See Borehole Location Plan**
 Local Benchmark: _____

Drilling Method: **Hollow Stem Augers** Drilling Machine: **Track Mount**
 Logged By: **MM** Northing: **4821191** Date Started: **Apr 23/24**
 Reviewed By: **GL** Easting: **535655** Date Completed: **Apr 23/24**



GEI CONSULTANTS
 Canada Ltd.
 www.geiconsultants.com

Groundwater depth encountered on completion of drilling: Dry Cave depth after auger removal: Open
 Groundwater depth observed on: May 03/24 at depth of: 3.6 m. Groundwater Elevation: 334.6 m

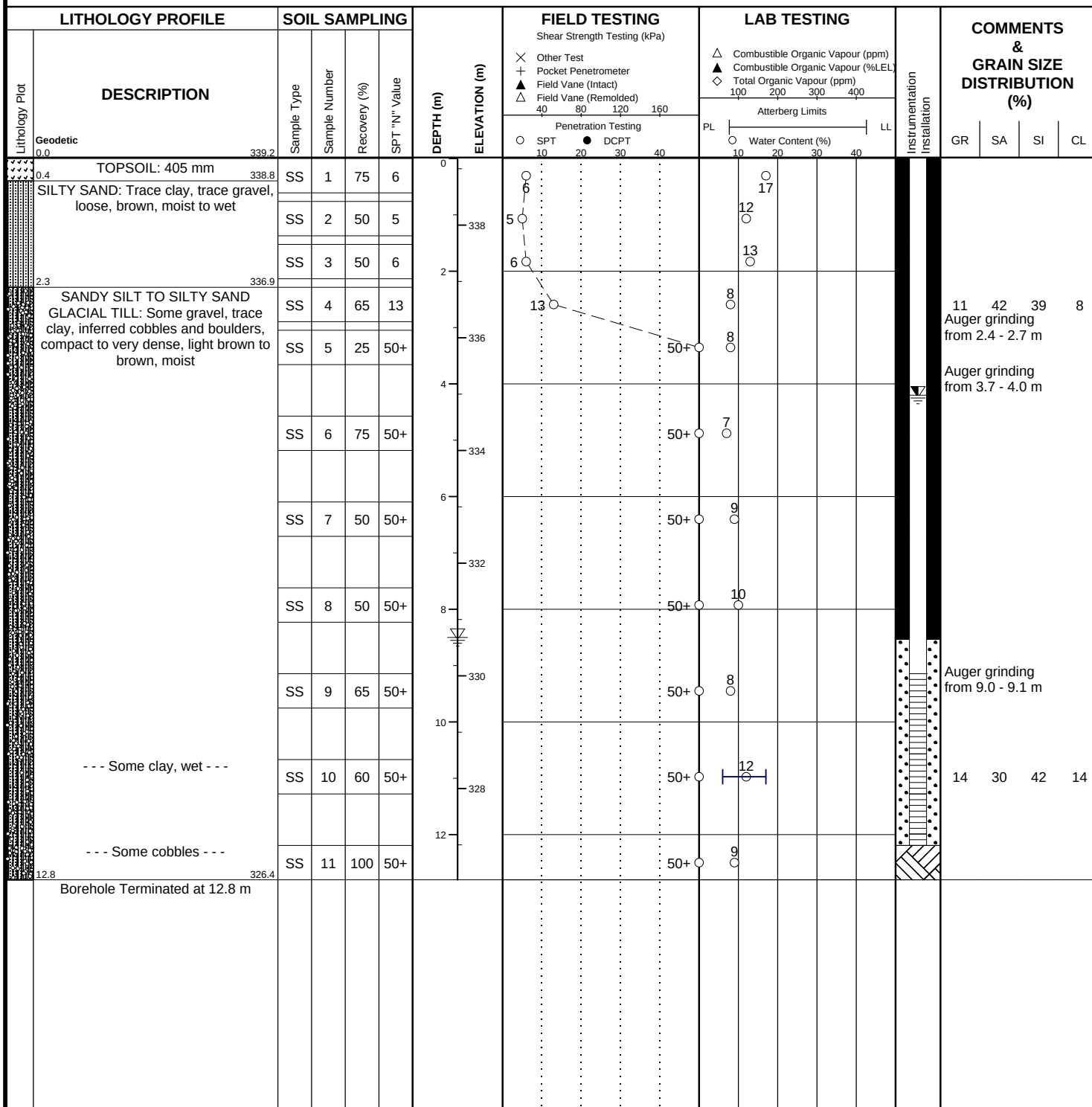
Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: 1:100
 Page: 1 of 6

RECORD OF BOREHOLE No. 24-02

Project Number: **2403070**
 Project Client: **Polocorp**
 Project Name: _____
 Project Location: **51-55 Hawkesville Road, St. Jacobs, ON**
 Drilling Location: **See Borehole Location Plan**
 Local Benchmark: _____

Drilling Method: **Hollow Stem Augers** Drilling Machine: **Track Mount**
 Logged By: **MM** Northing: **4821181** Date Started: **Apr 23/24**
 Reviewed By: **GL** Easting: **535822** Date Completed: **Apr 23/24**



GEI CONSULTANTS
 Canada Ltd.
 www.geiconsultants.com

Groundwater depth encountered on completion of drilling: 8.5 m. Cave depth after auger removal: Open
 Groundwater depth observed on: May 03/24 at depth of: 4.2 m. Groundwater Elevation: 335.0 m

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: 1:100

Page: 2 of 6

RECORD OF BOREHOLE No. 24-03

Project Number: **2403070**

Project Client: **Polocorp**

Project Name: _____

Project Location: **51-55 Hawkesville Road, St. Jacobs, ON**

Drilling Location: **See Borehole Location Plan**

Local Benchmark: _____

Drilling Method: **Hollow Stem Augers** Drilling Machine: **Track Mount**

Logged By: MM Northing: 4821245 Date Started: Apr 23/24

Reviewed By: **GL** Easting: **535712** Date Completed: **Apr 23/24**



Lithology Plot	LITHOLOGY PROFILE		SOIL SAMPLING				DEPTH (m)	ELEVATION (m)	FIELD TESTING		LAB TESTING		Instrumentation Installation	COMMENTS & GRAIN SIZE DISTRIBUTION (%)				
	DESCRIPTION		Sample Type	Sample Number	Recovery (%)	SPT "N" Value			Shear Strength Testing (kPa)		Atterberg Limits							
									Other Test	Penetration Testing	Combustible Organic Vapour (ppm)	Water Content (%)						
	Geodetic								+	+	+	+						
	0.0	340.1							○	○	○	○						
	0.3	339.8	SS	1	75	7			○	○	○	○						
			SS	2	75	7												
			SS	3	75	14												
	2.3	337.8																
			SS	4	50	29												
			SS	5	50	32												
	5.2	334.9	SS	6	50	48												
	Borehole Terminated at 5.2 m																	

GEI CONSULTANTS
Canada Ltd.
www.geiconsultants.com

	Groundwater depth encountered on completion of drilling: Dry		Cave depth after auger removal: Open
	Groundwater depth observed on: May 03/24 at depth of: 0.9 m.		Groundwater Elevation: 339.2 m

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale1 :100

Page: 3 of 6

RECORD OF BOREHOLE No. 24-04

Project Number: **2403070**
 Project Client: **Polocorp**
 Project Name: _____
 Project Location: **51-55 Hawkesville Road, St. Jacobs, ON**
 Drilling Location: **See Borehole Location Plan**
 Local Benchmark: _____

Drilling Method: **Hollow Stem Augers** Drilling Machine: **Track Mount**
 Logged By: **MM** Northing: **4821270** Date Started: **Apr 23/24**
 Reviewed By: **GL** Easting: **535795** Date Completed: **Apr 23/24**

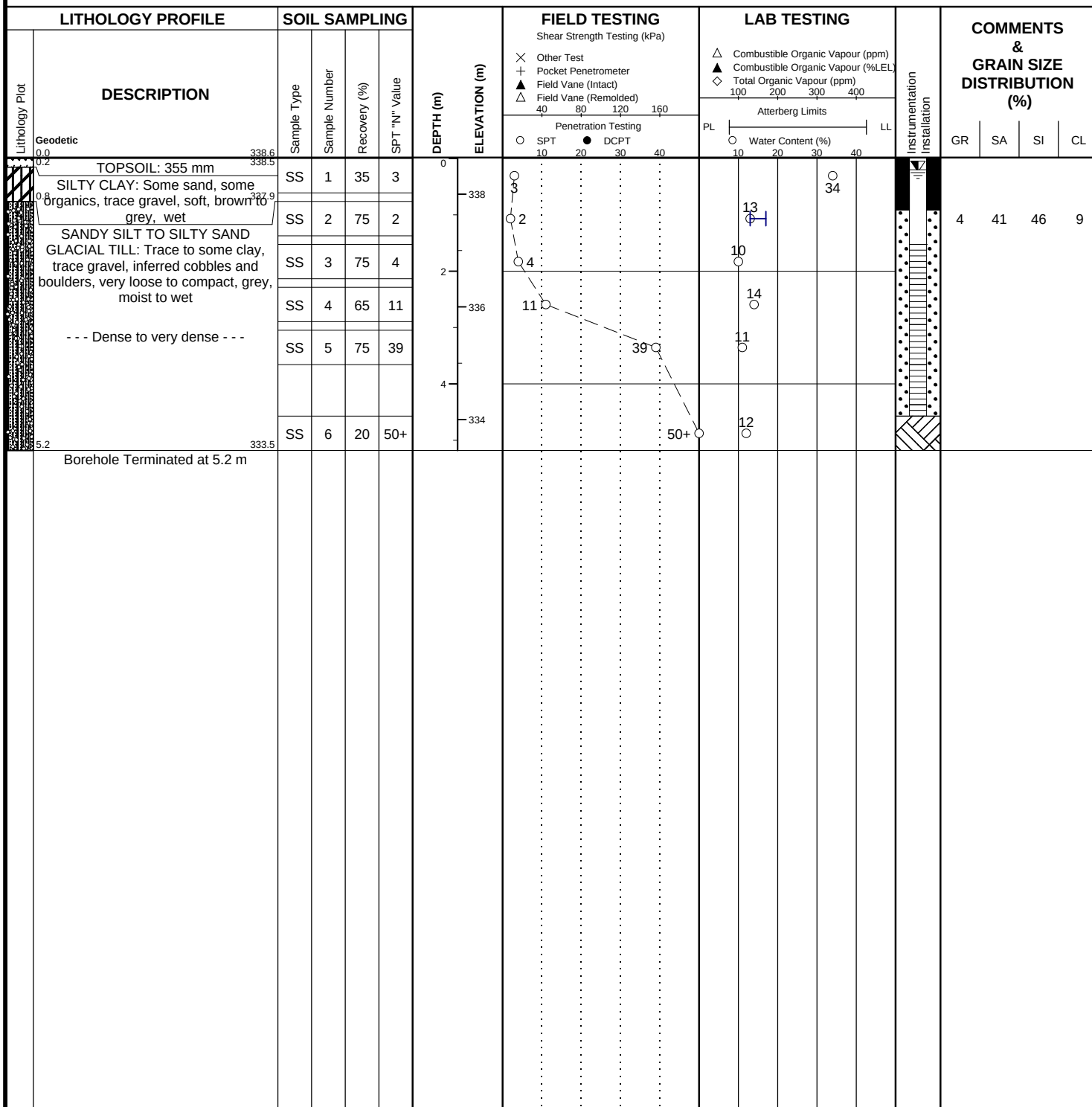


LITHOLOGY PROFILE		SOIL SAMPLING				DEPTH (m)	ELEVATION (m)	FIELD TESTING		LAB TESTING		Instrumentation Installation	COMMENTS & GRAIN SIZE DISTRIBUTION (%)				
Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value			Shear Strength Testing (kPa) × Other Test + Pocket Penetrometer ▲ Field Vane (Intact) △ Field Vane (Remolded) 40 80 120 160	Penetration Testing ○ SPT ● DCPT 10 20 30 40	Atterberg Limits PL Water Content (%) LL 10 20 30 40	GR		SA	SI	CL		
	Geodetic 0.0																
	TOPSOIL: 455 mm 346.4	SS	1	100	4												
	SANDY SILT TO SILTY SAND GLACIAL TILL: Trace clay, trace gravel, inferred cobbles and boulders, compact to dense, brown, moist	SS	2	75	15												
		SS	3	100	21												
		SS	4	100	31												
		SS	5	100	31												
	5.2	SS	6	100	45												
	Borehole Terminated at 5.2 m																

RECORD OF BOREHOLE No. 24-05

Project Number: **2403070**
 Project Client: **Polocorp**
 Project Name: _____
 Project Location: **51-55 Hawkesville Road, St. Jacobs, ON**
 Drilling Location: **See Borehole Location Plan**
 Local Benchmark: _____

Drilling Method: **Hollow Stem Augers** Drilling Machine: **Track Mount**
 Logged By: **MM** Northing: **4821316** Date Started: **Apr 23/24**
 Reviewed By: **GL** Easting: **535619** Date Completed: **Apr 23/24**



RECORD OF BOREHOLE No. 24-06

Project Number: **2403070**
 Project Client: **Polocorp**
 Project Name: _____
 Project Location: **51-55 Hawkesville Road, St. Jacobs, ON**
 Drilling Location: **See Borehole Location Plan**
 Local Benchmark: _____

Drilling Method: **Hollow Stem Augers** Drilling Machine: **Track Mount**
 Logged By: **MM** Northing: **4821347** Date Started: **Apr 23/24**
 Reviewed By: **GL** Easting: **535756** Date Completed: **Apr 23/24**



LITHOLOGY PROFILE		SOIL SAMPLING				DEPTH (m)	ELEVATION (m)	FIELD TESTING		LAB TESTING		Instrumentation Installation	COMMENTS & GRAIN SIZE DISTRIBUTION (%)					
Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value			Shear Strength Testing (kPa)	Penetration Testing	Atterberg Limits	Water Content (%)		GR	SA	SI	CL		
	Geodetic 0.0 345.1																	
	0.3 TOPSOIL: 305 mm 344.7	SS	1	75	4													
	SANDY SILT TO SILTY SAND GLACIAL TILL: Trace clay, trace gravel, inferred cobbles and boulders, loose to compact, brown, moist																	
		SS	2	100	8													
		SS	3	100	19													
		SS	4	100	21													
		SS	5	100	25													
	--- Dense ---																	
	5.2 339.9	SS	6	100	34													
	Borehole Terminated at 5.2 m																	

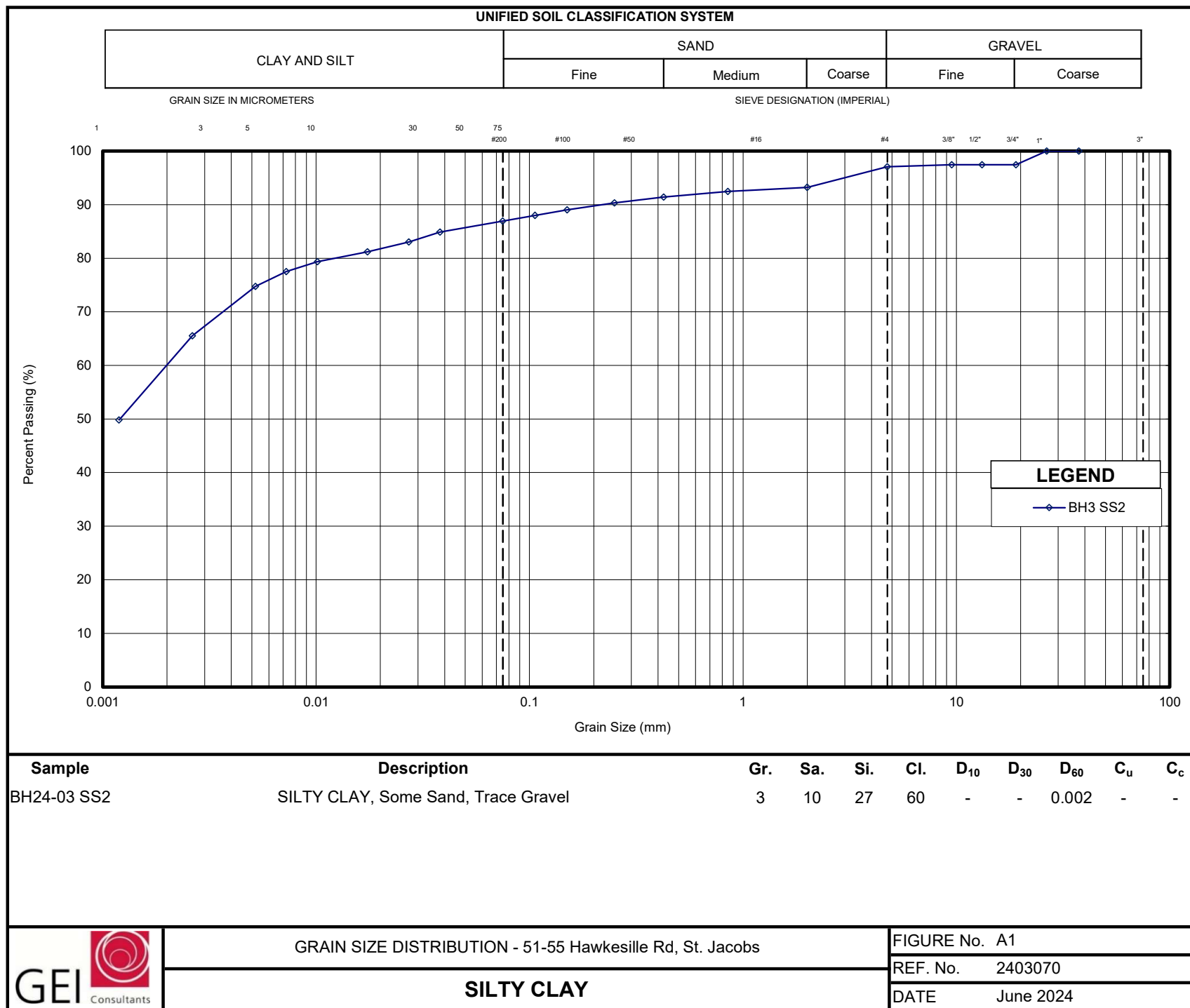
GEI CONSULTANTS
 Canada Ltd.
 www.geiconsultants.com

Groundwater depth encountered on completion of drilling: Dry Cave depth after auger removal: Open
 Groundwater depth observed on: May 03/24 at depth of: 3.2 m. Groundwater Elevation: 341.9 m

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: 1 :100

Page: 6 of 6

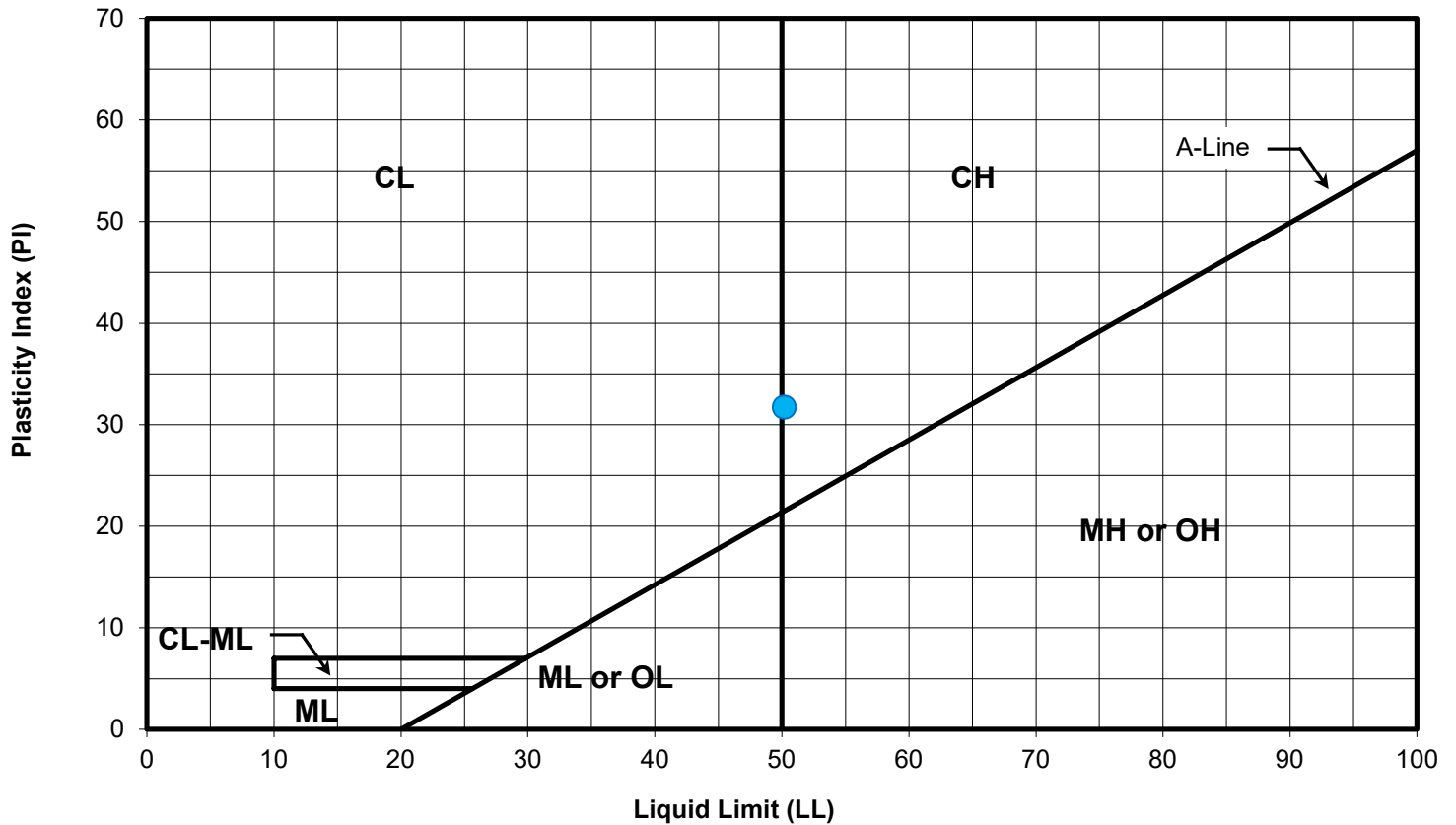


Project Name: 51-55 Hawkesville Rd, St. Jacobs
 Project No.: 2403070
 Client: Polocorp

Figure No.: A3
 Date Tested: June 3, 2024
 Date Sampled: April 23, 2024

SAMPLE INFORMATION								
SAMPLE ID	BH24-03 SS3	LIQUID LIMIT (LL):	50.2	PLASTIC LIMIT (PL)	18.5	PLASTIC INDEX (PI)	31.7	●

Plasticity Chart - Silty Clay



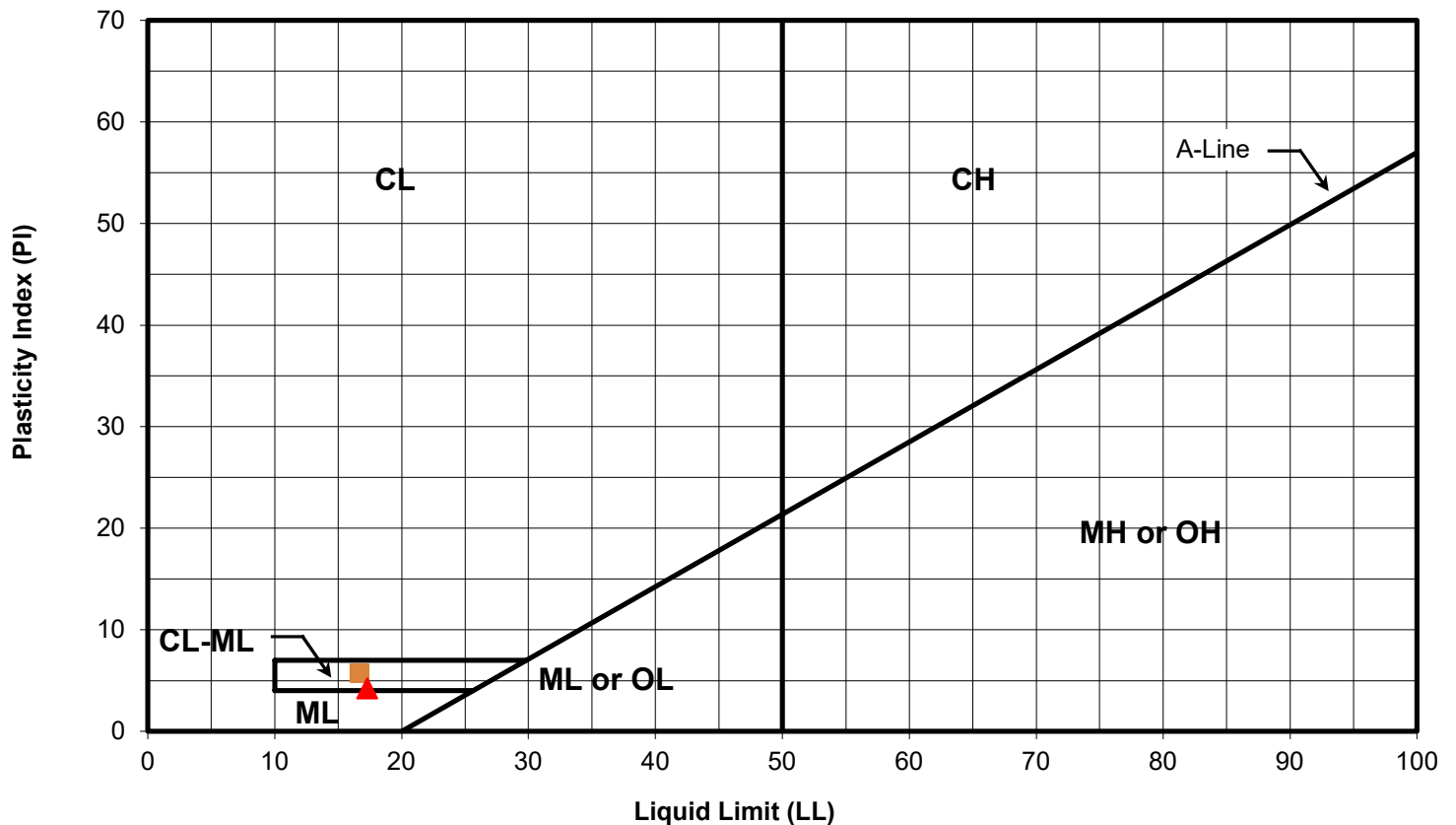
Atterberg Limits Report

Project Name: 51-55 Hawkesville Rd, St. Jacobs
Project No.: 2403070
Client: Polocorp

Figure No.: A4
Date Tested: May 2, 2024
Date Sampled: April 23, 2024

SAMPLE INFORMATION								
SAMPLE ID	BH2 SS10	LIQUID LIMIT (LL):	16.7	PLASTIC LIMIT (PL)	11.0	PLASTIC INDEX (PI)	5.7	■
SAMPLE ID	BH5 SS2	LIQUID LIMIT (LL):	17.3	PLASTIC LIMIT (PL)	13.1	PLASTIC INDEX (PI)	4.2	▲

Plasticity Chart - Glacial Till



Client: GEI Consultants Inc.
647 Welham Rd Unit 14
Barrie, ON
L4N 0B7
Attention: Mr. Geoff Lay
Invoice to: GEI Consultants Inc.
PO#:

Report Number: 3007005
Date Submitted: 2024-04-29
Date Reported: 2024-05-03
Project: 2403070 - 51
Hawkesville Rd, St. Jacobs
COC #: 914399
Temperature (C): 16
Custody Seal:

Page 1 of 6

Dear Geoff Lay:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

Emma-Dawn Ferguson, Chemist

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise stated

Eurofins Environment Testing Canada Inc. is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at <https://directory.cala.ca/>

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline or regulatory limits listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official guideline or regulation as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.



Environment Testing

Client: GEI Consultants Inc.
647 Welham Rd Unit 14
Barrie, ON
L4N 0B7
Attention: Mr. Geoff Lay
PO#:
Invoice to: GEI Consultants Inc.

Report Number: 3007005
Date Submitted: 2024-04-29
Date Reported: 2024-05-03
Project: 2403070 - 51
Hawkesville Rd, St. Jacobs
COC #: 914399

Exceedence Summary

Sample I.D.	Analyte	Result	Units	Criteria

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range



Environment Testing

Client: GEI Consultants Inc.
647 Welham Rd Unit 14
Barrie, ON
L4N 0B7
Attention: Mr. Geoff Lay
PO#:
Invoice to: GEI Consultants Inc.

Report Number: 3007005
Date Submitted: 2024-04-29
Date Reported: 2024-05-03
Project: 2403070 - 51
Hawkesville Rd, St. Jacobs
COC #: 914399

Guideline = O.Reg 153-T1-All Other Soils - Res/Par/Ins/Ind/Com/Prop

Inorganics

					Lab I.D.	1725583	1725584	1725585
					Sample Matrix	Soil	Soil	Soil
					Sample Type			
					Sample Date	2024-04-23	2024-04-23	2024-04-23
					Sampling Time			
					Sample I.D.	BH 4 SS 2	BH 2 SS 2	BH 3 SS 2
Analyte	Batch No	MRL	Units	Guideline				
Chloride	459431	0.002	%		0.003	<0.002	<0.002	
Electrical Conductivity	459372	0.05	mS/cm	STD 0.57	0.12	0.14	0.13	
pH	459372	2.00			7.96	7.93	8.02	
Resistivity	459372	1	ohm-cm		8333	7143	7692	
SO4	459338	0.01	%		<0.01	<0.01	<0.01	

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational
Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim
Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial
Water Quality Guideline, IPWQO = Interim Provincial Water Quality
Objective, TDR = Typical Desired Range



Client: GEI Consultants Inc.
647 Welham Rd Unit 14
Barrie, ON
L4N 0B7
Attention: Mr. Geoff Lay
PO#:
Invoice to: GEI Consultants Inc.

Report Number: 3007005
Date Submitted: 2024-04-29
Date Reported: 2024-05-03
Project: 2403070 - 51
Hawkesville Rd, St. Jacobs
COC #: 914399

Quality Assurance Summary

Batch No	Analyte	Blank	QC % Rec	QC Limits	Spike % Rec	Spike Limits	Dup % RPD	Duplicate Limits
459338	SO4		100	70-130	102		0	-50-50
459372	Electrical Conductivity	<0.05	92	90-110			0	0-10
459372	pH	6.21	99	90-110			0	0-1
459372	Resistivity							
459431	Chloride	<0.002 %	97	90-110			0	

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Client: GEI Consultants Inc.
647 Welham Rd Unit 14
Barrie, ON
L4N 0B7
Attention: Mr. Geoff Lay
PO#:
Invoice to: GEI Consultants Inc.

Report Number: 3007005
Date Submitted: 2024-04-29
Date Reported: 2024-05-03
Project: 2403070 - 51
Hawkesville Rd, St. Jacobs
COC #: 914399

Test Summary

Batch No	Analyte	Instrument	Preparation Date	Analysis Date	Analyst	Method
459338	SO4	Manual	2024-05-01	2024-05-02	M_B	AG SOIL
459372	Electrical Conductivity	Electrical Conductivity Meter	2024-05-02	2024-05-02	IP	Cond-Soil
459372	pH	pH Meter	2024-05-02	2024-05-02	IP	AG Soil
459372	Resistivity	Calculation	2024-05-02	2024-05-02	IP	Resistivity - soil
459431	Chloride	Manual	2024-05-02	2024-05-03	AsA	C CSA A23.2-4B

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Environment Testing

Client: GEI Consultants Inc.
647 Welham Rd Unit 14
Barrie, ON
L4N 0B7
Attention: Mr. Geoff Lay
PO#:
Invoice to: GEI Consultants Inc.

Report Number: 3007005
Date Submitted: 2024-04-29
Date Reported: 2024-05-03
Project: 2403070 - 51
Hawkesville Rd, St. Jacobs
COC #: 914399

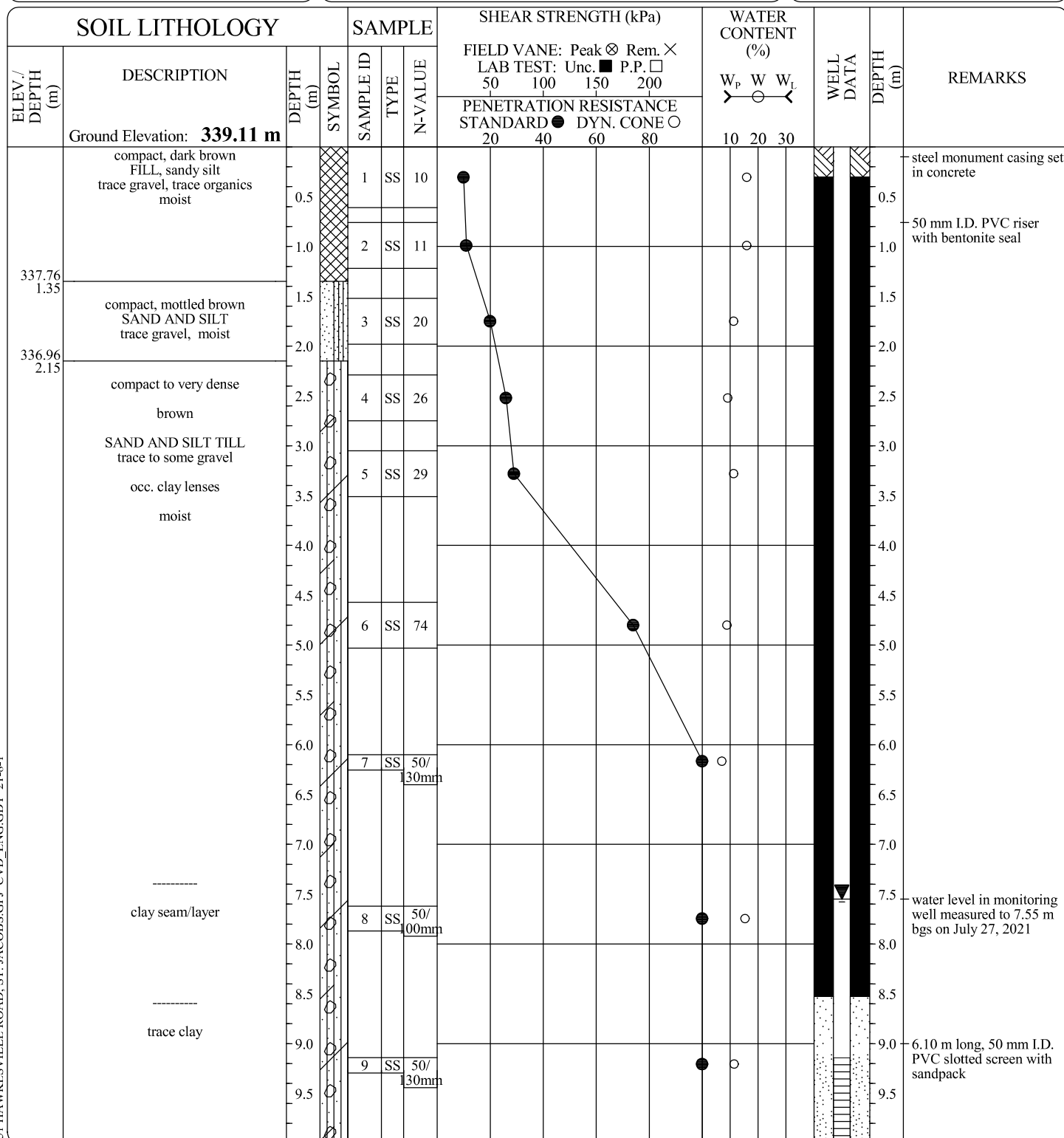
CWS for Petroleum Hydrocarbons in Soil - Tier 1**Notes:**

1. The laboratory method complies with CCME Tier 1 reference method for PHC in soil. It is validated for laboratory use.
2. Where the F1 fraction (C6 to C10) and BTEX are both measured, F1-BTEX is reported.
3. Where the F2 fraction (C10 to C16) and naphthalene are both measured, F2-naphthalene is reported.
4. Where the F3 fraction (C16 to C34) and PAHs* are both measured, F3-PAH is reported.
5. F4G is analyzed if the chromatogram does not descend to baseline before C50. Where F4 (C34 to C50) and F4G are both reported, the higher result is compared to the standard.
6. Unless otherwise stated in the sample comments, the following criteria have been met where applicable:
 - nC6 and nC10 response factors within 30% of response factor for toluene;
 - nC10, nC16, and nC34 response factors within 10% of each other;
 - C50 response factors within 70% of nC10 + nC16 + nC34 average; and,
 - Linearity is within 15%.
7. Unless otherwise stated in the sample comments, sampling requirements and analytical holding times have been met.
8. Gravimetric heavy hydrocarbons (F4G) cannot be added to the C6 and C50 hydrocarbons.
9. *PAHs = phenanthrene, benzo(a)anthracene, benzo(b)fluoranthene, benzo(k)fluoranthene, benzo(a)pyrene, fluoranthene, dibenz(a,h)anthracene, indeno(1,2,3-c,d)pyrene and pyrene.

Appendix B

Borehole Log - CVD

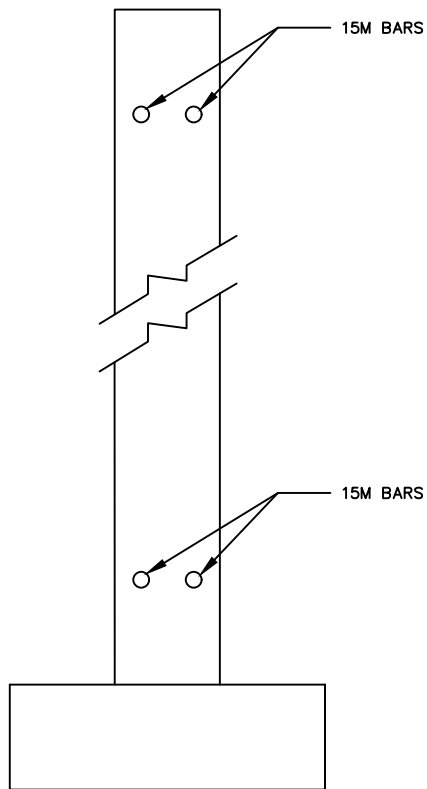


FILE No: G21262**BOREHOLE No. 2**Client: **Polocorp Inc.**Project: **Slope Stability and Fill Investigation**Location: **51 Hawkesville Road, St. Jacobs, Ontario****EQUIPMENT DATA**Machine: **Diedrich D50T**Method: **Hollow Stem Auger**Size: **108 mm I.D.**Date: **Jul 27 - 21 TO Jul 27 - 21**PROJECT MANAGER: **EYC****CHUNG & VANDER DOELEN
ENGINEERING LTD.**311 Victoria Street North
Kitchener, Ontario N2H 5E1
ph. (519) 742-8979, fx. (519) 742-7739

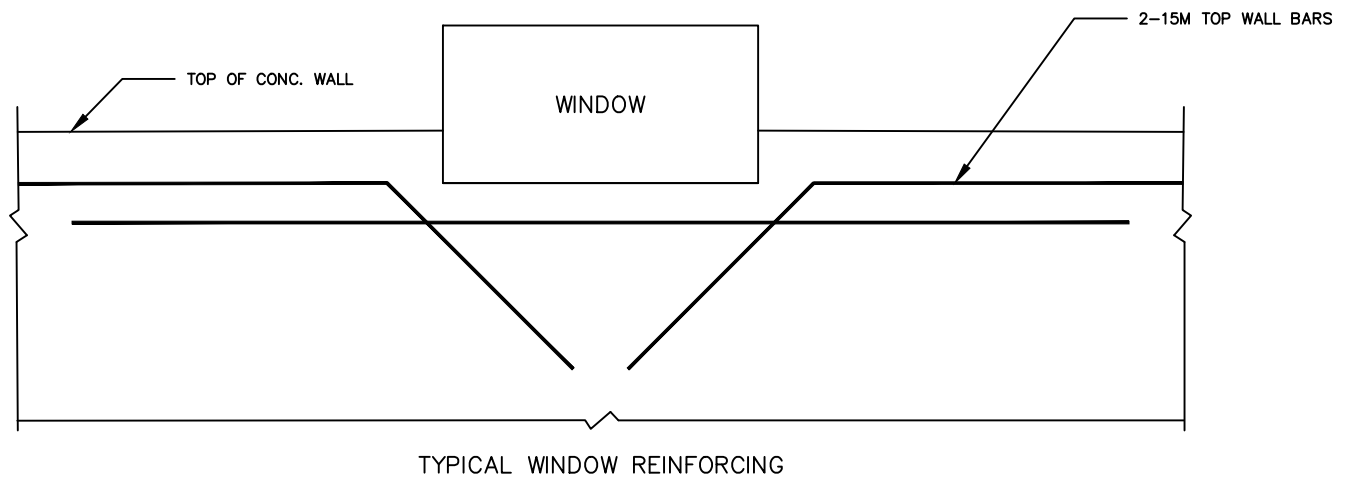
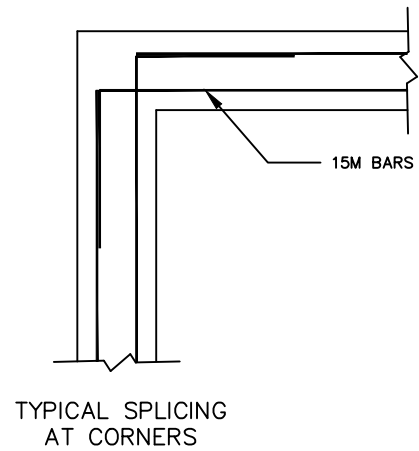
Appendix C

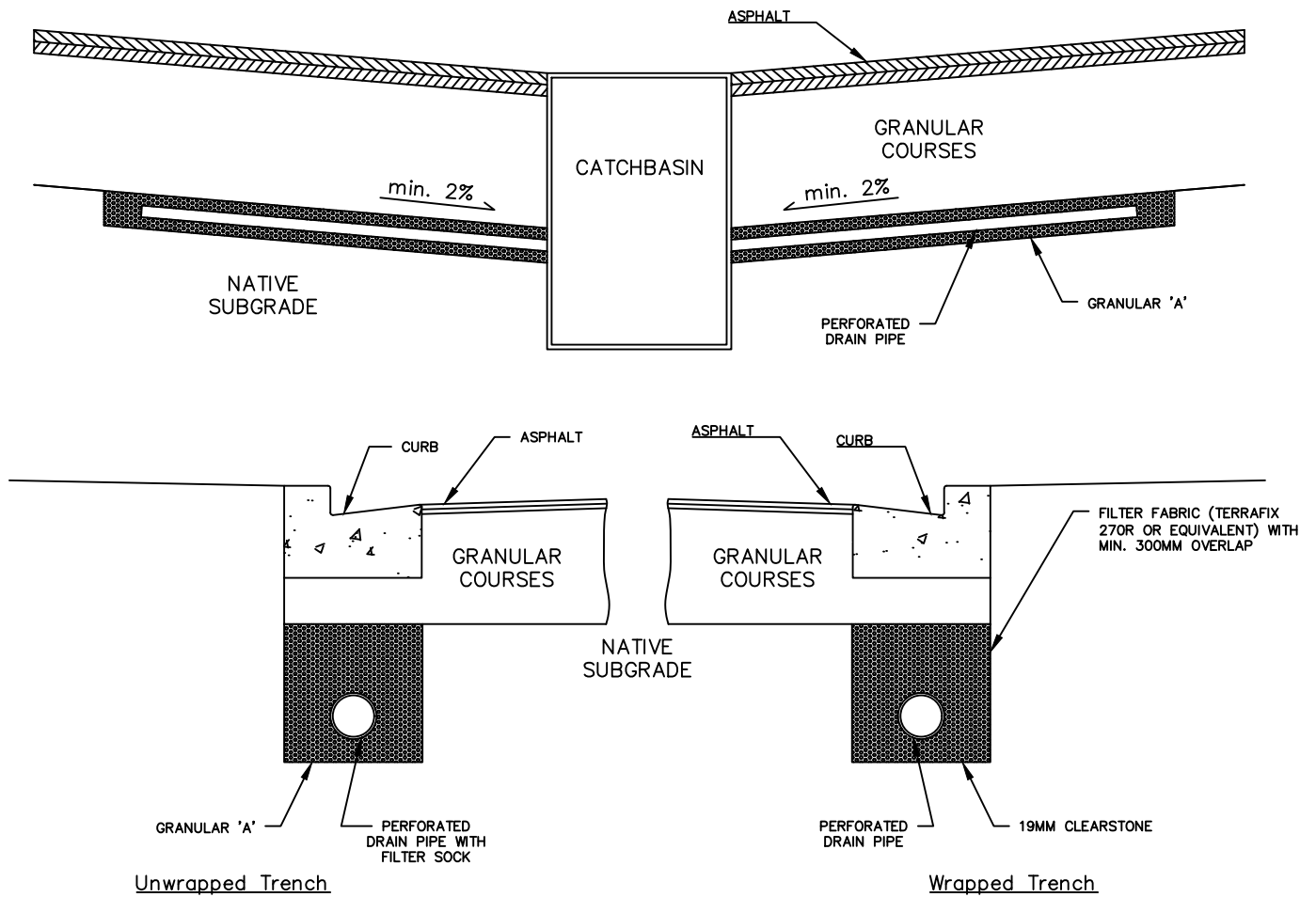
Typical Details



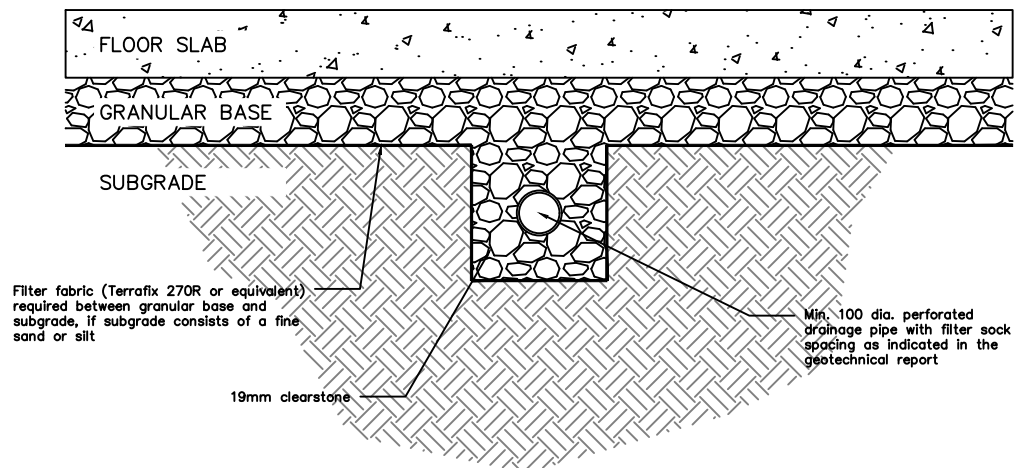
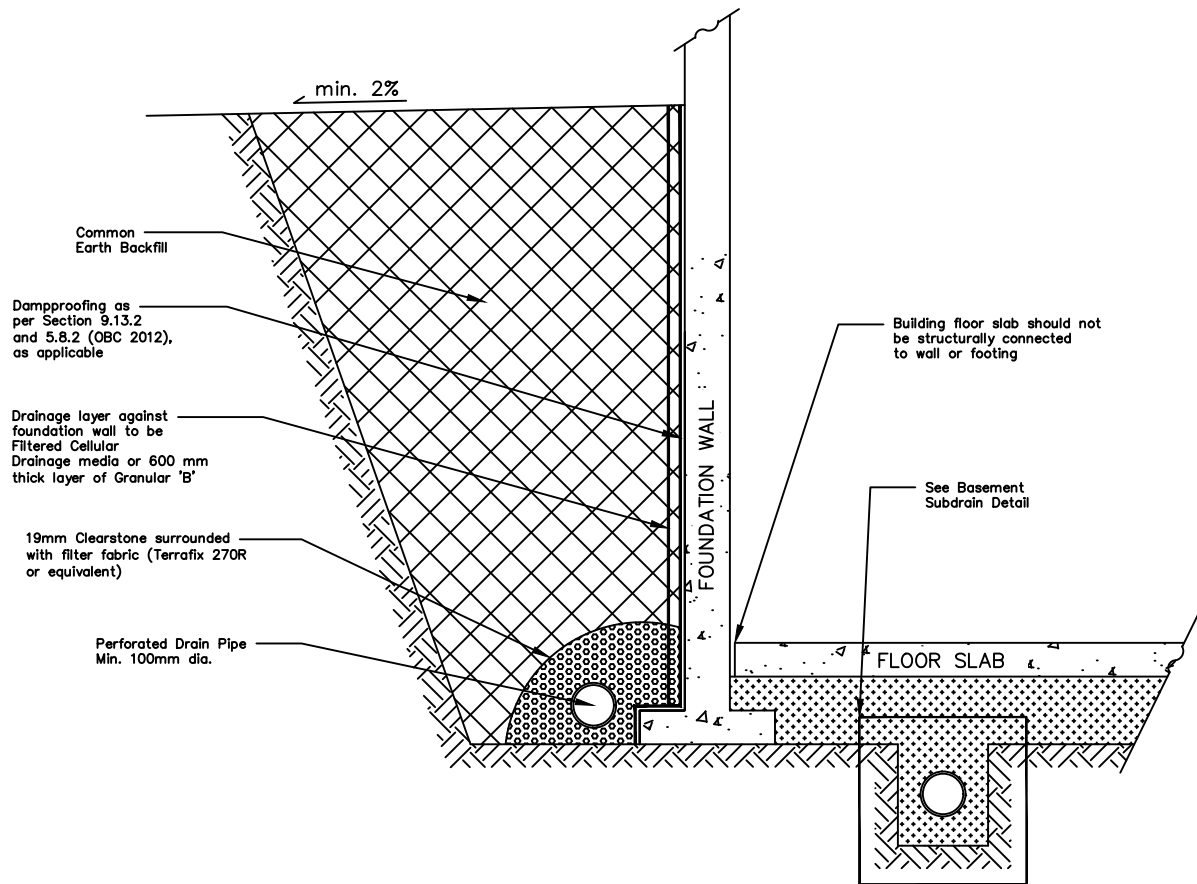


TYPICAL REINFORCED
WALL





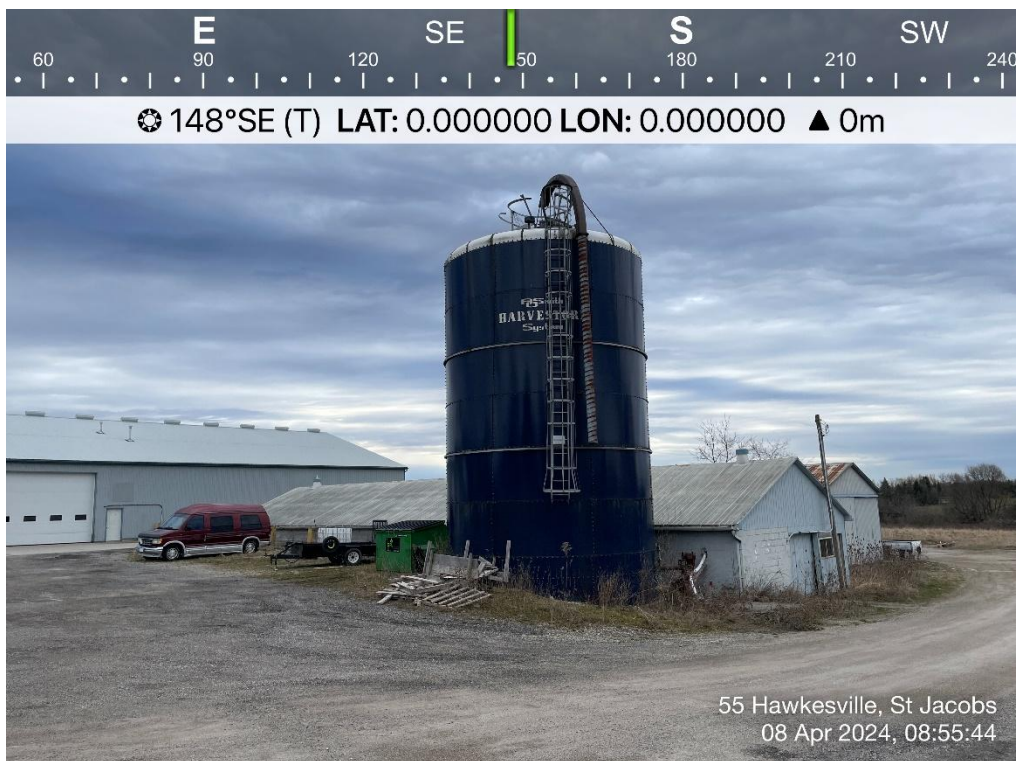
Urban Cross Sections



Appendix D

Site Photographs





PHOTOGRAPH 1

GEI (2024)

Description:
View of buildings
present at the site.



PHOTOGRAPH 2

GEI (2024)

Description:
View of pond near
northwest corner of
the site.





PHOTOGRAPH 3

GEI (2024)

Description:

View of field east of existing buildings. Photograph captured close to Hawkesville Road.



PHOTOGRAPH 4

GEI (2024)

Description:

View of existing buildings.



SE S SW W
20 150 180 210 240 270 300

☼ 215°SW (T) LAT: 43.542857 LON: -80.558677 ±4m ▲ 338m



PHOTOGRAPH 5

GEI (2024)

Description:

View of railway to west of site.

S SW W NW
180 210 240 270 300 330

☼ 264°W (T) LAT: 43.542633 LON: -80.558188 ±4m ▲ 334m



PHOTOGRAPH 6

GEI (2024)

Description:

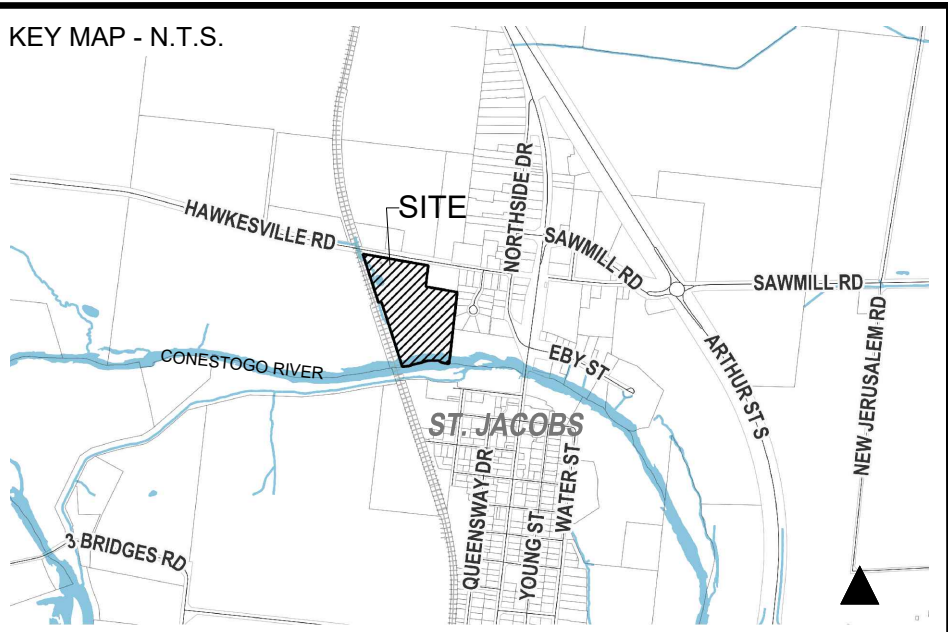
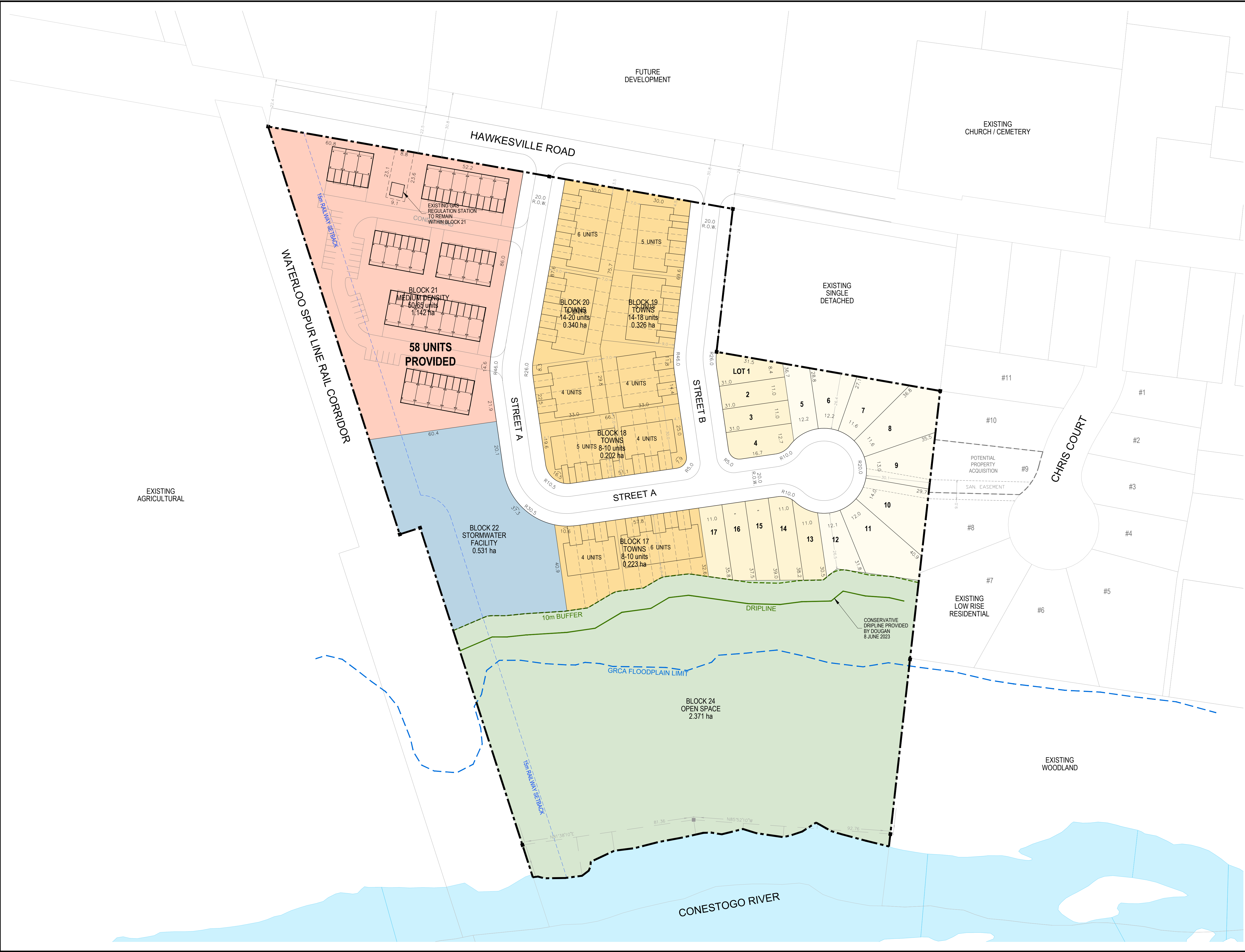
Side view of existing fill located near southwest corner of site.



Appendix E

Site Plan

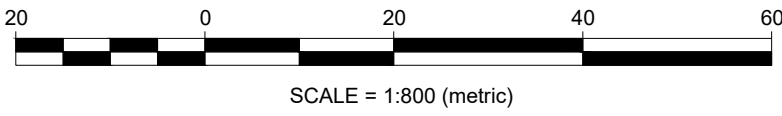




DRAFT PLAN OF SUBDIVISION

PART OF LOT 37, GERMAN COMPANT TRACT,
TOWNSHIP OF WOOLWICH
REGIONAL MUNICIPALITY OF WATERLOO

51 HAWKESVILLE ROAD, ST. JACOBS



LAND USE SCHEDULE			
DESCRIPTION	LOTS/BLKS.	UNITS	AREA (ha.)
Single Detached	1-17	17	0.792
Street Townhouses	17-20	44-58	1.097
Medium Density	21	50-65	1.142
Stormwater Facility	22	-	0.531
Open Space	24	-	2.371
Public Roads	-	-	0.914
TOTAL		111-140	6.847

DENSITY	GROSS	NET
Area (ha)	6.847	4.477
Density (uph)	16.2 - 20.5	24.8 - 31.3
People & Jobs per ha. (P&J/ha)	62.9 - 78.7	41.1 - 51.4

LEGEND	
	Dripline Limit
	Dripline 10m Buffer
	Railway 15m Setback
	GRCA Floodline

INFORMATION REQUIRED
UNDER SECTION 51 (17) OF THE PLANNING ACT, R.S.O. 1990, c.P.13 AS AMENDED
INFORMATION REQUIRED BY CLAUSES a,b,c,d,e,f,i,g,j AND l ARE AS SHOWN ON THE DRAFT PLAN.
(n) Municipal Water Supply
(i) Silty Sands
(k) Municipal Sanitary and Storm Sewers

SURVEYOR'S CERTIFICATE
I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED ON THIS PLAN AND THEIR RELATIONSHIP TO THE ADJACENT LANDS ARE ACCURATELY AND CORRECTLY SHOWN.

JASON ELLIOTT, O.L.S.
J.D. Barnes Limited

OWNER'S CERTIFICATE
I HEREBY CONSENT TO THE FILING OF THIS PLAN BY POLOCORP INC., IN DRAFT FORM

MIKE PUOPOLO
Polocorp Inc.

APPROVALS

DRAFT FOR COORDINATION

REVISIONS		
DATE	BY	DESCRIPTION

POLOCORP 379 Queen Street South
Kitchener, ON N2T 1W6
polocorpinc.com
519-745-3249

DATE: 2026.02.09
PROJECT: 1010 (Hawkesville)
FILE: dps1010-2026.02.09.dwg

SCALE 1:800 (metric)
DRAWN BY: GFE/RT