Preliminary Geotechnical Investigation Report, Rev. 1

Future Commercial Development

2095 Dilworth Road
Kars, ON

Prepared for:

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1 Introduction

Englobe Corp. (Englobe) is pleased to present the findings of our preliminary geotechnical investigation in support of a rezoning application for a future commercial development (Project) at municipal address 2095 Dilworth Road in Kars, Ontario (Site).

Englobe was retained by Dilworth Development Inc. (Client) to carry out a preliminary geotechnical investigation to evaluate the subsurface conditions at the Site and provide typical geotechnical recommendations and comments for foundation design and construction in general. The investigation included the drilling of 10 boreholes within the footprint of the future commercial development site

Written authorization to proceed with this investigation was provided by Mr. Walter Griesseier of Dilworth Development Inc. by means of a signed Project Authorization, dated January 28, 2021.

This report is prepared for the sole use of the Client. The use of the report, or any reliance on it by any third party, is the responsibility of such third party. This report is subject to the limitations shown in Appendix A. It is understood that the Project will be performed in accordance with all applicable codes and standards present within its jurisdiction.

2 Site and Project Description

The Site consists of an approximately 935 m by 375 m undeveloped rectangular lot located at municipal address 2095 Dilworth Road in Kars, ON. The existing Site is bounded by the Veterans Memorial Highway 416 to the west, adjacent residential properties along Third Line Road to the east, Dilworth Road to the south, and an undeveloped property to the north. The existing property is relatively flat and is occupied by a combination of tilled agricultural area and undeveloped treed area. An existing residential structure and garage with a potable well is located southeast of the Site. A watercourse crosses the property from the south to the north on the east side of the Site. The location of the Site is shown in **Appendix B, Figure 1: Site Location Map.**

It is our understanding that the Client is intending to develop the Site with a low-rise commercial development with private well and septic servicing

The balance of the Site is understood to be developed with on grade asphalt parking areas.

Englobe understands that this is a pre-design geotechnical investigation only, as the location and layout of the development has not been finalized. Further targeted investigation will likely be required once plans are finalized. Englobe should be retained to review the proposed development design drawings once they become available to ensure conformance with the general recommendations provided within this report.

The following assumptions about the Project were made by Englobe during the preparation of this preliminary report. Designers should review these assumptions and inform Englobe if there are any discrepancies, as they may affect the recommendations provided herein:

- The buildings may be supported on conventional pad and strip footings on Engineered Fill or alternatively on deep foundations.

- The buildings will be single story slab on grade structures with no sub-surface levels

- No grade raises in excess of 1.0 m is proposed for this Site:

- The structures are governed under Part 4 of the Ontario Building Code (OBC-2012) and therefore will require a site classification for seismic site response; and

- Floor slabs will be lightly loaded with no special point loading from racking, stationary equipment, or process machinery. A typical maximum distributed loading of 24 kPa in compression is assumed.

3 Scope of Work

Englobe's scope of the work was outlined in proposal with Ref No: P2101208, dated January 28, 2021, and was agreed to by the Client on January 28, 2021 by means of a signed Project Authorization. Englobe's mandate generally consisted of the follow activities:

- Retain a utility locating subcontractor to provide underground utility locates;
- Retain a geotechnical drilling subcontractor to drill the following boreholes with a track mounted drill rig:
  - Nine (9) boreholes advanced to a maximum depth of 6.0 m below the existing ground surface (mbgs),
  - One (1) borehole advanced to bedrock, including 1.5 m rock coring, up to a maximum of 30 mbgs, and
  - Two (2) monitoring wells installed and screened within the overburden in two (2) of the above 6.0 m deep boreholes;
- Supervision of the fieldwork and logging of the soil conditions based on the samples that are recovered;
- Submittal of select soil samples to our geotechnical laboratory for the following testing:
  - Three (3) consolidation tests;
  - Six (6) unit weight tests;
  - Ten (10) Atterberg Limit test;
  - Ten (10) grain size analyses,

  - Two (2) standard corrosion packages, and
  - Moisture contents on selected soil samples;
- Preparation of this Preliminary Geotechnical Investigation report.

The thickness and areas of clay soils were inconsistent across the site and relatively thin in some areas, therefore consolidation and unit weight testing were not conducted.

4 Field Investigations and Laboratory Testing

4.1 Geotechnical Drilling Fieldwork

The drilling component of this preliminary geotechnical investigation was performed from February 16 to 19, 2021. The drilling consisted of the advancement of 10 boreholes across the Site. They were labelled as boreholes MW21-01 through BH21-10.
The boreholes were drilled to depths ranging from approximately 4.2 to 7.2 mbgs, with BH21-05 cored into limestone bedrock.

A geotechnical drilling subcontractor, CCC Group, was retained to perform the drilling. All boreholes were drilled using a track mounted drill rig. The boreholes were advanced through the overburden using hollow-stem augers and casings and into the bedrock using wireline diamond coring methods. Monitoring wells were installed with screens sealed into the overburden in MW20-01 and MW20-06. All monitoring well installations and backfilling was completed in accordance with O.Reg. 903 Wells.

Standard Penetration Tests (SPTs) were undertaken in each borehole at 0.76 m intervals with soil samples retrieved using a split spoon sampler. The compaction of cohesionless soils was assessed using recorded SPT N-values. In-situ field vane tests and Pocket Penetrometer (PP) were performed at selected depth intervals to estimate the undrained shear strength of cohesive soils. Rock was cored in BH21-05 with HQ-sized wireline coring equipment to confirm the presence and quality of bedrock.

The subsurface conditions encountered in the boreholes were described by Englobe field staff based on the samples that were recovered. Selected soil samples were sent to Englobe's Ottawa geotechnical laboratory and a third-party geotechnical laboratory for further testing. Two soil samples were sent to a third-party environmental laboratory to undergo limited environmental screening and testing of the standard corrosion package.

The elevation of the ground surface at the borehole locations was interpolated from a survey drawing entitled, “Original Ground Field Topo, 2095 Dilworth Road”, prepared by Tomlinson Limited and should be taken as approximate only. The approximate locations of the boreholes are shown in **Appendix B, Figure 2: Borehole Location Plan**.

4.2 Laboratory Testing

Ten (10) grain size analyses were conducted on 7 samples of the glacial till, 2 samples of the sand/silt and 1 sample of sandy silty clay overburden in MW21-01, BH21-02, BH21-03, BH21-04, BH21-05, BH21-09, and BH21-10. Four (4) Atterberg Limit tests were conducted on four (4) samples of the cohesive soils in BH21-02 and BH21-05. Selected soil samples from all boreholes were submitted for moisture content testing.

Selected soil samples from BH21-05 and MW21-06 were subjected to standard corrosion package tests to evaluate the attack parameters to buried iron and concrete. The analytical test results are provided in **Appendix D.**

5 Description of Subsurface Conditions

The subsoil conditions encountered at the discrete geotechnical borehole locations are briefly discussed in the following subsections with a graphical representation of stratigraphy presented on the detailed Borehole Logs provided in **Appendix C**. A summary of the boreholes drilled at this Site, with soil layers encountered in each borehole is presented in **Table 5-1** below.

Table 5-1: Summary of Borehole Stratigraphy at Discrete Borehole Locations

Borehole ID	Topsoil/ Organic Soil (mm)	Granular FILL (mm)	Sand/Silt (mbgs)	Silty Clay (mbgs)	Glacial Till (mbgs)	Limestone Bedrock (m)
MW21-01	-	0 - 1100	1.1 - 2.3	2.3 - 3.0	3.0 - 6.1*	-
BH21-02	-	-	0 - 1.7	1.7 - 5.1	5.1 - 6.1*	-
BH21-03	-	-	0 - 2.4	-	2.4 - 6.1*	-
BH21-04	-	-	0 - 1.7	-	1.7 - 6.1*	-
BH21-05	0 - 900	-	0.9 - 1.7	1.7-3.2	3.2 - 5.8	5.8 - 7.2*
MW21-06	0 - 1700	-	-	-	1.7 - 6.1*	-
BH21-07	0 - 900	-	0.9 -1.7	1.7 - 4.0	4.0 - 5.9**	

Borehole ID	Topsoil/ Organic Soil (mm)	Granular FILL (mm)	Sand/Silt (mbgs)	Silty Clay (mbgs)	Glacial Till (mbgs)	Limestone Bedrock (m)
BH21-08	0 - 200	-	-	0.2 - 3.5	3.5 - 5.7**	-
BH21-09	0 - 200	-	0.2 - 1.7	-	1.7 - 4.2**	-
BH21-10	0 - 900	-	-	-	0.9 - 5.3**	-

* End of Borehole (EOB)/Termination Depth

** End of Borehole on Auger Refusal

It is important to note that the soil descriptions presented below and in the Borehole Logs represent the soils encountered at the test locations only. They may vary between and beyond borehole locations. This is especially true in previously excavated and/or filled areas such as near existing and former utility trenches and building foundations, such as near and at the existing house and garage identified at the Site.

5.1 Topsoil/Organic Soil

A surficial layer of topsoil was identified at boreholes BH21-08 and BH21-09. The thickness of the topsoil was approximately 200 mm and consisted of silty clay and silt in BH21-08 and BH21-09 respectively.

A layer of organic soil was identified at ground surface at boreholes BH21-05, MW21 06, BH21-07, and BH21-10. The thickness of the organic soil ranged from approximately 900 to 1700 mm and consisted of silty clay to silty sand, dark brown to brown.

It is important to note that the thickness and description of the topsoil noted above are for planning purposes only. They should not be used for quality assessments or quantity take-offs.

5.2 FILL

FILL soils associated with the house construction was identified in borehole MW21-01. The thickness of the FILL layer encountered within the borehole MW21-01 was approximately 1.1 m and consisted of sand and gravel, brownish grey, underlain by silty sand some gravel, grey. Split spoon refusal was encountered within the fill suggesting that it was frozen at the time of investigation. Therefore, the compactness of the fill was not assessed.

5.3 Sand/Silt

Native sand/silt material was identified below the fill in borehole MW21-01, at surface in boreholes BH21-02 to BH21-04, and below the topsoil/organic soil in BH21-05, BH21-07 and BH21 09. The sand/silt material extended to depths ranging from 1.7 to 2.3 mbgs, corresponding to elevations ranging from 84.5 to 85.6 masl. The sand/silt material ranged from silty sand, to silt and sand, to sandy silt, to silt and was mainly brown in color.

SPT ‘N’ values recorded in the sand/silt soils range from 1 to 18 blows per 305 mm penetration, indicating a very loose to compact state.

Two (2) selected soil samples from this deposit were tested for grain size and the summary of the result is presented in Table 5-2 below and attached in **Appendix D**.

Table 5-2 Summary of Grain Size Analysis Results for Sand/Silt

Sample ID	Sample Depth (mbgs)	% Gravel	% Sand	% Silt	% Clay	Description
MW21-01, SS3	1.7-2.3	0	56	44		Silt and Sand
BH21-03, SS3	1.7-2.3	0	10	76	13	Silt some sand some clay
BH21-05, SS3	1.7-2.3	4	29	56	11	Sandy silt some clay trace gravel

5.4 Silty Clay

Native silty clay was identified below the sand/silt material in boreholes MW21¬ 01, BH21 02, BH21-05, BH21-07 and below the topsoil in borehole BH20-08. The native silty clay in boreholes BH21-02, BH21-07 and BH21-08 was first encountered in a weathered/desiccated crustal state at depths ranging from 0.2 to 1.7 mbgs and extending to depths ranging from 1.7 to 2.4 mbgs. The corresponding thickness of the crustal layer ranged from 0.7 to 1.5 m. Below the crust, the silty clay was encountered in an unweathered condition. The weathered silty clay was brown, and the unweathered material was grey in color. The silty clay deposit in MW21¬ 01, BH21 02, BH21-07 extended to depths ranging from 3.5 to 5.1 mbgs, corresponding to elevation ranging from 82.5 to 83.4 masl and a total thickness ranging from 2.3 to 3.3 m. Only a weathered silty clay layer was encountered in boreholes MW21-01 and BH21-05 extending to depths ranging from 3.0 to 3.2 mbgs, corresponding to elevations ranging from 84.2 to 84.5 m.

Pocket penetrometer readings obtained within the crustal layer in boreholes BH21-07 suggest estimated undrained shear strength (Su) values ranging between 49 and 98 kPa, indicating a firm to stiff consistency. Field vane tests conducted in the unweathered sublayer in boreholes BH21-02 and BH21-07, indicated peak undrained shear strength values of 39 to 63 kPa and remoulded undrained shear strength values of 7 to 10 kPa. Based on the peak undrained shear strength values, the unweathered silty clay can be considered to have a firm to stiff consistency. The corresponding sensitivity of the unweathered silty clay ranged from 5 to 9, suggesting the silty clay deposit is sensitive to extra sensitive in accordance with Section 3.1.3.4 of the Canadian Foundation Engineering Manual, 4th Ed. (2006).

One (1) selected soil samples from this deposit was tested for grain size and the summary of the result is presented in Table 5-3 below and attached in **Appendix D.**

Table 5-3 Summary of Grain Size Analysis Results for Sandy Silty Clay

Sample ID	Sample Depth (mbgs)	% Gravel	% Sand	% Silt	% Clay	Description
BH21-05, SS3	1.7-2.3	4	29	56	11	Sandy silty clay trace gravel

Atterberg Limit Test was conducted on three (3) selected soil samples from this deposit and the summary of the result is presented in Table 5-4 below and attached in **Appendix D.**

Table 5-4 Summary of Atterberg Limits Test Results for Selected Silty Clay Samples

Sample ID	Sample Depth (mbgs)	Natural Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity index
BH21-02, SS3	1.7 - 2.3	32	34	17	17
BH21-02, SS5	2.5 - 3.1	84	54	22	32
BH21-05, SS3	1.7 - 2.3	25	27	19	8

The Atterberg Limits testing conducted on representative samples from this deposit indicate the material can be classified as a silty clay of low plasticity (CL) within the crust, transitioning to high plasticity (CH) around 2.5 mbgs.

5.5 Glacial Till

Glacial till was encountered below the topsoil in boreholes MW21-06 and BH21-10, below the sand/silt in boreholes BH21-03, BH21-04, BH21-05, and BH21-09 and below the silty clay in boreholes MW21-01, BH21-02, BH21-07, BH21-08. Boreholes MW21-01, BH21-02, BH21-03, BH21-04 and MW21-06 were terminated in the till at a depth of 6.1 mbgs, corresponding to elevations ranging from 81.1 to 81.8 masl. Boreholes BH21-07, BH21-08, BH21-09, and BH21-10 were also terminated in the till on auger refusal at depths ranging from 4.2 to 5.9 mbgs, corresponding to elevations ranging from 81.0 to 83.1 masl. The glacial till at the Site ranged from clayey sandy silt, clayey silty sand, silty sand, gravelly silty sand, silty sand and gravel to silty sandy gravel. Sporadic blow counts and spoon refusals were encountered in the till material along with labouring of the drill rig; therefore, cobbles and boulders are present throughout the glacial till deposit. A boulder was cored within the glacial till from 3.8 to 4.0 m in borehole MW21-06.

SPT ‘N’ values recorded in the glacial till soils range from 6 to 134 blows per 305 mm penetration, indicating a loose to very dense state, but mainly a compact to very dense state.

Seven (7) selected soil samples from this deposit was tested for grain size and the summary of the result is presented in Table 5-5 below and attached in **Appendix D.**

Table 5-5 Summary of Grain Size Analysis Results for Glacial Till

Sample ID	Sample Depth (mbgs)	% Gravel	% Sand	% Silt	% Clay	Description
MW21-01, SS7	4.6-5.2	28	49	24		Silty Sand some gravel till
MW21-02, SS7	5.5-6.1	32	41	26	1	Silty Sand and Gravel trace clay till
MW21-03, SS5	3.2-3.8	33	40	27		Gravelly Silty Sand till
MW21-04, SS5	3.2-3.8	34	39	26	1	Gravelly Silty Sand trace clay till
MW21-06, SS4	2.4-3.0	18	54	28		Silty Sand some gravel till
MW21-09, SS4	2.4-3.0	36	32	31	1	Silty Sandy Gravel trace clay till
MW21-10, SS3	1.7-2.3	31	36	31	2	Gravelly Silty Sand trace clay till

Atterberg Limit Test was conducted on one (1) selected soil samples from the till deposit and the summary of the result is presented in Table 5-6 below and attached in Appendix D.

Table 5-6 Summary of Atterberg Limits Test Results for Selected Glacial Till Samples

Sample ID	Sample Depth (mbgs)	Natural Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity index
BH21-05, SS8	5.5 - 5.8	11	20	13	7

The Atterberg Limits testing conducted on representative samples from this deposit indicate the matrix of the material can be classified as sandy silt.

5.6 Limestone Bedrock

Bedrock was encountered and cored within borehole BH20-05, underlying the glacial till layer at a depth of 5.8 mbgs corresponding to elevation of 81.9 masl. The bedrock was identified as a grey, limestone.

The core recovery parameters for the completed core run are summarized below in Table 5.7. The measured Rock Quality Designation (RQD) for the completed core run indicates the rock mass is of very poor quality.

Table 5.7 Summary of Core Recovery Parameters

Sample ID	Sample Depth (mbgs)	Total Core Recovery TCR (%)	Solid Core Recovery SCR (%)	Rock Quality Designation RQD (%)	Estimated Rock Mass Quality
BH20-05, RC9	5.8 - 7.2	100	22	22	Very Poor

Auger refusal were encountered in boreholes BH21-07, BH21-08, BH21-09, and BH21-10 at depths ranging from 4.2 to 5.9 mbgs, corresponding to elevations ranging from 81.0 to 83.1 masl. Auger refusal were encountered without confirmatory rock coring. Designers and Contractors are cautioned that these may represent refusal on a cobble and/or boulder as posed to the bedrock surface. Especially given the cobble and boulder content is expected in the native soils.

5.7 Groundwater

Englobe installed a total of two (2) monitoring wells. A summary of the water level observations recorded for each of the installed monitoring wells is provided below in Table 5.8.

Table 5-8 Summary of Monitoring Well Observations

Monitoring Well ID	Approx. Elevation of Screened Interval (masl)	Screened Material	Water Level Observations	
			Date	Approx. Elevation (masl)
MW21-01	84.8 - 81.8 (3.1-6.1 mbgs)	Glacial Till	March 15, 2021	86.4 (1.5 mbgs)
MW21-06	84.5-83.0 (3.1 to 4.6 mbgs)	Glacial Till	March 15, 2021	86.4 (1.2 mbgs)

Monitoring well details and water level measurements are shown on the borehole logs provided in **Appendix C**.

It should be noted that groundwater levels are subject to seasonal fluctuations and response to precipitation, flooding, and snowmelt events. Typically, they are at their highest during the spring thaw. It is recommended that additional water level observations be obtained prior to construction.

6 Discussion and Recommendations

Based on the results of this preliminary geotechnical field investigation and testing performed at the 10 borehole locations on this Site, the following discussion is provided to assist the Client and their Designers with the preliminary foundation design for the Project. The recommendations provided within this report are based on our understanding of the Project, which is summarized above in Section 2 and are general in nature at this preliminary stage. If any of these understandings change, then Englobe should be contacted to assess the implications of those changes on the recommendations provided herein.

Based on the soil conditions encountered in the ten (10) discrete boreholes, and assuming that they are representative of the soil conditions across the Site, the most important geotechnical considerations for the design of the foundations for the Project are expected to be the following:

- **Unsuitable FILL and Low Bearing Capacities on Native Sandy/Silty and Clayey Soils:** Foundation construction on this Site is not generally expected to be straight-forward. The upper native sand/silt and silty clay soils on this Site are not suitable to support the intended buildings with conventional shallow footing construction. The three options available to construct these buildings would be to:
    - Option One: Sub-excavate the FILL and upper native silts, clays and sands down to competent native till and replace with new Engineered Fill
    - Option Two: Sub-excavate down to competent native till and have deeper footings founded on till and longer foundation walls
    - Option Three: Use deep foundations such as driven piles or micropiles.

In the case of Option One or Option Two significant sub-excavation below the water table would be required, along with significant dewatering requirements.

- **Pre-Design Geotechnical Investigation:** It is understood that the Project is currently in the pre-design stage. Therefore, it important to emphasize that this report should be considered as preliminary in nature. Englobe requests to be retained to review the designs once they become available to review for conformance with the recommendations provided within this report.

- **Settlement from Grade Raises:** The native silty clays at the Site have a low shear strength and are subject to consolidation if loaded beyond their pre-consolidation pressure. For grade raises up to 1.0 m, 40 mm of settlement could be expected across the Site from the grade raise alone. Therefore, any future underground utilities and hard surface features, such as retaining walls or hard landscaping, will be subject to settlement and will need to be designed with flexibility and tolerance to increased settlement. For footings supported on undisturbed, glacial till, the settlement of the structure is not a concern. **Please refer to our letter entitled, "Proposed Commercial Development 1.0 m Grade Raise, 2095 Dilworth Road, Kars, ON", dated July 18, 2024 provided in Appendix G.**

- **Dewatering:** Groundwater levels are generally expected to be less than 2.0 mbgs, therefore excavations are likely to extend below the groundwater table. Significant dewatering efforts should be expected, especially if foundation **Option One** and **Option Two** are used which will involve excavation down to the native till.

- **Assumed Slab on Grade Loadings:** A typical floor slab loading for a lightly loaded slab on grade would involve a maximum pressure of 24 kPa. Englobe has not been provided with any specific floor slab requirements such as racking, process equipment or other concentrated loadings. If higher loadings are envisioned, then Englobe should be retained to perform additional consulting in regard to design of the floor slab.

Again, it is important to reiterate that at the time of this geotechnical investigation, Englobe has not been provided with any structural plans of the proposed structures or civil plans for the Project. It is understood that this is a pre-design geotechnical investigation being prepared in support of a rezoning application and should therefore be considered as preliminary in nature. Englobe requests to be retained once designs have progressed to ensure the designs are in conformance with this report.

6.1 Site Preparation

The Site should be graded in the early stages of construction to provide for positive control of surface water and directing it away from any excavations and subgrades. Appropriate provisions should be made for collection and disposal of storm water and runoff including an adequate pumping, or ditching system, if necessary.

6.1.1 Subgrade Preparation Below Footings and Engineered Fill

This Site lays within a possible historic meander of the Rideau River. The floodplain mapping overlay appears to cover a significant portion of the Site. The results of the geotechnical drilling to date indicate that the surficial soils consist of either firm to soft clays, or loose saturated sandy silts. As such the existing sand/silts and clays soils are not suitable to support any new footings. As previously indicated, there are three options for the foundations for this Site. **Option One** and **Option Two** both consist off sub-excavating the silts, clays and sands down to competent native till.

In the case of **Option One** or **Option Two** excavations should extend below all FILL soils, or silty, sandy or clayey soils down to the native undisturbed till. Based on the recent boreholes, the undisturbed native glacial till is expected to be encountered at approximate depths of 1.7 to 5.1 mbgs, corresponding to elevations ranging from approximately 82.5 to 85.7 masl. At the proposed commercial development site , the glacial till was encountered at a depth ranging from 1.7 to 5.1 mbgs, corresponding to elevations of approximately 84.5 to 85.5 masl.

All footing subgrades must be evaluated and approved by a Geotechnical Engineer to ensure that the native subgrade is free of any organics, roots, FILL, loose/disturbed material and can support the suggested design bearing pressure. Any identified local anomalies or soft spots should be subsequently sub-excavated and replaced with Engineered Fill in accordance to the comments in **Section 6.12.2**.

The existing soils on this Site are sensitive to strength loss upon disturbance. If it is disturbed by over-excavation, remoulding, equipment and foot traffic, or subjected to excess water, it will lose its initial strength and will need to be sub-excavated. Contractors should use excavation methods that minimize disturbance to the subgrades. Final excavations should be performed with a smooth-edged ditching bucket. It is recommended that designs consider the use of a lean mix concrete mud mat on the approved subgrade surfaces to protect the subgrades and to provide for a clean dry working surface.

6.1.2 Subgrade Preparation Below Floor Slabs

Any existing topsoil, organic soils, or FILL are not suitable to remain under future floor slabs. Therefore, excavations should extend below all existing topsoil, organic, FILL, expose native undisturbed subgrade. The resultant subgrade must be reviewed and approved by the Geotechnical Engineer.

Subgrades must be free of fills, disturbed or loosened soils, roots, and any organics. The exposed subgrades should be proof rolled using a loaded dump truck or large vibratory roller to identify soft spots, deflection, rutting, or local anomalies. Any identified local anomalies or soft spots should be subsequently sub-excavated, replaced with suitable imported fill, and compacted. Any imported fill underlying floor slabs should be considered as Engineered Fill and treated in accordance to the comments in **Section 6.12.2.**

6.2 Excavations

As previously indicated, there are three options for the foundations for this Site. **Option One** and **Option Two** both consist off sub-excavating the silts, clays and sands down to competent native till ranging in depth from approximately 2.4 mbgs to approximately 5.1 mbgs. These excavations will prove to be quite deep and will be below the water table. **Option Three** consisted of piled foundations and therefore will limit excavation depths to approximately 1.5 to 2.0 mbgs. In assessing the foundations for this Site designers should consider that dewatering efforts will be significant.

Based on foundation excavations down to undisturbed native glacial till subgrade, we anticipate that the deepest excavations will be a maximum of approximately 5.1 m deep at the site. As space permits, it is expected that excavations will be performed using sloped open excavations.

All excavations must be undertaken in accordance with the requirements of the Occupational Health and Safety Act of Ontario (OHSA), Regulations for Construction O.Reg. 213/91, as amended. The comments within this subsection are intended to be in addition to, and not a replacement of the OHSA requirements.

- The existing FILL soils would be considered as a “Type 3 Soil” according to the OHSA regulations. However, if they become wet or muddy, are below the water table, or shows signs of seepage, they would be considered as a “Type 4 Soil”.

- The existing native silt and sand to silty sand to sandy silt to silt soils would also be considered as a “Type 4 Soil” according to the regulations.

- The existing native silty clay and glacial till soils would be considered as a “Type 3 Soil” according to the OHSA regulations. However, if they become wet or muddy, are below the water table, or shows signs of seepage, they would be considered as a “Type 4 Soil”.

- According to the OHSA regulations, excavations which penetrate through multiple soil types should be considered as having the highest soil type.

The stability of the excavation side slopes will be highly dependent on the Contractor’s methodology. No surface surcharges should be placed closer to the edge of the excavation than a distance equal to twice the depth of the excavation, unless an excavation support system has been designed to accommodate such a surcharge.

No excavations should penetrate below an imaginary line drawn downwards and outwards at 7V:10H slope from the toe of any existing footing or load bearing elements, in order to avoid undermining them. Designers and Contractors should plan out the approximate excavation area and compare them to the location of any existing footings to ensure they are not undermined. If the limit of not undermining adjacent structures, or other space or property line restrictions are encountered then Engineered Shoring may be necessary.

It is important to emphasize that cobbles and boulders were visible on the surface in the wooded portion of this Site. Furthermore, frequent split spoon refusals were encountered during drilling along with sporadic blow counts in the till. Contractors should expect cobbles and boulders within the till soils. These may be difficult to excavate, and if removed will leave an uneven subgrade. Contract documents should include an item for removal of boulders from subgrades and replacement with Engineered Fill to compensate for this issue.

6.3 Temporary Construction Dewatering

As discussed in Section 5.7.7, a total of two (2) monitoring wells were installed in MW21-01 and MW21-06. The water levels recorded on March 15, 2021 were found to range in depth from approximately 1.2 to 1.5 mbgs.

The foundation excavations are likely to extend below the groundwater table, therefore significant groundwater seepage may be expected. Furthermore, the Designer and Contractor must consider the possibility of fluctuations depending on the time of year and climate or weather events before or during construction.

Surface water infiltration may also be expected where relatively permeable soils or soil layers exist, such as the relatively permeable sand/silt soils and within the glacial till. Perched water may also be encountered above the relatively less permeable native silty clay deposit encountered at the Site.

Any surface water infiltration and groundwater seepage into excavations will need to be adequately controlled. Water quantities will depend on seasonal conditions, depths of excavations, and the duration that excavations are left open. Groundwater will travel easily through the observed sand/silt soils and within the glacial till.

Effective groundwater control prior to and during construction will be required. Designer and Contractors are to refer to the hydrogeological study and terrain analysis provided under a separate cover. Recommendations for appropriate dewatering measures beyond conventional sump pump techniques, such as a positive dewatering system (e.g., well points or other specialized methods) to effectively lower the static groundwater level shall be provided by a specialized dewatering contractor based on the findings and recommendations of the hydrogeological investigation, if required.

Option One and **Option Two** both consist off sub-excavating the sands, silts, and clays down to competent native till ranging in depth from approximately 2.4 mbgs to approximately 5.1 mbgs. These excavations will prove to be quite deep and will be below the water table. **Option Three** consists of piled foundations and therefore will limit excavation depths to approximately 1.5 to 2.0 mbgs. This option would reduce dewatering volumes.

6.4 Foundations

It is important to note that at the time of this geotechnical investigation, the Project is still considered to be in the pre-feasibility stage and Englobe has not been provided with the complete proposed development plan, nor foundation details.

As mentioned above, this Site lays within a possible historic meander of the Rideau River. The floodplain mapping overlay appears to cover a significant portion of the Site. The results of the geotechnical drilling to date indicate that the surficial soils consist of either firm to soft clays, or loose saturated sandy silts. These loose soils, the high-water table, and the shells noted while drilling suggest that these soils are of an alluvial deposition. As such the existing sand/silts and clays soils are not suitable to support any new footings. There are three options for the foundations for this Site:

- **Option One:** Sub-excavate the FILL and upper native silts, clays and sands down to competent native till and replace with new Engineered Fill,

- **Option Two**: Sub-excavate down to competent native till and have deeper footings founded on till and longer foundation walls, or

- **Option Three:** Use deep foundations such as driven piles or micropiles.

In the case of **Option One** or **Option Two** significant deep excavation below the water table would be required would result in significant dewatering requirements.

Table 6-1 outlines the expected depth/elevation to glacial till and expected founding condition at the proposed commercial development site. asl.

Table 6-1 Expected Founding Conditions at Proposed Site

Borehole No.	Glacial Till		Expected Founding Conditions
	Depth (mbgs)	Elevation (masl)	
BH21-02	5.1*	82.5*	Footings founded on new **Engineered Fill** directly overlying native undisturbed till. Piled foundations may also be considered to limit excavation depth and dewatering.
BH21-03	2.4*	84.8*	Footings founded on **new Engineered Fill** directly overlying native undisturbed till. Piled foundations may also be considered to limit excavation depth and dewatering
BH21-04	1.7*	85.5*	Footings founded **directly on native undisturbed till**
BH21-05	3.2*	84.5*	Footings founded on **new Engineered Fill** directly overlying native undisturbed till. Piled foundations may also be considered to limit excavation depth and dewatering
MW21-06	1.7*	85.9*	Footings founded **directly on native undisturbed till**

* Suggested estimated depths/elevations are subject to settlement calculations during the detailed design to ensure settlement remain within tolerable limits.

Conventional shallow spread or strip footings, if used, must extend below all FILL, organics, sand/silt, and silty clay soils and bear on the native undisturbed glacial till, or on new Engineered Fill directly overlying native undisturbed glacial till. All foundation subgrades must be approved by the Geotechnical Engineer.

Footings at varying levels and/or constructed adjacent to utility trenches, sump pits or similar should be constructed, such that the higher footings be set at a level below an imaginary line constructed 7V:10H from the base of the lower excavation. Step footings, if required should be designed with benching no steeper than 2H:1V along their length, and steps no higher than 0.3 m.

6.4.1 Recommended Bearing Pressures for Footings on Glacial Till Subgrade (Option Two)

For conventional pad footings 2.0 m by 2.0 m, or greater, and extended strip footings 1.0 m wide, or greater, founded on the native undisturbed glacial till below all FILL and weak native soils at or below about 3 m depth, a factored bearing capacity of 150 kPa under Ultimate Limit States (ULS) conditions is recommended. This includes a geotechnical resistance factor of Φ = 0.5. A Serviceability Limit States (SLS) design bearing pressure of 100 kPa is recommended. This assumes a maximum tolerable differential settlement in the order of 19 mm and a maximum tolerable total settlement in the order of 25 mm.

Based on our conversation with the Client, up to 1.0 m grade raise is anticipated to be at the Site, therefore 40 mm of settlement could be expected across the Site from the grade raise alone. However, considering the footing will be supported on undisturbed glacial till, the settlement of the structure is not a concern. **Please refer to our letter entitled, "Proposed Commercial Development 1.0 m Grade Raise, 2095 Dilworth Road, Kars, ON", dated July 18, 2024 provided in Appendix G.** If grade raises, larger dimensioned footings, or higher bearing pressures are used, then Englobe should be retained to perform additional fieldwork, testing, and Engineering analysis to support a specific settlement estimate on a case-by-case basis.

Subgrade preparation for glacial till subgrades will involve the removal of all fills, organics, disturbed or previously excavated soil to expose a native, undisturbed glacial till. The exposed surface should be examined by the Geotechnical Engineer to assess the competency. Any identified local anomalies or soft spots should be subsequently excavated, replaced with new Engineered Fill.

6.4.2 Recommended Bearing Pressures on Engineered Fill (Option One)

Engineered Fill may be used, to correct irregularities in the design subgrades or raise the subgrade elevation between the till and the footings. For this Site, if Engineered Fill is used below footings, then the recommended bearing pressures on Engineered Fill founded on approved competent subgrade are the same as for the native undisturbed glacial till (i.e., 150 kPa factored ULS and 100 kPa SLS). Again, this assumes a maximum tolerable differential settlement in the order of 19 mm and a maximum tolerable total settlement in the order of 25 mm.

Based on our conversation with the Client, up to 1.0 m grade raise is anticipated to be at the Site, therefore 40 mm of settlement could be expected across the Site from the grade raise alone. However, considering the footing will be supported on Engineered Fill in contact with undisturbed glacial till, the settlement of the structure is not a concern. **Please refer to our letter entitled, "Proposed Commercial Development 1.0 m Grade Raise, 2095 Dilworth Road, Kars, ON", dated July 18, 2024 provided in Appendix G.** If grade raises, larger dimensioned footings, or higher bearing pressures are used, then Englobe should be retained to perform additional fieldwork, testing, and Engineering analysis to support a specific settlement estimate on a case-by-case basis.

Designers and Contractors must ensure that any Engineered Fill used to repair locally damaged subgrades below the footings and above the glacial till has sufficient lateral extent. At a minimum, the Engineered Fill beneath foundations should extend laterally a distance of 0.3 m beyond the edge of the footing and then be sloped downward and outward at 1H:1V slope. Designers and Contractors are cautioned that the resultant excavation can be quite large if a significant thickness of Engineered Fill is required. Alternatives would be to lengthen foundation walls and step footing down, if necessary.

Subgrade preparation below Engineered Fill will be similar to that for footings on glacial till as noted above. The exposed surface should be examined by the Geotechnical Engineer or a qualified technologist working under the supervision of a Geotechnical Engineer to assess the competency. Engineered Fill must be treated in strict accordance to the requirements in Section 6.10.1.

6.4.3 Piled Foundations (Option Three)

If the proposed future structures require higher bearing capacities than those provided above for conventional spread or strip footings as per **Option One** and **Option Two** or if Designers went to keep excavations and dewatering to a minimum, then **Option Three** would include the use of piled foundations.

Steel H-Piles or concrete-filled steel end-bearing pipe piles driven to refusal on bedrock are considered viable deep foundation options for this Site. Bedrock was encountered approximately 5.8 m depth, corresponding to elevation of 81.9 masl in BH21-05, however the bedrock elevation in BH21-02 to BH21-04 the is unknown. If piles are considered during detailed design phase, an additional geotechnical investigation and laboratory analysis will be required to confirm the depth, quality of the bedrock.

For such end-bearing piles driven to bedrock refusal, a SLS condition for a nominal 25 mm of settlement is not applicable. The following table presents typical values of the anticipated factored bearing resistance of some available pile sizes under ULS conditions. This table is intended to provide assistance to the designer in estimating approximate quantities and possible layout of piles within the structure in conceptual/preliminary design stage. It should be noted that the actual achievable pile resistance depends on several parameters as discussed below and could vary considerably across the Site, therefore supplementary geotechnical investigation and review of conceptual foundation design are recommended for detailed design.

Table 6-2 Typical Preliminary End-Bearing Capacities of Common Piles

Pile Type	Pile Designation Imperial (Metric)	Preliminary Factored Axial Capacity at ULS
H-Piles	HP8x36 (HP200x53)	500 to 600 kN
	HP10x57 (HP250x85)	750 to 900 kN
	HP12x53 (HP310x110)	900 to 1100 kN

Pile Type	Pile Designation Imperial (Metric)	Preliminary Factored Axial Capacity at ULS
Concrete Filled Pipe Piles	7” x 0.500” (117 x 12.7)	600 to 700 kN
	9 5/8” x 0.545” (244 x 13.8)	900 to 1100 kN

On private construction projects such as this, it is typical for the piling to be performed under a design-build type contract. Given the required resistances provided by the Structural Engineer, Piling Contractors will provide the most economical piles depending on the equipment and material availability. The values provided above are approximate because of anticipated variability and depth of bedrock conditions across the Site, the methods used by Piling Contractors to drive the piles and determine pile capacity could vary somewhat from one Contractor to another. However, the preliminary values provided may be considered for preliminary estimation of the number of piles required.

For short piles driven to bedrock refusal, the design is not expected to be governed by the SLS conditions. For preliminary design purposes the settlement of piles driven to sound bedrock under SLS conditions is generally expected to be less than 5 mm (excluding the elastic deformation of the piles themselves), and less than 10 mm in total settlement.

The Piling Contractor will need to confirm the estimated pile capacity considering the driving energy of their proposed equipment using approved empirical methods at the outset of the Project. The Piling Contractor’s piling calculations should be carried out according to Section 18.2 of the Canadian Foundation Engineering Manual (CFEM-2006). Typical piling calculations would include the Hiley formula, wave equation, or other methods based on the Contractor’s equipment. The Geotechnical Engineer must be retained to review and approve the piling calculations prior to mobilization and confirm the development of the necessary piling refusal criteria for use with this Project at the onset of piling operations.

Englobe recommends that the installation of all piles be witnessed and reviewed by a Geotechnical Engineer, or a qualified Technician acting under the supervision of a Geotechnical Engineer on a full time basis to verify the tip elevation, location, verticality, and to ensure that the design set criteria and the required pile capacity has been achieved. Pile splices will require inspection by a CWB welding inspector.

It is recommended that Pile Driving Analysis (PDA) be performed on a minimum of 10% of the piles and be completed at onset and production stages of the Project. First at the onset of pile driving to confirm the set criteria established for this Project; and secondly, on any piles that are considered suspect. In addition, re-striking of all piles is recommended for this Site, to ensure that uplift of adjacent piles is avoided.

Lateral Resistance

Based on the soils that are present on the Site and the expected shallow pile depth it is not recommended to rely on the passive earth pressure to resist lateral forces. The deflection required to mobilize the passive earth pressures will likely exceed any deflection tolerances of the structure. Furthermore, short steel H-piles and pipe piles should not be considered as moment-resisting piles.

Resistance to lateral forces on the structure, if required, are recommended to be achieved using battered piles, or with the use the use of rock anchors.

Uplift Resistance

If uplift or overturning forces need to be resisted, this resistance should be provided by considering the dead weight of the structural elements, increasing the dead weight of the structure, or alternatively, with the use of grouted rock anchors.

In the case that grouted rock anchors are considered, they may be designed based on a frictional stress between grout and intact limestone bedrock. Based upon typical published values and a conservative approach, a conservative allowable working stress value of 600 kPa can be used to calculate the length of the required bond zone in “sound bedrock”. The bond zone must be entirely within "sound bedrock" which is below the weathered zone. An allowance for a weathered rock zone of at least 1.0 m in each hole should be considered.

Designing in accordance with the Limit States Design method, Designers may take the approach that working stress value is approximately equivalent to the SLS value. The ULS and SLS must be based upon both performance and structural criteria. However, based upon typical published values, the unfactored ULS values may be approximately 750 kPa to more than 1000 kPa. As per CFEM2006, a geotechnical resistance factor of $\Phi = 0.3$ should be applied to the empirical unfactored ULS values. Higher stress values may be available; however, performance load testing in the field will be required to prove the capacities. If performance testing is carried out at the outset of the Project, then a resistance factor of $\Phi = 0.4$ could be applied.

In order to mobilize the shear stress in the rock, the load at the top of the anchor must be properly transferred through the overburden and weathered rock to the bond zone to prevent progressive grout fail and ensure proper performance. Therefore, a "free length" is required through the foundation element, overburden soil, the weathered rock zone, and down to the top of the bond zone.

The mass of rock mobilized in limestone may be assumed to be based upon a 60° cone drawn upward from a point located at the lower one-third point of the bond zone and spaced such that the theoretical cones do not overlap. Designers should review the spacing of anchors and take into account any overlapping cones (i.e., avoid doubling-up on rock mass calculations for overlapping cones). The bulk unit weight of bedrock may be assumed to be approximately 26 kN/m3. The corresponding buoyant unit weight would be approximately 16 kN/m3. It is recommended that Designers consider the water level to be near the surface for rock anchor designs, and therefore, use submerged unit weights for the rock mass calculations.

6.5 Frost Protection

All footings for heated structures must be provided with a minimum of 1.5 m of earth cover, and 1.8 m of earth cover for unheated or isolated structures in the Kars area. Otherwise an equivalent insulation detail would be required in order to provide adequate protection against frost action. Where soil cover cannot be provided, an insulation detail should be designed or approved by a Geotechnical Engineer. Contractors must be aware that this detail may be such that the insulation may need to be placed below the footing and then the footing poured on top, and therefore pre-approval is recommended to ensure excavations and backfill are properly planned. The glacial till on this Site should be considered to be frost susceptible.

Should construction take place during winter, surfaces that support foundations or Engineered Fill must be protected by Contractors against freezing for the entire duration of construction or until adequate soil cover is in place. Backfill soils should not be placed in a frozen condition or placed on frozen subgrades.

6.6 Seismic Site Classification

In accordance with the Ontario Building Code (OBC-2012), structures designed under Part Four of the Code must be designed to resist a minimum earthquake force. A seismic site class ‘D’ is estimated for the Site in based on the underside of foundation or top of pile cap at 1.5 mbgs at the Site.

If a higher seismic site class is desired, shear wave velocity testing is recommended. The shear wave velocity should be measured from 30 m below the underside of the proposed or contemplated footings, in accordance with Commentary J of NBCC-2015. The seismic site class can then be adopted by referring to Table 4.1.8.4.A (OBC-2012) and selecting the site class that corresponds to the estimated shear wave velocity for the Site. Shear wave velocity testing should be carried out by a qualified geophysics subcontractor.

6.7 Floor Slabs

Englobe was not provided with any design criteria for floor slab loadings and therefore we have assumed that floor slabs are lightly loaded with no heavy racking or process machinery that require specific support. A typical floor slab loading for a lightly loaded slab on grade would involve a maximum pressure of 24 kPa. If this is not the case, then Englobe should be retained to perform additional consulting in regard to design of the floor slab. For design purposes and based upon a properly prepared native subgrade surface covered with 200 mm of Ontario Provincial Standard Specification (OPSS) 1010 “Granular A”, a typical preliminary modulus of subgrade reaction appropriate for the slab design would be approximately 25,000 kN/m3 on Engineered Fill compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD). Alternative values would require additional analysis and testing.

A capillary moisture barrier consisting of a layer of either 19 mm clear stone or an OPSS 1010 ‘Granular A’ at least 200 mm thick should underlie the slab. This layer should be compacted to 100% of its SPMDD and placed on approved subgrade surfaces.

If floor coverings are to be used, vapour barriers are also recommended to be incorporated beneath the slab. Floor toppings may be impacted by curing and moisture conditions of the concrete. Floor finish manufacturer’s specifications and requirements should be consulted, and procedures outlined in the specifications should be followed. The slabs should be free floating and should not be tied into the foundation walls. The placement of construction and control joints in the concrete should be in accordance with generally accepted practice.

In the case that piled foundations are used, the floor slabs should be designed as a structural floor slabs fully supported on pile caps and grade beams to avoid differential settlement. Between the floor and the foundation wall.

6.8 Permanent Drainage

Under floor drainage is typically not necessary for structures with no basement level and which have floor slabs at a minimum of 0.3 m above the exterior grade. Perimeter drainage around the commercial building is recommended. Perimeter drainage may consist of either a conventional weeping tile with clear stone and geotextile, or alternatively a compositive drainage blanket may be used. Regardless of the drainage system used, it is recommended that the foundation walls be backfilled with a free-draining, non-frost-susceptible soil such as an OPSS 1010 “Granular B, Type I”. The perimeter drain should be connected to a frost-free outlet for year-round drainage.

If the floor slab is set at or below the exterior grade, then both perimeter and under-floor drainage is recommended. Perimeter and underfloor drainage systems should not be connected.

6.9 Backfill

All new fill soils that underlie floor slabs, footings, are in building interiors, or other structural applications is considered as Engineered Fill and must be treated as such.

6.9.1 Engineered Fill

For this Project, Engineered Fill may be required to repair locally damaged subgrade below footings and floor slabs, and for interior foundation wall backfill. If **Option One** or **Option Two** are used, then mass Engineer Fill will be required to raise the footing elevations in the area where the undisturbed native glacial till subgrade is relatively deep. If the Designers select to construction the building foundations on Engineered Fill overlying the native till, then there will be significant Engineered Fill required for this Project. Engineered Fill must meet the strict requirements as shown below:

- The proposed material must be tested for grain size and Proctor and reviewed and approved by the Geotechnical Engineer before being considered as Engineered Fill. Typically, a crushed well-graded material such as an OPSS 1010 “Granular A” or “Granular B Type II” type material is suitable. However, other suitable granular materials may be proposed and considered depending on the Site-specific conditions;

- Prior to placing any Engineered Fill, all unsuitable fill materials must be removed, and the subgrade approved by the Engineer. Any deficient areas should be repaired prior to placement;

- Engineered Fill should be placed in maximum loose lifts of 300 mm and adequately compacted to achieve 100% of its Standard Proctor Maximum Dry Density (SPMDD). Engineered Fill must have full-time compaction testing by geotechnical personnel.

- At a minimum, the Engineered Fill beneath foundations should extend laterally a distance of 1.0 m beyond the edge of the footings and then be sloped downward and outward at a 1H:1V slope. Designers and Contractors are cautioned that the resultant excavation can be quite large if a significant thickness of Engineered Fill is required. For this Project, of **Option One** is used then the Engineered Fill pad will extend well beyond the footprint of the building.

6.9.2 Exterior Foundation Backfill

Any backfill placed against exterior foundations should be a free draining granular material meeting the grading requirements of an OPSS 1010 “Granular B, Type I” or “Granular B, Type II”. Exterior foundation backfill should be placed and compacted as outlined below:

- Backfill should not be placed in a frozen condition, or place on a frozen subgrade;
- Backfill should be placed and compacted in maximum loose lift thickness compatible with the selected construction equipment, but not thicker than 0.3 m;
- In landscaped areas the upper 0.3 m of backfill below landscape details should be a low permeable soil to reduce surface water infiltration;
- Backfill should be placed uniformly on both sides of the foundation walls to avoid build-up of unbalanced lateral pressures;
- For backfill that would underlie paved areas, sidewalks or exterior slabs-on-grade, each lift should be uniformly compacted to achieve 98 % percent SPMDD.
- For backfill on the building exterior that would underlie landscaped areas, each lift should be uniformly compacted to at least 95 % SPMDD;
- Exterior grades should be sloped away from the foundation wall, and roof drainage downspouts should be placed so that water flows away from the foundation wall;
- Entrance slabs should be placed on frost walls or alternatively have insulation details developed to prevent frost heaving at the entrances to buildings;
- In areas where the building backfill underlies a pavement, sidewalk, or other hard landscaping, the excavation should have a frost taper incorporated to prevent differential heaving around the building.

6.10 Future Settlement of Underground Utilities and Hard Surface Features

Considering 1.0 m grade raise is anticipated to be at the Site, settlement of 40 mm could be expected across the Site. Therefore, any future underground utilities and hard surface features, such as retaining walls or hard landscaping, will continue to be subject to settlement and will need to be designed with flexibility and tolerance to increased settlement. **Please refer to our letter entitled,**

“Proposed Commercial Development 1.0 m Grade Raise, 2095 Dilworth Road, Kars, ON”, dated July 18, 2024 provided in Appendix G.

6.11 Underground Utilities

The recommendations within this section are intended to be a supplement to, and not a replacement of, the most recent local municipal requirements.

6.11.1 Bedding and Cover

The following are recommendations for service trench bedding and cover materials:

- Bedding for buried utilities should consist of an OPSS 1010 "Granular A" material and placed in accordance with municipal requirements over undisturbed subgrade native subsoils;

- The use of clear stone is not recommended for use as pipe bedding. The voids in the stone may result in a low gradient water flow and infiltration of fines from the surrounding soils and cover materials, causing settlement and loss of support to pipes and associated structures;

- The cover material should be a service sand material or an OPSS 1010 "Granular A". The dimensions should comply with pertinent specification section;

- The bedding, springline, and cover should be compacted to at least 95% SPMDD;

- Compaction equipment should be used in such a way that the utility pipes are not damaged during construction.

6.11.2 Trench Backfill

Backfill above the cover for buried utilities should be in accordance with the following recommendations:

- For service trenches underlying pavement areas, the backfill should be placed and compacted in uniform lift thickness compatible with the selected compaction equipment and not thicker than 300 mm. Each lift should be compacted to a minimum of 98% SPMDD;

- The backfill placed in the upper 0.3 m below the pavement subgrade elevation should be compacted to a minimum of 100% SPMDD;

- Excavation backfill should attempt to match texture of the existing adjacent soils. If imported materials are used, side slopes with frost tapers are recommended. Frost tapers should be back sloped at 10H:1V through the frost zone (i.e., down to 1.8 m below finished grade);
- During backfilling, care should be taken to ensure the backfill proceeds in equal stages simultaneously on both sides of the pipe;
- No frozen material should be used as backfill; neither should the trench base be allowed to freeze.

The quality and workmanship in the construction is as important as the compaction standards themselves. It is imperative that the guidelines for the compaction be followed for the full depth of the trench to achieve satisfactory performance.

6.11.3 Clay Seals

Clay seals should be incorporated into the design of the any utility trenches. If clay seals are not used, then there is the potential for the trench to act as a drain and dewater the local clays causing settlement. The location of the clay seals should be at a frequency prescribed by the Civil Engineer.

Ontario Provincial Standard Drawing (OPSD) 1205 and OPSD 802.095 are referred to both the Designers and Contractor for guidance on clay seals. Acceptable imported clay material may be used for the construction of the clay seals.

6.12 Corrosion Potential of Soils

Analytical testing was carried out on two (2) soil sample collected from the boreholes (MW21-05 SS2 and MW21-06 SS2) to evaluate corrosion potential of the subsurface soils. The selected soil samples were tested for pH, resistivity, chlorides, sulphides, and sulphates. The test results are summarized in the following table.

Table 6-3 Corrosion Parameter Results

Parameter	Result	
	BH21-05, SS2 (0.9 – 1.5 mbgs)	MW21-06, SS2 (0.9 – 1.5 mbgs)
pH	7.54	6.91
Chloride (ug/g)	16	45
Sulphate (ug/g)	122	15
Resistivity (Ohm-m)	36.2	63.5
Sulphide (%)	< 0.04	< 0.04

Parameter	Result	
	BH21-05, SS2 (0.9 – 1.5 mbgs)	MW21-06, SS2 (0.9 – 1.5 mbgs)
Redox Potential (mV)	330	286

The American Water Works Association (AWWA) publication ‘Polyethylene Encasement for Ductile-Iron Pipe Systems’ ANSI/AWWA C105/A21.5-10 dated October 1, 2010 assigns points based on the results of the above tests. A soil that has a total point score of 10 or more is considered to be potentially corrosive to ductile iron pipe. Based on the test results and using the guidelines provided by AWWA publication ANSI/AWWA C105/A21.5-10, the tested soil specimens from BH21-05 and MW21-06 received corrosivity scores of less than 10. Thus, based on the results for the test soil specimens, buried ductile iron components are not expected to require corrosion protection measures at the Site.

The analytical results of the soil samples were also compared with applicable Canadian Standards association (CSA Standards A23.1-04) standards, given in Table 6-4 below.

Table 6-4 Additional Requirement for Concrete Subjected to Sulphate Attack

Class of Exposure	Degree of Exposure	Water soluble Sulphate in soil sample (%)	Cementing Material to be used
S-1	Very Severe	> 2.0	HS or HSb
S-2	Severe	0.20 - 2.0	HS or HSb
S-3	Moderate	0.10 - 0.20	MS, MSb, LH, HS, or HSb

The sulphate content analyses for the tested soil samples indicate a maximum sulphate concentration of 122 ug/g (0.0122 %) in soil, as shown in Table. This suggests there is negligible risk for sulphate attack on concrete material at the Site. Accordingly, conventional GU or MS Portland cement may be used in the construction of any proposed concrete elements.

6.13 Recommended Asphalt Pavement

All existing fill soils should be excavated down to the proposed new subgrade level. The final subgrade should be proof rolled to look for deflection, soft spots, or local anomalies. Typically, a heavy-duty steel drum roller or a loaded dump truck is sufficient for proof rolling. Proof-rolling of proposed subgrades should be witnessed by geotechnical staff. Any non-performing areas should be sub-excavated and replaced with an appropriate new fill soil. An appropriate fill soil would be a free-draining non-frost susceptible soil similar to a “Granular ‘B Type I” or “Granular B Type II” material.

Newly backfilled soils should attempt to match the texture of the existing adjacent soils. Localized sub-excavations should have frost tapers to avoid concentrated frost heaves across the roadway at the transition zones between sub-excavated and un-excavated subgrades.

In order to accommodate the recommended thicknesses, designers will need to review existing and proposed grades and determine where stripping or filling is necessary. Drainage of the pavement layers is important. Surface runoff should be directed to storm sewers or surface ditches where possible. The subgrade surface and each layer of the pavement section should also be provided with a suitable cross fall (approximately 3%) to prevent water from ponding on each layer. The installation of subdrains may be recommended as designs progress based on the surrounding topography and drainage conditions to assist in the long-term performance of the pavement structures. Non-woven geotextile as a separation is prudent based on the observations during proof rolling.

For the proposed pavement base and subbase courses the material should consist of a “Granular A” and “Granular B Type II” material, respectively. The material should be placed in maximum loose lifts of 300 mm and compacted to 100 % of its SPMDD.

Sufficient field-testing should be carried out during construction to assess compaction of each lift of the pavement structure layers. This should be accompanied by laboratory testing of the proposed granular materials and asphalt materials.

In the case of winter work, which is not recommended, no frozen material should be used as backfill, and backfill should not be placed on frozen subgrades.

Based on the results of the field and laboratory testing, Englobe is recommending the following preliminary minimum pavement sections. It is important to note that at the time of this investigation, Englobe has not been provided with any traffic counts, or level of service requirements or equipment loadings for pavement structures. The pavement sections being provided are what we would consider to be suitable for a private development. Table 6-5 below summarizes proposed asphalt designs for the parking lot and fire route respectively.

Table 6-5 Recommended Minimum Pavement Sections

Material	Layer Thickness
Parking Lots – Light Duty (Parking Stalls)	
Asphalt Wearing Course	50 mm
Well Graded Granular Base Course (Granular ‘A’)	150 mm
Well Graded Granular Sub-Base Course (Granular ‘B’ Type II)	300 mm
Non-woven geotextile for material separation	Terrafix 270R or equivalent

Material	Layer Thickness
Approved Subgrade	Undisturbed Native Organic-free Silt/Silty Sand or Silty Clay or Glacial Till
Parking Lots – Heavy Duty (Aisles and Fire Routes)	
Asphalt Wearing Course	40 mm
Asphalt Binder Course	50 mm
Well Graded Granular Base Course (Granular ‘A’)	150 mm
Well Graded Granular Sub-Base Course (Granular ‘B’ Type II)	450 mm
Non -woven geotextile for material separation	Terrafix 270R or equivalent
Approved Subgrade	Undisturbed Native Organic Free Silt/Silty Sand or Silty Clay or Glacial Till

Annual or regular maintenance will be required to achieve maximum life expectancy. Generally, the asphalt pavement maintenance will involve periodic crack sealing and repair of local distress.

It is important to emphasize that the pavement sections described above are for a proposed end use condition that is expected to include light vehicular traffic and occasional service trucks. It may be necessary to over-design these sections if they are intended to support heavy construction equipment throughout construction.

7 Monitoring During Construction

Englobe requests to be retained once the plans and specifications are finalized to review the documents and ensure the recommendations in this report are adequately addressed.

The recommendations presented in this report are based on the assumption that an adequate level of construction monitoring by qualified geotechnical personnel during construction will be provided. Based on our understanding of the scope of the Project, an adequate level of construction monitoring is considered to be as follows:

- Review and approval of all footing subgrades by geotechnical personnel;
- Proof rolling, review, and approval of subgrades below the floor slab;
- Laboratory testing and pre-approval of FILL material that are proposed to be used on Site;
- Full time compaction testing of Engineered Fill and part time compaction testing of backfill for exterior foundation walls;
- Periodic testing of cast in place concrete;

An important purpose of providing an adequate level of monitoring is to check that recommendations, based on data obtained at the discrete borehole locations, are relevant to other areas of the Site.

8

8 Closure

A description of limitations which are inherent in carrying out site investigation studies is given in **Appendix A** and forms an integral part of this report.

We trust this report meets your present requirements. Should you have any questions, please do not hesitate to contact our office.

Sincerely,
Englobe

Appendix A
Limitations of Report

LIMITATIONS OF REPORT

GEOTECHNICAL STUDIES

The data, conclusions and recommendations which are presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. Conditions can also change with time. It is recommended practice that Englobe Corp, A Division of Englobe. be retained during construction to confirm that the subsurface conditions throughout the Site do not deviate materially from those encountered in the boreholes.

The design recommendations given in this report are applicable only to the Project, as described in the text, and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid. Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the Site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this Project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

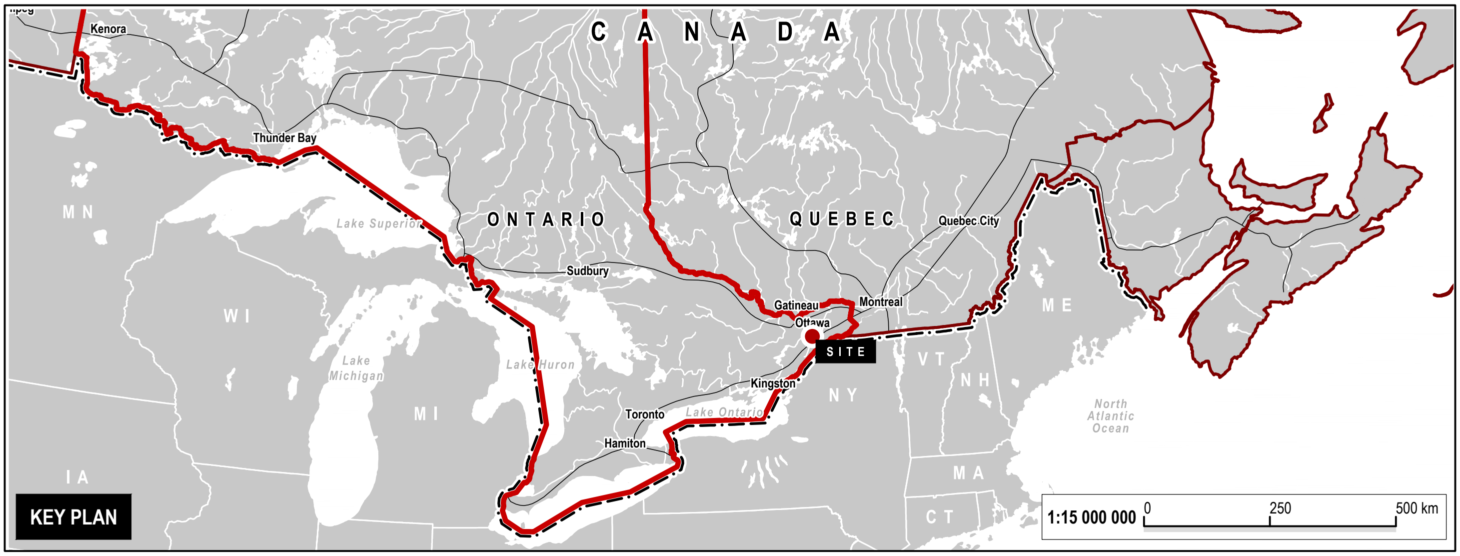
Any results from an analytical laboratory or other subcontractor reported herein have been carried out by others, and Englobe Corp. cannot warranty their accuracy. Similarly, Englobe Corp cannot warranty the accuracy of information supplied by the Client.

Appendix B

Figure 1: Site Location Map

Figure 2: Borehole Location Plan Fence Diagram

ENGLOBE



Source:
Bing Maps 2021

Note

1. This drawing shall be read in conjunction with the associated technical report.

Revision	Date	Issue	Approval
A	04/30/2024	Final	SR

Client: **Dilworth Development Inc.**

Site: **2095 Dilworth Road, Kars, ON**

Report Title: **Geotechnical Investigation Report**
Proposed Commercial Subdivision
and Private Servicing

Drawing Title: **Site Location Map**

Designed By: S.R.

Date: March 2021

Drawn By: K.M.

Project No.: 02101208.000

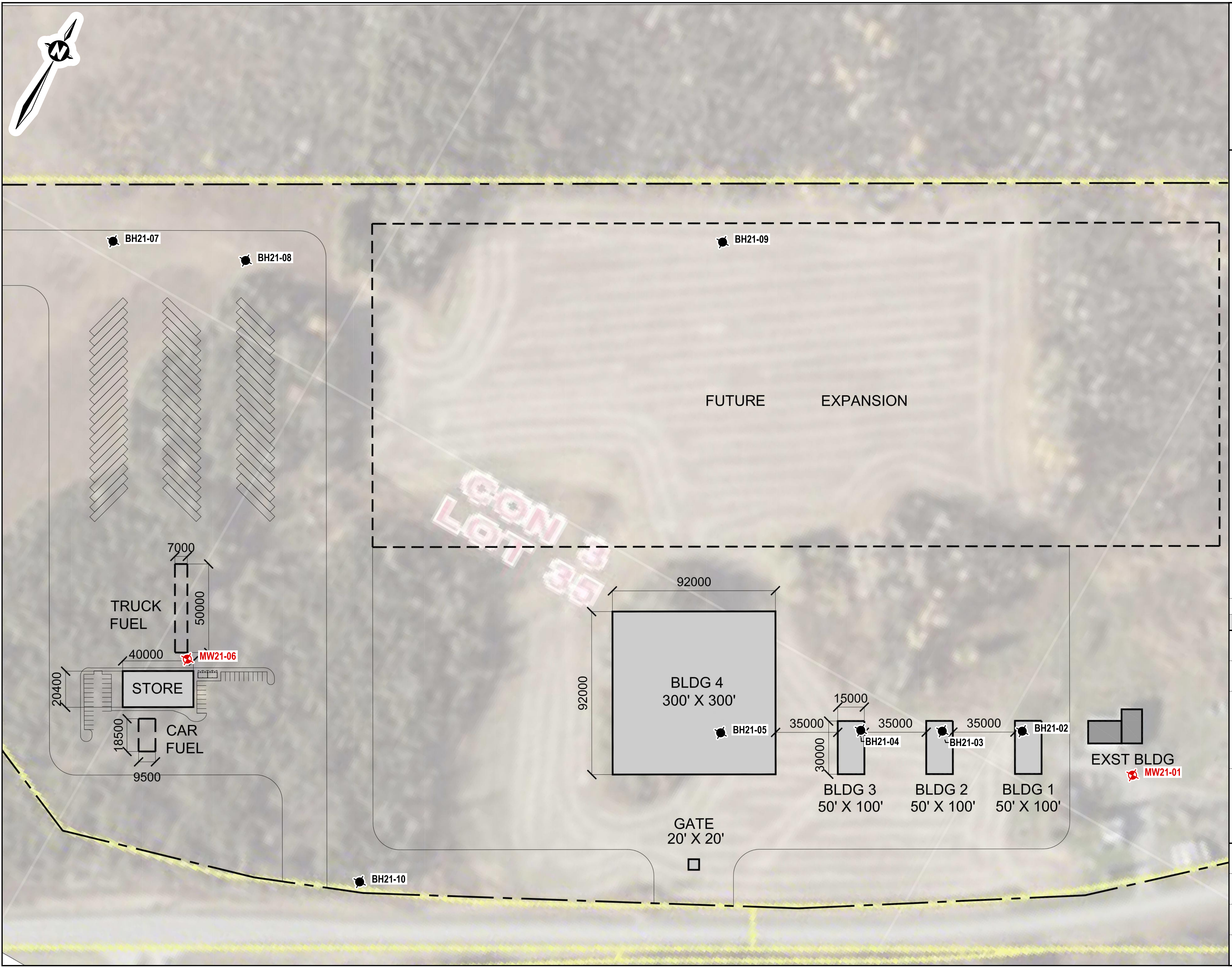
Approved By:

Figure No.: 1

Scale: As shown

Folder: C:\DST\02101208.000 2095 Dilworth\Geotechnical Investigation\DWGs Thursday, March 18, 2021 @ 11:41 by Kris Morin

Drawing: 1 site location map.dwg



ENGLOBE

Note

1. This drawing shall be read in conjunction with the associated technical report.

Legend

●	Approximate location of borehole
⊕	Approximate location of monitoring well

1:2 000

0 20 40 60 80 100 m

0 1:1 5 cm

Scale accurate when printed at 100% using paper size ANSI full bleed B (11.0 x 17.0 inches)

A	04/30/2024	Final	SR
Revision	Date	Issue	Approval

Client

Dilworth Development Inc.

Site

2095 Dilworth Road, Kars, ON

Report Title

Geotechnical Investigation Report
Proposed Commercial Subdivision
and Private Servicing

Drawing Title

Borehole Location Plan

Designed By	S.R.	Scale	As shown
Drawn By	K.M.	Date	March 2021
Approved By		Project No.	02101208.000
Figure No.	2		

Drawing: **2 site plan**.dwg *Folder:* **C:\DST\02101208.000 2095 Dilworth\Geotechnical Investigation\DWGs** Monday, March 29, 2021 @ 15:21 by Kris Morin

Appendix C

List Of Symbols And Definitions
Borehole Logs

LIST OF SYMBOLS AND DEFINITIONS FOR GEOTECHNICAL SAMPLING AND COMMON LITHOLOGIES

The following is a reference sheet for commonly used symbols and definitions within this report and in any figures or appendices, including borehole logs and test results. Symbols and definitions conform to the standard proposed by the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) wherever possible. Discrepancies may exist when comparing to third-party results using the Unified Soil Classification System (USCS).

PART A – SOILS

Standard Penetration Test (SPT) 'N'

The number of blows required to drive a 50-mm (2 in) split barrel sampler 300 mm (12 in). The standard hammer has a mass of 63.5 kg (140 lbs) and is dropped vertically from a height of 760 mm (30 in). Additional information can be found in ASTM D1586-11 and in §4.5.2 of the CFEM 4th Ed.

For penetration less than 300 mm, 'N' is recorded with the penetration that was achieved.

Non-Cohesive Soils

The relative density of non-cohesive soils relates empirically to SPT 'N' as follows:

Relative Density	'N'
Very Loose	0 – 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	> 50

Cohesive Soils

The consistency and undrained shear strength of cohesive soils relates empirically to SPT 'N' as follows:

Consistency	Undrained Shear Strength (kPa)	'N'
Very Soft	< 12	0 – 2
Soft	12 – 25	2 – 4
Firm	25 – 50	4 – 8
Stiff	50 – 100	8 – 15
Very Stiff	100 – 200	15 – 30
Hard	> 200	> 30

PART B – ROCK

The following parameters are used to describe core recovery and to infer the quality of a rockmass.

Total Core Recovery, TCR (%)

The total length of solid drill core recovered, regardless of the quality or length of the pieces, taken as a percentage of the length of the core run.

Solid Core Recovery, SCR (%)

The total length of solid, full-diameter drill core recovered, taken as a percentage of the length of the core run.

Rock Quality Designation, RQD (%)

The sum of the lengths of solid drill core greater than 100 mm long, taken as a percentage of the length of the core run. RQD is commonly used to infer the quality of the rockmass, as follows:

Rockmass Quality	RQD (%)
Very Poor	< 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	> 90

Weathering

The terminology used to describe the degree of weathering for recovered rock core is defined as follows, as suggested by the *Geological Society of London*:

Completely weathered: All rock material is decomposed and/or disintegrated to soil. The original mass structure is largely intact.

Highly weathered: More than half the rock material is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a discontinuous framework or as core stone.

Moderately weathered: Less than half the rock material is decomposed and/or disintegrates to soil. Fresh or discolored rock is present ether as a continuous framework or as core stone.

Slightly weathered: Discoloration indicates weathering of rock material and discontinuity of surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than its fresh condition.

Fresh: No visible signs of weathering.

PART C – SAMPLING SYMBOLS

Symbol	Description
SS	Split spoon sample
TW	Thin-walled (Shelby Tube) sample
PH	Sampler advanced by hydraulic pressure
WH	Sampler advanced by static weight
SC	Soil core

PART D – IN-SITU AND LAB TESTING

SOIL NAMING CONVENTIONS

Particle sizes are described as follows:

Particle Size Descriptor		Size (mm)
Boulder		> 300
Cobble		75 – 300
Gravel	Coarse	19 – 75
	Fine	4.75 – 19
Sand	Coarse	2.0 – 4.75
	Medium	0.425 – 2.0
	Fine	0.075 – 0425
Silt		0.002 – 0.075
Clay		< 0.002

The principle constituent of a soil is written in uppercase. The minor constituents of a soil are written according to the following convention:

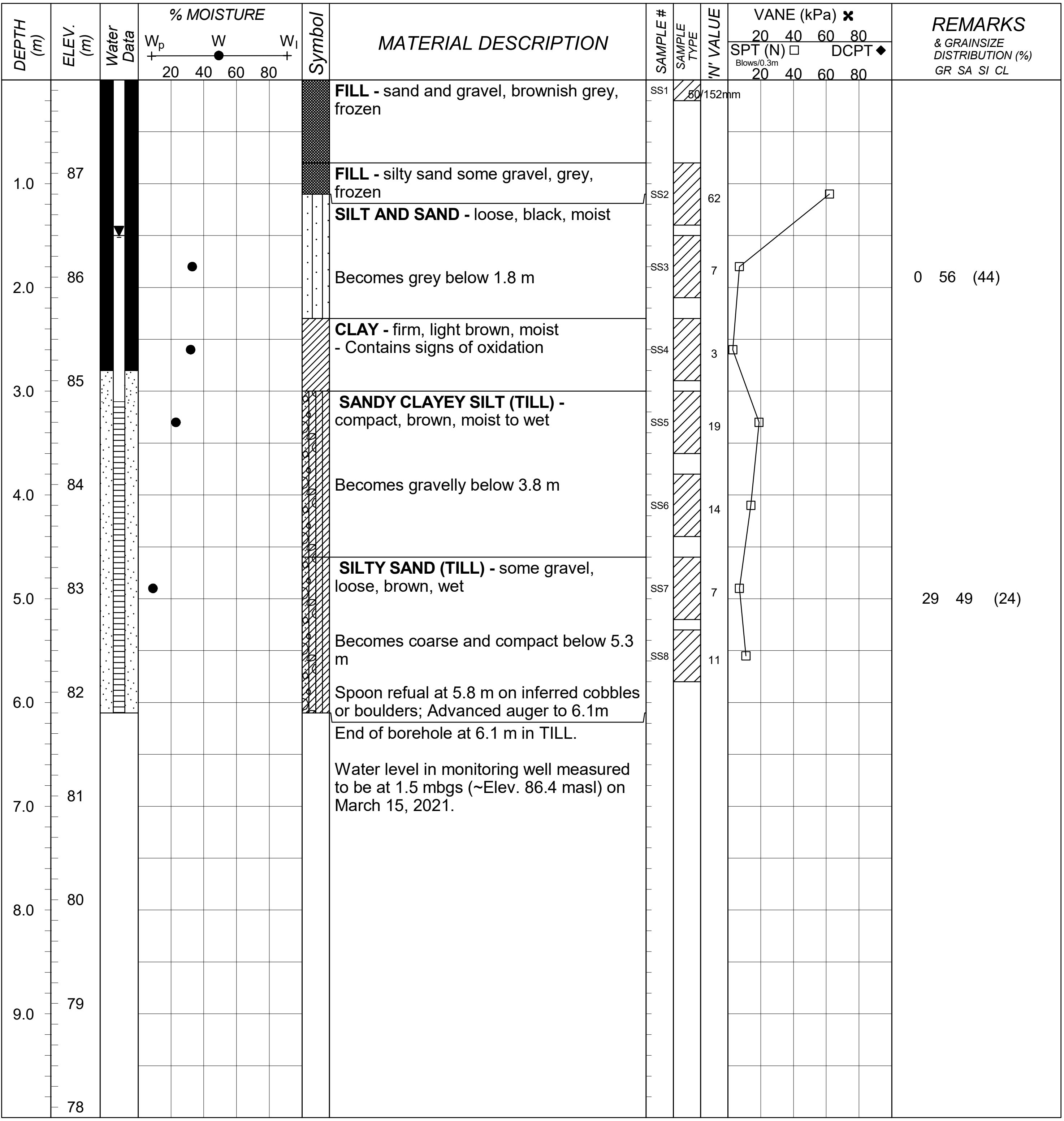
Descriptive Term	Proportion of Soil (%)
Trace	1 – 10
Some	10 – 20
(ey) or (y)	20 – 35
And	35 – 50

Eg.: A soil comprising 65% Silt, 21% Sand and 14% Clay would be described as a: Sandy SILT, Some Clay

LOG OF BOREHOLE MW21-01

DST REF. No.: **02101208**
CLIENT: **Walter Greisseier**
PROJECT: **Proposed Commercial Subdivision and Private Servicing**
LOCATION: **2095 Dilworth Road, Kars, ON**
SURFACE ELEV.: **87.90 metres**

Drilling Data
METHOD: **Hollow Stem Augers**
DIAMETER: **203 mm**
DATE: **February 16, 2021**
COORDINATES: m N, m E



FILL - sand and gravel, brownish grey, frozen

FILL - silty sand some gravel, grey, frozen

SILT AND SAND - loose, black, moist

Becomes grey below 1.8 m

CLAY - firm, light brown, moist
- Contains signs of oxidation

SANDY CLAYEY SILT (TILL) - compact, brown, moist to wet

Becomes gravelly below 3.8 m

SILTY SAND (TILL) - some gravel, loose, brown, wet

Becomes coarse and compact below 5.3 m

Spoon refual at 5.8 m on inferred cobbles or boulders; Advanced auger to 6.1m

End of borehole at 6.1 m in TILL.

Water level in monitoring well measured to be at 1.5 mbgs (~Elev. 86.4 masl) on March 15, 2021.

SAMPLE #	'N' VALUE	REMARKS
SS1	50/152mm	
SS2	62	
SS3	7	0 56 (44)
SS4	3	
SS5	19	
SS6	14	
SS7	7	29 49 (24)
SS8	11	

SAMPLE TYPE LEGEND

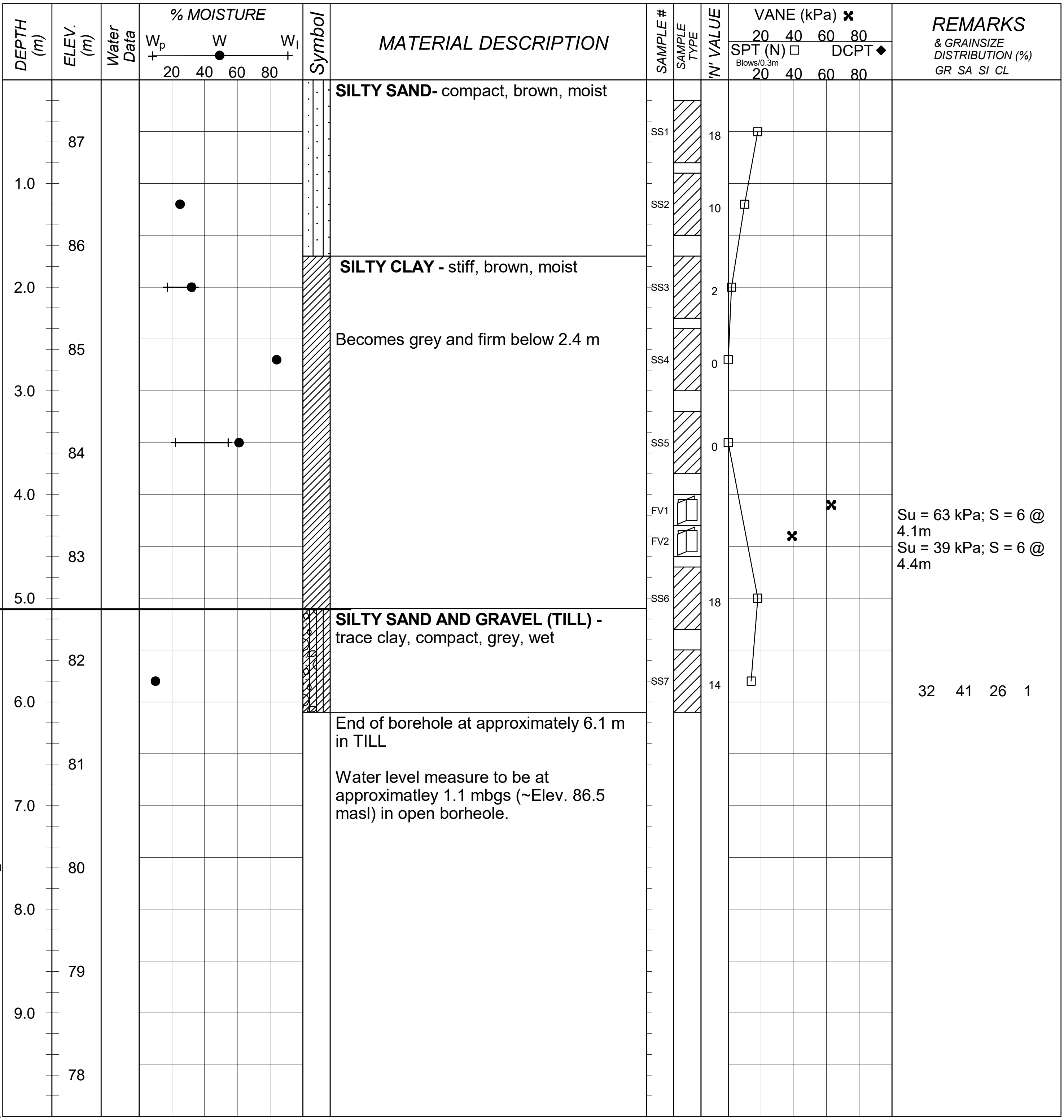
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	◫	70mm Thick Wall Tube		

ENCLOSURE 9

LOG OF BOREHOLE BH21-02

DST REF. No.: **02101208**
CLIENT: **Walter Greisseier**
PROJECT: **Proposed Commercial Subdivision and Private Servicing**
LOCATION: **2095 Dilworth Road, Kars, ON**
SURFACE ELEV.: **87.60 metres**

Drilling Data
METHOD: **Hollow Stem Augers**
DIAMETER: **203 mm**
DATE: **February 16, 2021**
COORDINATES: m N, m E



SILTY SAND- compact, brown, moist

SILTY CLAY - stiff, brown, moist

Becomes grey and firm below 2.4 m

SILTY SAND AND GRAVEL (TILL) - trace clay, compact, grey, wet

End of borehole at approximately 6.1 m in TILL

Water level measure to be at approximatley 1.1 mbgs (~Elev. 86.5 masl) in open borheole.

SAMPLE #	'N' VALUE
SS1	18
SS2	10
SS3	2
SS4	0
SS5	0
FV1	
FV2	
SS6	18
SS7	14

Su = 63 kPa; S = 6 @ 4.1m
Su = 39 kPa; S = 6 @ 4.4m

GR	SA	SI	CL
32	41	26	1

BOREHOLE (STANDARD) 02100377 2095 DILWORTH ROAD.GPJ DST_MIN.GDT 4/12/21

SAMPLE TYPE LEGEND

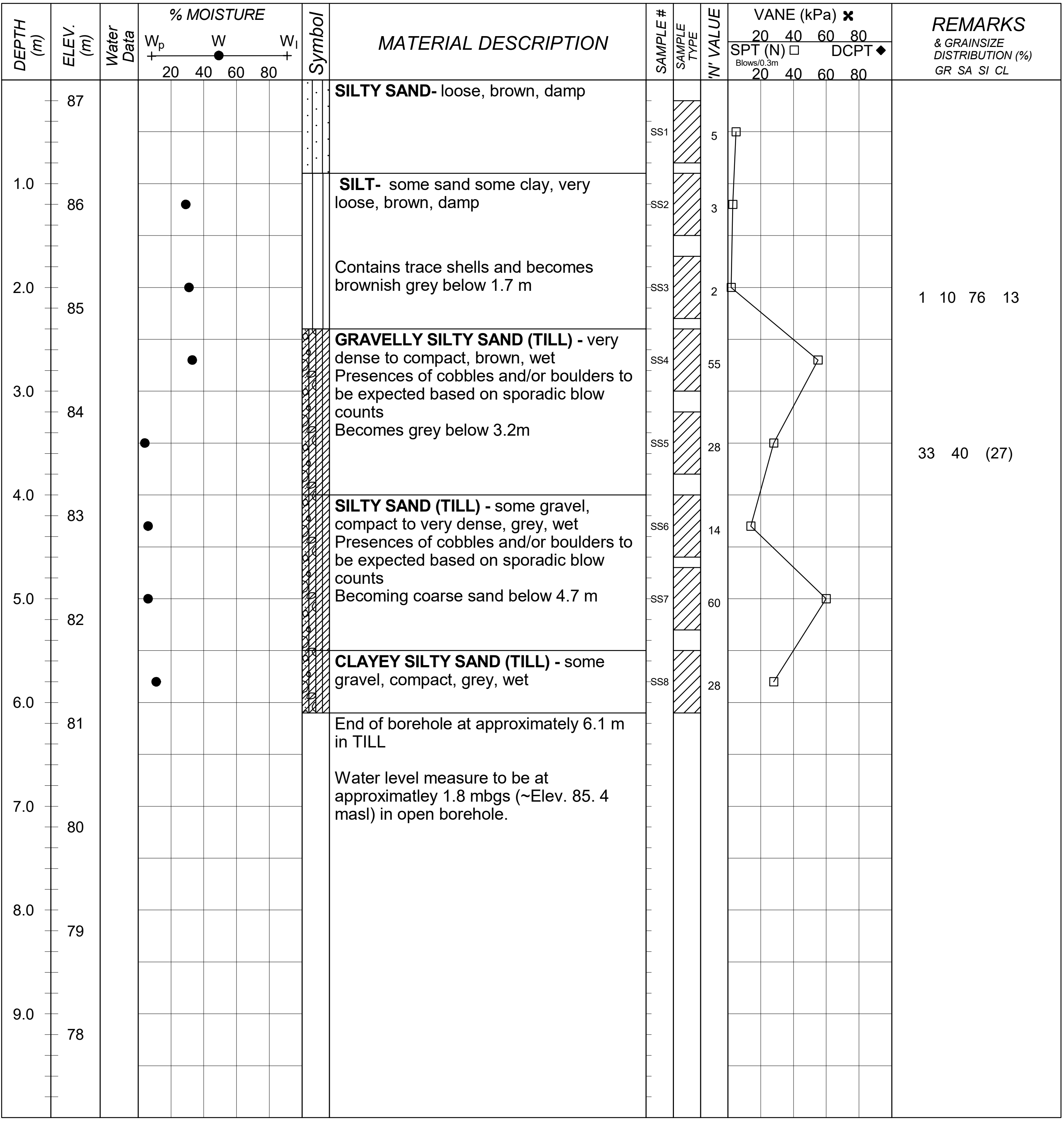
- ☰ Auger Sample
- ▨ Split Spoon Sample
- ▥ Bulk Sample
- ▩ Rock Core
- ▧ Hiller Peat Sampler
- ◫ 70mm Thick Wall Tube
- ■ Bentonite
- ▦ Sand

ENCLOSURE 1

LOG OF BOREHOLE BH21-03

DST REF. No.: **02101208**
CLIENT: **Walter Greisseier**
PROJECT: **Proposed Commercial Subdivision and Private Servicing**
LOCATION: **2095 Dilworth Road, Kars, ON**
SURFACE ELEV.: **87.20 metres**

Drilling Data
METHOD: **Hollow Stem Augers**
DIAMETER: **203 mm**
DATE: **February 16, 2021**
COORDINATES: m N, m E



SILTY SAND- loose, brown, damp

SILT- some sand some clay, very loose, brown, damp

Contains trace shells and becomes brownish grey below 1.7 m

GRAVELLY SILTY SAND (TILL) - very dense to compact, brown, wet
Presences of cobbles and/or boulders to be expected based on sporadic blow counts
Becomes grey below 3.2m

SILTY SAND (TILL) - some gravel, compact to very dense, grey, wet
Presences of cobbles and/or boulders to be expected based on sporadic blow counts
Becoming coarse sand below 4.7 m

CLAYEY SILTY SAND (TILL) - some gravel, compact, grey, wet

End of borehole at approximately 6.1 m in TILL

Water level measure to be at approximatley 1.8 mbgs (~Elev. 85. 4 masl) in open borehole.

SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
SS1	5	
SS2	3	
SS3	2	1 10 76 13
SS4	55	
SS5	28	33 40 (27)
SS6	14	
SS7	60	
SS8	28	

englobe

SAMPLE TYPE LEGEND

☰ Auger Sample	▦ Rock Core	■ Bentonite
▨ Split Spoon Sample	▧ Hiller Peat Sampler	░ Sand
▥ Bulk Sample	◫ 70mm Thick Wall Tube	

ENCLOSURE 2

LOG OF BOREHOLE BH21-04

DST REF. No.: **02101208**
CLIENT: **Walter Greisseier**
PROJECT: **Proposed Commercial Subdivision and Private Servicing**
LOCATION: **2095 Dilworth Road, Kars, ON**
SURFACE ELEV.: **87.20 metres**

Drilling Data
METHOD: **Hollow Stem Augers**
DIAMETER: **203 mm**
DATE: **February 17, 2021**
COORDINATES: m N, m E

DEPTH (m) | ELEV. (m) | Water Data | % MOISTURE Wp W Wl 20 40 60 80 | Symbol | MATERIAL DESCRIPTION | SAMPLE # | SAMPLE TYPE | 'N' VALUE | VANE (kPa) ✖ 20 40 60 80 | SPT (N) □ DCPT ◆ Blows/0.3m 20 40 60 80 | REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL

DEPTH (m): 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0
ELEV. (m): 87, 86, 85, 84, 83, 82, 81, 80, 79, 78

SILT - loose, brown, damp

Becomes clayey silt below 0.9 m

GRAVELLY SILTY SAND (TILL) - trace clay, compact, brown, wet
Presences of cobbles and/or boulders to be expected based on sporadic blow counts
Becomes grey below 2.4 m

Becomes very dense below 4.0 m

Spoon refusal at 4.7 m on inferred cobble or bouldler

Spoon refusal at 5.8 m on inferred cobble or bouldler; Advanced auger to 6.1m

End of borehole at approximately 6.1 m in TILL

Water level measure to be at approximatley 1.3 mbgs (~Elev. 85.9 masl) in open borehole.

SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
SS1	7	
SS2	1	
SS3	30	
SS4	19	
SS5	25	34 39 26 1
SS6	59	
SS7	>99	
SS8	98/305mm	

englobe

SAMPLE TYPE LEGEND

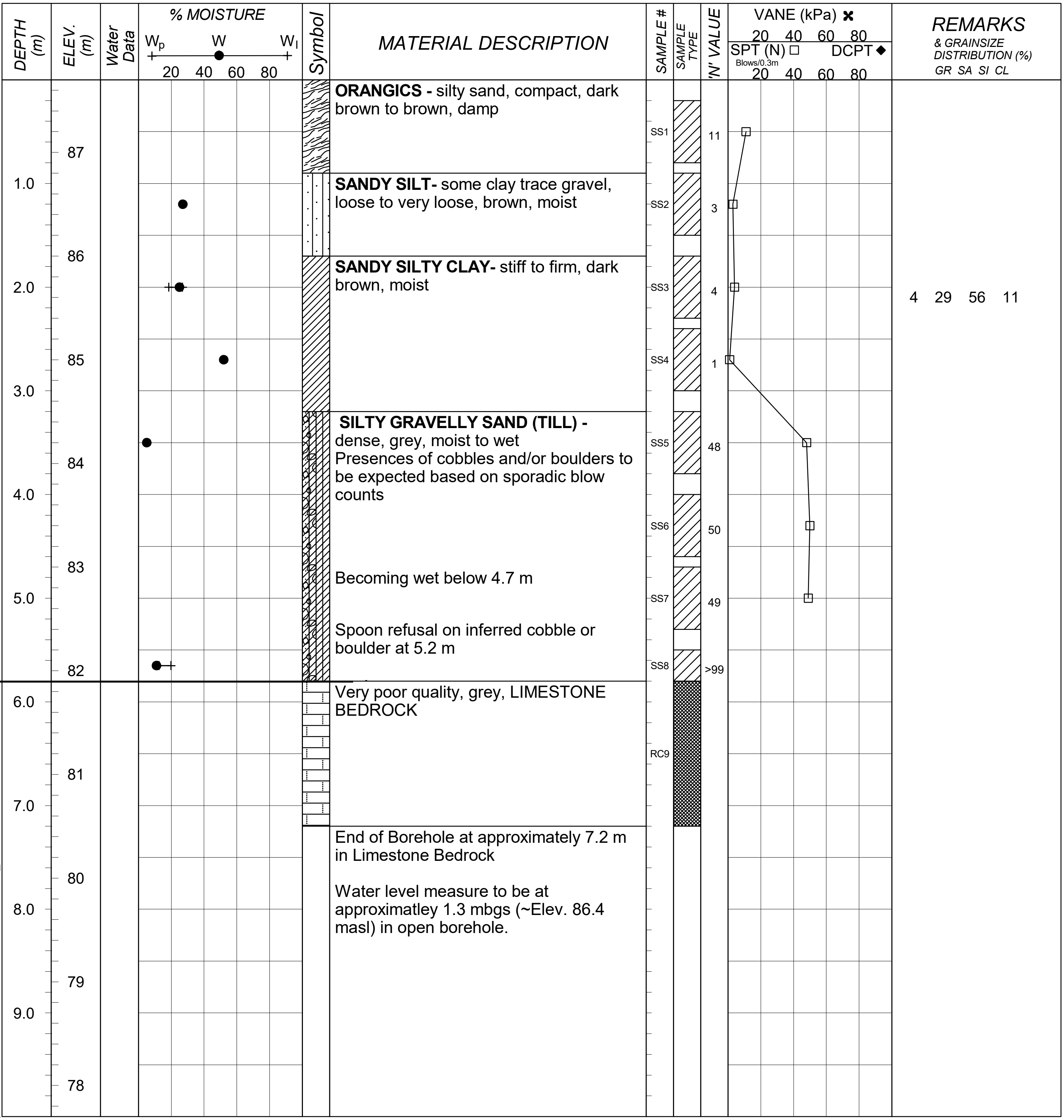
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	▧ Hiller Peat Sampler	⬚ Sand
▥ Bulk Sample	◫ 70mm Thick Wall Tube	

ENCLOSURE 3

LOG OF BOREHOLE BH21-05

DST REF. No.: **02101208**
CLIENT: **Walter Greisseier**
PROJECT: **Proposed Commercial Subdivision and Private Servicing**
LOCATION: **2095 Dilworth Road, Kars, ON**
SURFACE ELEV.: **87.70 metres**

Drilling Data
METHOD: **Casings**
DIAMETER: **203 mm**
DATE: **February 19, 2021**
COORDINATES: m N, m E



MATERIAL DESCRIPTION

ORANGICS - silty sand, compact, dark brown to brown, damp

SANDY SILT- some clay trace gravel, loose to very loose, brown, moist

SANDY SILTY CLAY- stiff to firm, dark brown, moist

SILTY GRAVELLY SAND (TILL) - dense, grey, moist to wet
Presences of cobbles and/or boulders to be expected based on sporadic blow counts

Becoming wet below 4.7 m

Spoon refusal on inferred cobble or boulder at 5.2 m

Very poor quality, grey, LIMESTONE BEDROCK

End of Borehole at approximately 7.2 m in Limestone Bedrock

Water level measure to be at approximatley 1.3 mbgs (~Elev. 86.4 masl) in open borehole.

SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
SS1	11	
SS2	3	
SS3	4	4 29 56 11
SS4	1	
SS5	48	
SS6	50	
SS7	49	
SS8	>99	
RC9		

BOREHOLE (STANDARD) 02100377 2095 DILWORTH ROAD.GPJ DST_MIN.GDT 4/12/21

ENGLOBE

SAMPLE TYPE LEGEND

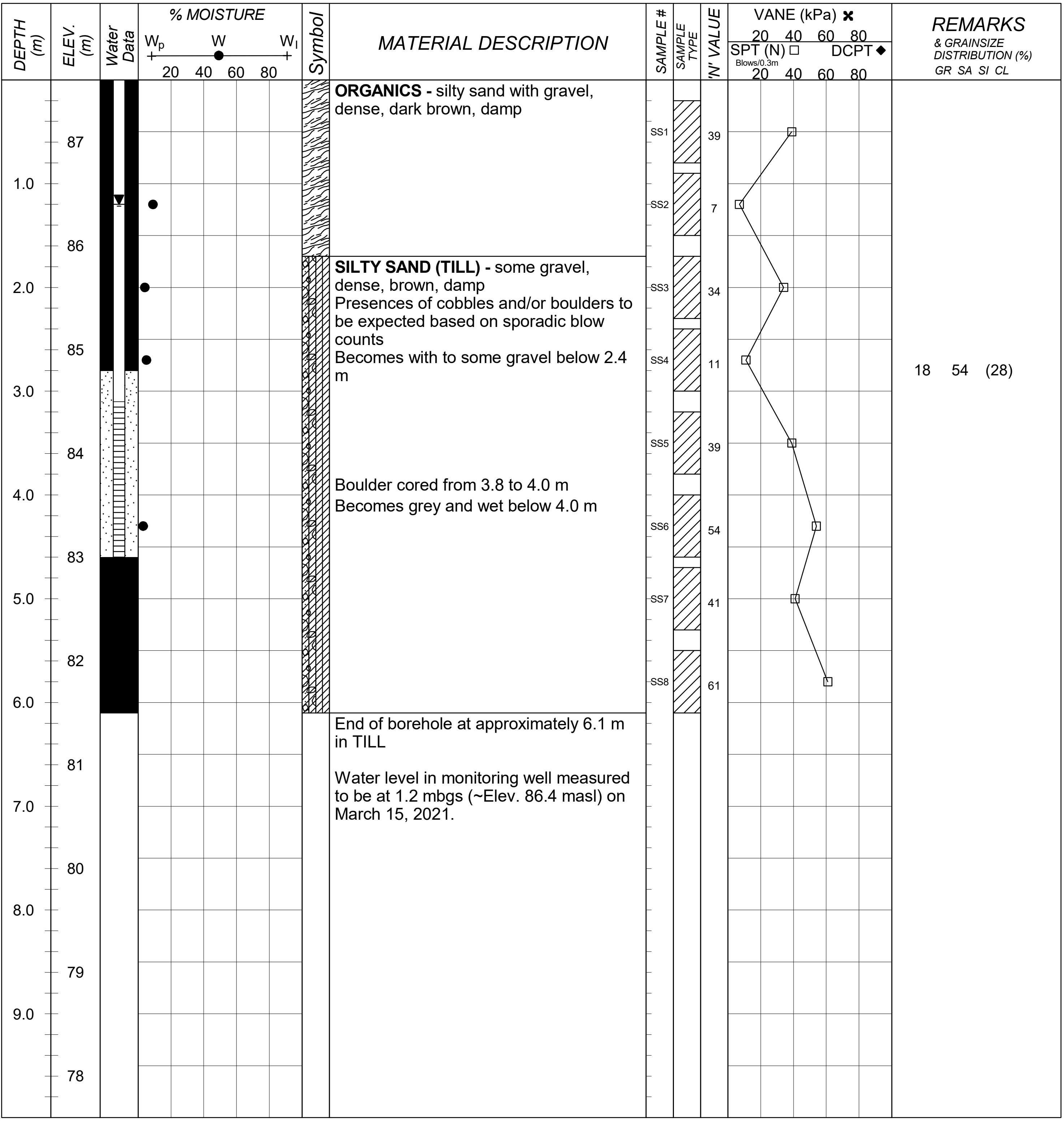
☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	░	Sand
▥	Bulk Sample	◫	70mm Thick Wall Tube		

ENCLOSURE 4

LOG OF BOREHOLE MW21-06

DST REF. No.: **02101208**
CLIENT: **Walter Greisseier**
PROJECT: **Proposed Commercial Subdivision and Private Servicing**
LOCATION: **2095 Dilworth Road, Kars, ON**
SURFACE ELEV.: **87.60 metres**

Drilling Data
METHOD: **Hollow Stem Augers**
DIAMETER: **203 mm**
DATE: **February 17, 2021**
COORDINATES: m N, m E



ORGANICS - silty sand with gravel, dense, dark brown, damp

SILTY SAND (TILL) - some gravel, dense, brown, damp
Presences of cobbles and/or boulders to be expected based on sporadic blow counts
Becomes with to some gravel below 2.4 m

Boulder cored from 3.8 to 4.0 m
Becomes grey and wet below 4.0 m

End of borehole at approximately 6.1 m in TILL

Water level in monitoring well measured to be at 1.2 mbgs (~Elev. 86.4 masl) on March 15, 2021.

SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
SS1	39	
SS2	7	
SS3	34	
SS4	11	18 54 (28)
SS5	39	
SS6	54	
SS7	41	
SS8	61	

SAMPLE TYPE LEGEND

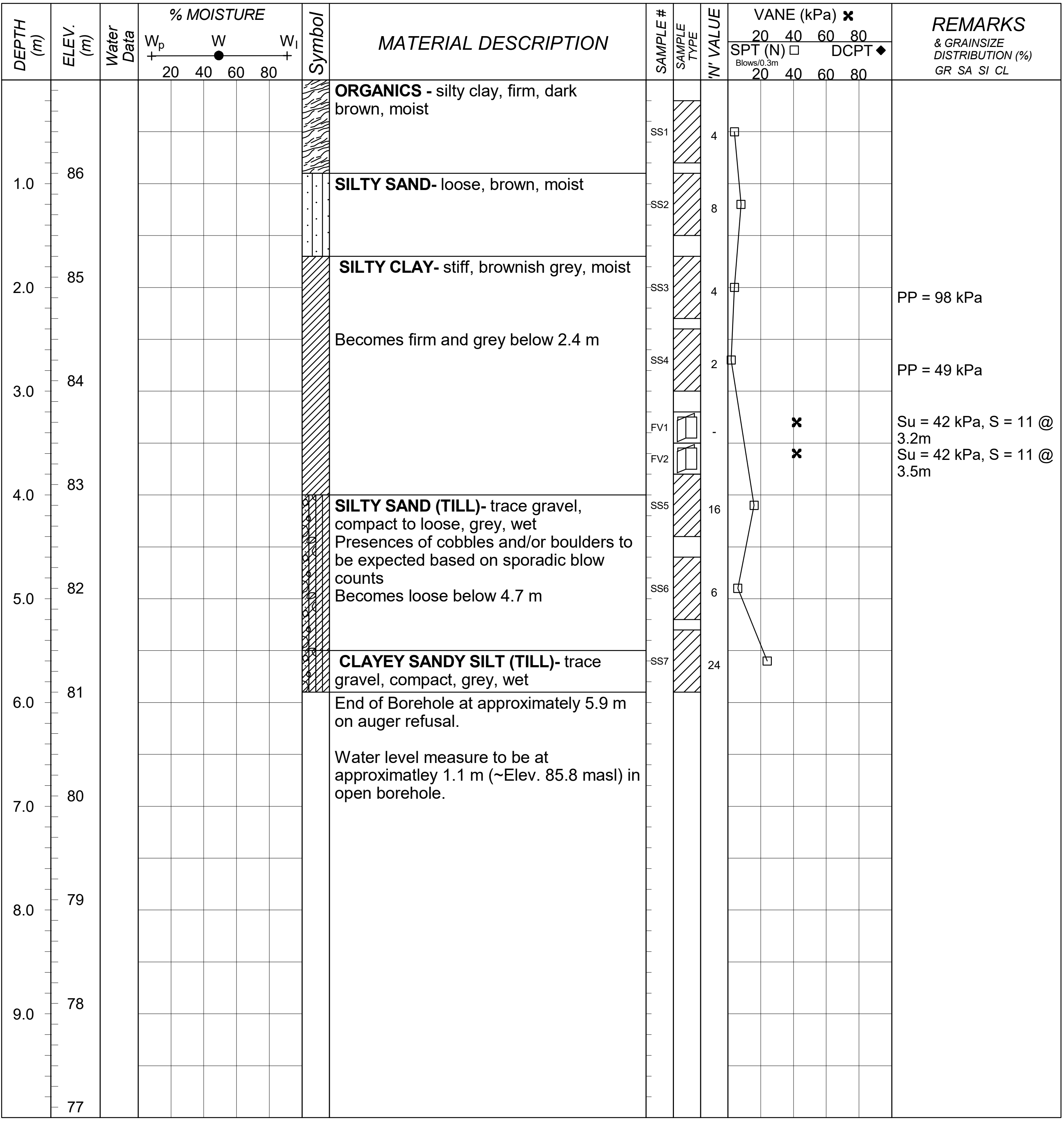
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	▧ Hiller Peat Sampler	░ Sand
┆ Bulk Sample	▯ 70mm Thick Wall Tube	

ENCLOSURE 10

LOG OF BOREHOLE BH21-07

DST REF. No.: **02101208**
CLIENT: **Walter Greisseier**
PROJECT: **Proposed Commercial Subdivision and Private Servicing**
LOCATION: **2095 Dilworth Road, Kars, ON**
SURFACE ELEV.: **86.90 metres**

Drilling Data
METHOD: **Hollow Stem Augers**
DIAMETER: **203 mm**
DATE: **February 18, 2021**
COORDINATES: m N, m E



ORGANICS - silty clay, firm, dark brown, moist

SILTY SAND- loose, brown, moist

SILTY CLAY- stiff, brownish grey, moist

Becomes firm and grey below 2.4 m

SILTY SAND (TILL)- trace gravel, compact to loose, grey, wet
Presences of cobbles and/or boulders to be expected based on sporadic blow counts
Becomes loose below 4.7 m

CLAYEY SANDY SILT (TILL)- trace gravel, compact, grey, wet

End of Borehole at approximately 5.9 m on auger refusal.

Water level measure to be at approximatley 1.1 m (~Elev. 85.8 masl) in open borehole.

SAMPLE #	'N' VALUE	REMARKS
SS1	4	
SS2	8	
SS3	4	PP = 98 kPa
SS4	2	PP = 49 kPa
FV1	-	Su = 42 kPa, S = 11 @ 3.2m
FV2		Su = 42 kPa, S = 11 @ 3.5m
SS5	16	
SS6	6	
SS7	24	

ENGLOBE

SAMPLE TYPE LEGEND

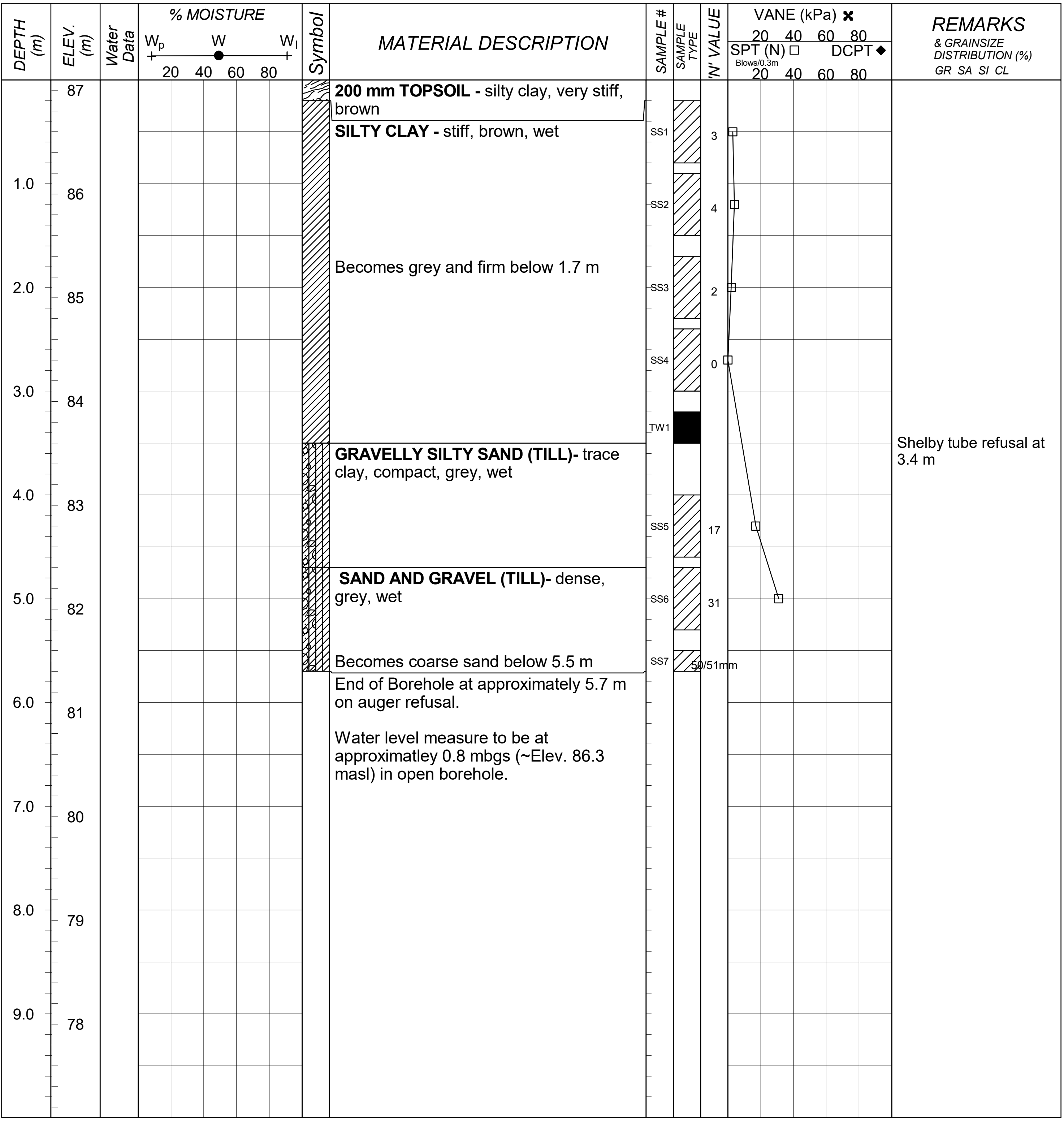
▤ Auger Sample
▨ Split Spoon Sample
▥ Bulk Sample
▦ Rock Core
▧ Hiller Peat Sampler
◫ 70mm Thick Wall Tube
■ Bentonite
░ Sand

ENCLOSURE 5

LOG OF BOREHOLE BH21-08

DST REF. No.: **02101208**
CLIENT: **Walter Greisseier**
PROJECT: **Proposed Commercial Subdivision and Private Servicing**
LOCATION: **2095 Dilworth Road, Kars, ON**
SURFACE ELEV.: **87.10 metres**

Drilling Data
METHOD: **Hollow Stem Augers**
DIAMETER: **203 mm**
DATE: **February 18, 2021**
COORDINATES: m N, m E



MATERIAL DESCRIPTION	SAMPLE #	'N' VALUE	REMARKS
200 mm TOPSOIL - silty clay, very stiff, brown			
SILTY CLAY - stiff, brown, wet	SS1	3	
	SS2	4	
Becomes grey and firm below 1.7 m	SS3	2	
	SS4	0	
	TW1		
GRAVELLY SILTY SAND (TILL)- trace clay, compact, grey, wet			Shelby tube refusal at 3.4 m
	SS5	17	
SAND AND GRAVEL (TILL)- dense, grey, wet	SS6	31	
Becomes coarse sand below 5.5 m	SS7	50/51mm	
End of Borehole at approximately 5.7 m on auger refusal. Water level measure to be at approximatley 0.8 mbgs (~Elev. 86.3 masl) in open borehole.			

ENGLOBE

SAMPLE TYPE LEGEND

- ▤ Auger Sample
- ▨ Split Spoon Sample
- ▥ Bulk Sample
- ▩ Rock Core
- ▧ Hiller Peat Sampler
- ◫ 70mm Thick Wall Tube
- ■ Bentonite
- ░ Sand

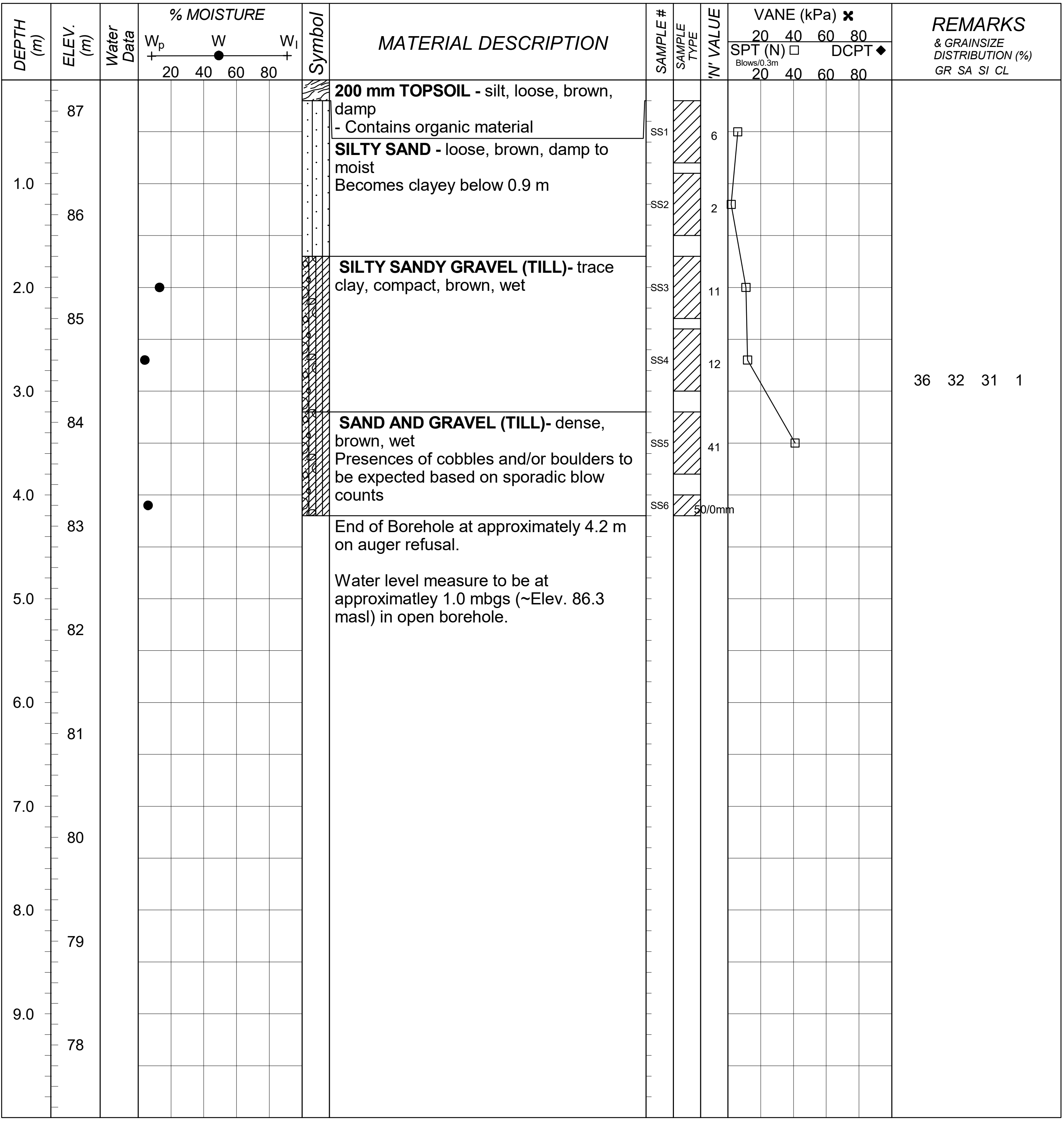
ENCLOSURE 6

BOREHOLE (STANDARD) 02100377 2095 DILWORTH ROAD.GPJ DST_MIN.GDT 4/12/21

LOG OF BOREHOLE BH21-09

DST REF. No.: **02101208**
CLIENT: **Walter Greisseier**
PROJECT: **Proposed Commercial Subdivision and Private Servicing**
LOCATION: **2095 Dilworth Road, Kars, ON**
SURFACE ELEV.: **87.30 metres**

Drilling Data
METHOD: **Hollow Stem Augers**
DIAMETER: **203 mm**
DATE: **February 18, 2021**
COORDINATES: m N, m E



MATERIAL DESCRIPTION
200 mm TOPSOIL - silt, loose, brown, damp - Contains organic material
SILTY SAND - loose, brown, damp to moist Becomes clayey below 0.9 m
SILTY SANDY GRAVEL (TILL)- trace clay, compact, brown, wet
SAND AND GRAVEL (TILL)- dense, brown, wet Presences of cobbles and/or boulders to be expected based on sporadic blow counts

End of Borehole at approximately 4.2 m on auger refusal.

Water level measure to be at approximatley 1.0 mbgs (~Elev. 86.3 masl) in open borehole.

SAMPLE #	'N' VALUE	REMARKS & GRAINSIZE DISTRIBUTION (%) GR SA SI CL
SS1	6	
SS2	2	
SS3	11	
SS4	12	36 32 31 1
SS5	41	
SS6	50/0mm	

SAMPLE TYPE LEGEND

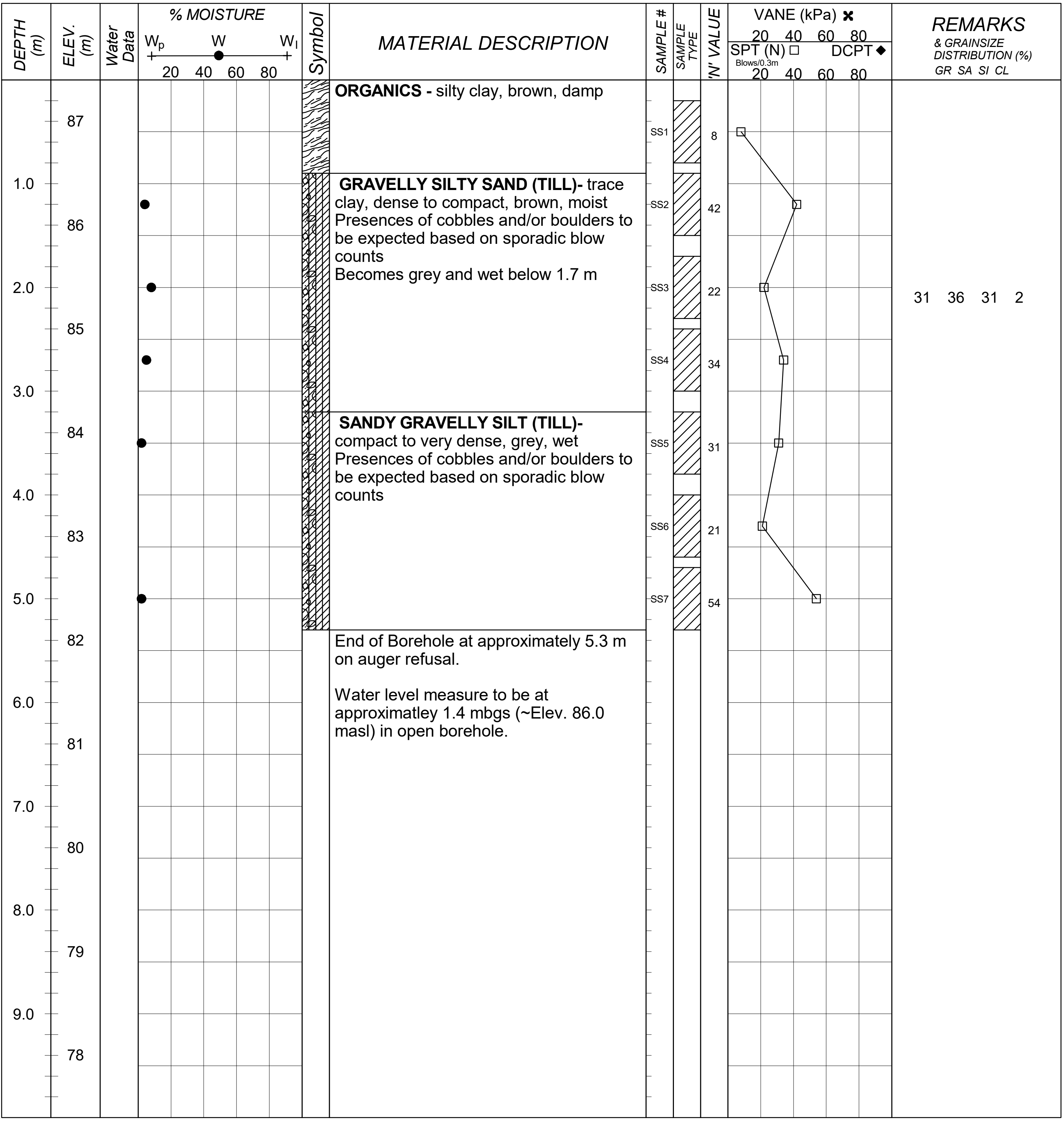
☰ Auger Sample	▩ Rock Core	■ Bentonite
▨ Split Spoon Sample	▧ Hiller Peat Sampler	░ Sand
▥ Bulk Sample	◫ 70mm Thick Wall Tube	

ENCLOSURE 7

LOG OF BOREHOLE BH21-10

DST REF. No.: **02101208**
CLIENT: **Walter Greisseier**
PROJECT: **Proposed Commercial Subdivision and Private Servicing**
LOCATION: **2095 Dilworth Road, Kars, ON**
SURFACE ELEV.: **87.40 metres**

Drilling Data
METHOD: **Hollow Stem Augers**
DIAMETER: **203 mm**
DATE: **February 17, 2021**
COORDINATES: m N, m E



ORGANICS - silty clay, brown, damp

GRAVELLY SILTY SAND (TILL)- trace clay, dense to compact, brown, moist
Presences of cobbles and/or boulders to be expected based on sporadic blow counts
Becomes grey and wet below 1.7 m

SANDY GRAVELLY SILT (TILL)- compact to very dense, grey, wet
Presences of cobbles and/or boulders to be expected based on sporadic blow counts

End of Borehole at approximately 5.3 m on auger refusal.

Water level measure to be at approximatley 1.4 mbgs (~Elev. 86.0 masl) in open borehole.

SAMPLE #	SAMPLE TYPE	'N' VALUE	GR SA SI CL
SS1	▨	8	
SS2	▨	42	
SS3	▨	22	31 36 31 2
SS4	▨	34	
SS5	▨	31	
SS6	▨	21	
SS7	▨	54	

SAMPLE TYPE LEGEND

☰	Auger Sample	▩	Rock Core	■	Bentonite
▨	Split Spoon Sample	▧	Hiller Peat Sampler	⬚	Sand
▥	Bulk Sample	◫	70mm Thick Wall Tube		

ENCLOSURE 8

PAGE 1 OF 1

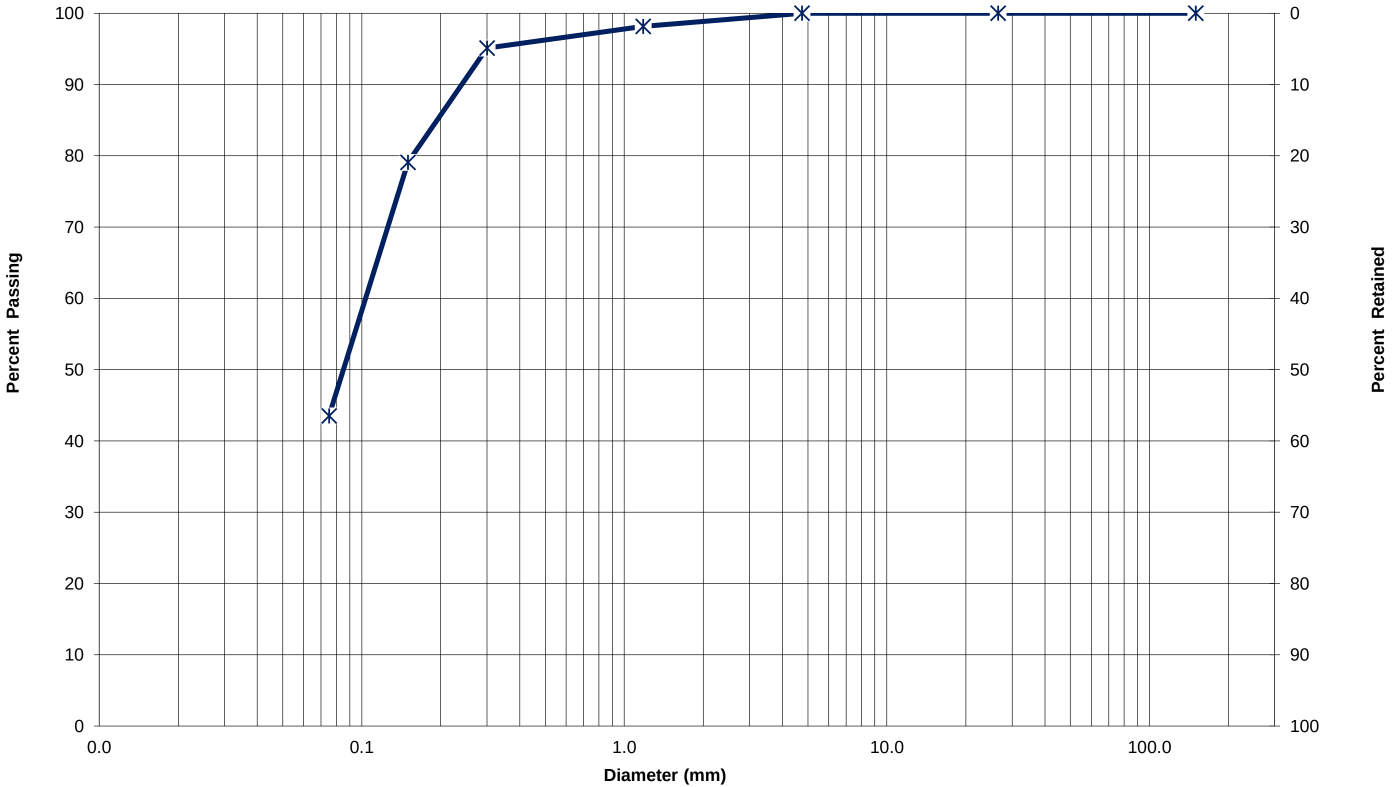
BOREHOLE (STANDARD) 02100377 2095 DILWORTH ROAD.GPJ DST_MIN.GDT 4/12/21

Appendix D

Geotechnical Laboratory Results
Rockcore Photograph

GRAIN SIZE ANALYSIS

Ref. No.:	2101208.000	**Date Sampled:**	16-Feb-21
Project:	Proposed Commercial Subdivision	**Sampled By:**	Cameron Fischl
Client:	2095 Dilworth Road	**Material Source:**	MW21-01, SS3A
Project Location:	Dilworth Development Inc.	**Sampling Location:**	1.5m-2.1m
Sample #:	21-042	**Material Description:**	SILT AND SAND

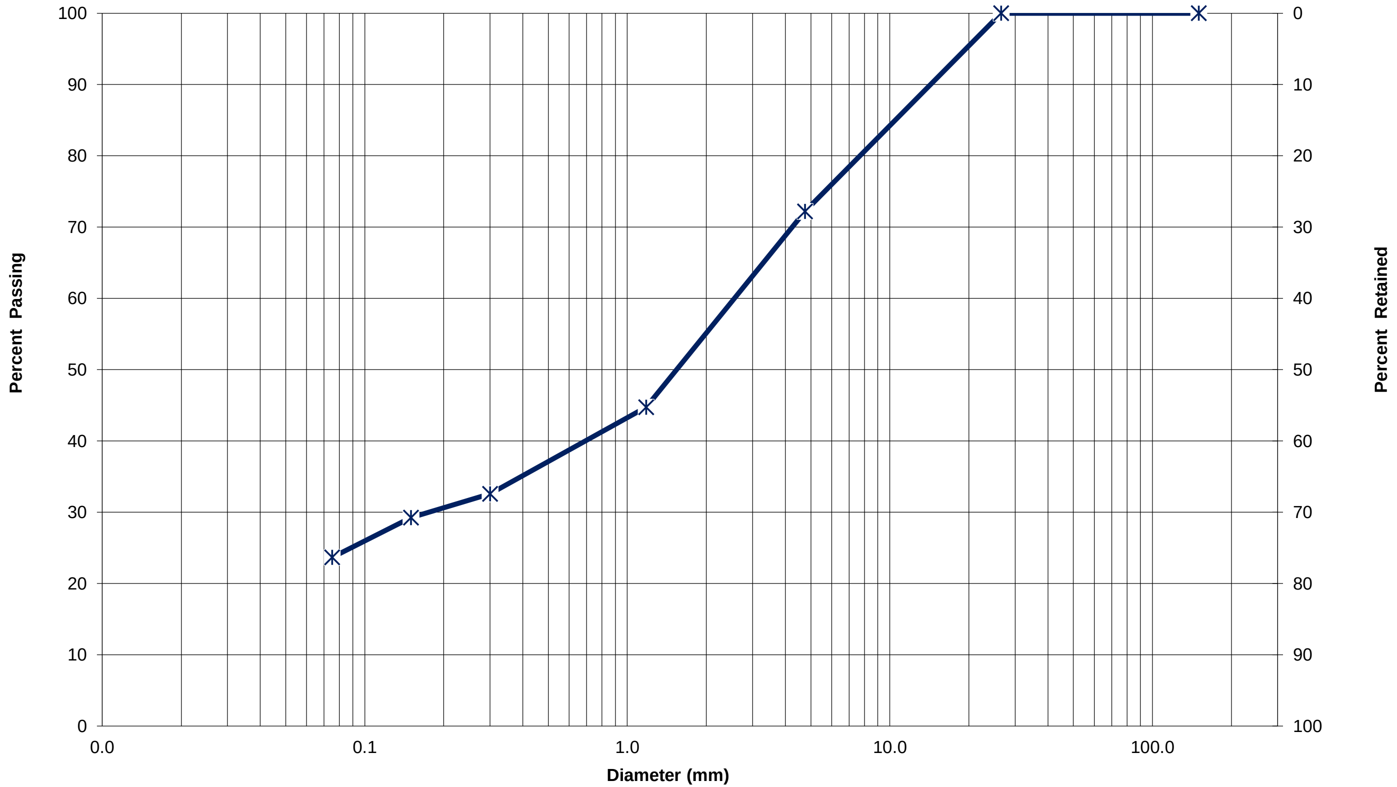


Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

SUMMARY			
Soil Description	**Gravel (%)**	**Sand (%)**	**Clay and Silt (%)**
SILT AND SAND	0	56	44

GRAIN SIZE ANALYSIS

Ref. No.:	2101208.000	**Date Sampled:**	16-Feb-21
Project:	Proposed Commercial Subdivisions	**Sampled By:**	Cameron Fischl
Client:	Dilworth Development Inc.	**Material Source:**	MW21-01, SS7
Project Location:	2095 Dilworth Road	**Sampling Location:**	4.6m-5.2m
Sample #:	21-042	**Material Description:**	SILTY SAND, some gravel (TILL)

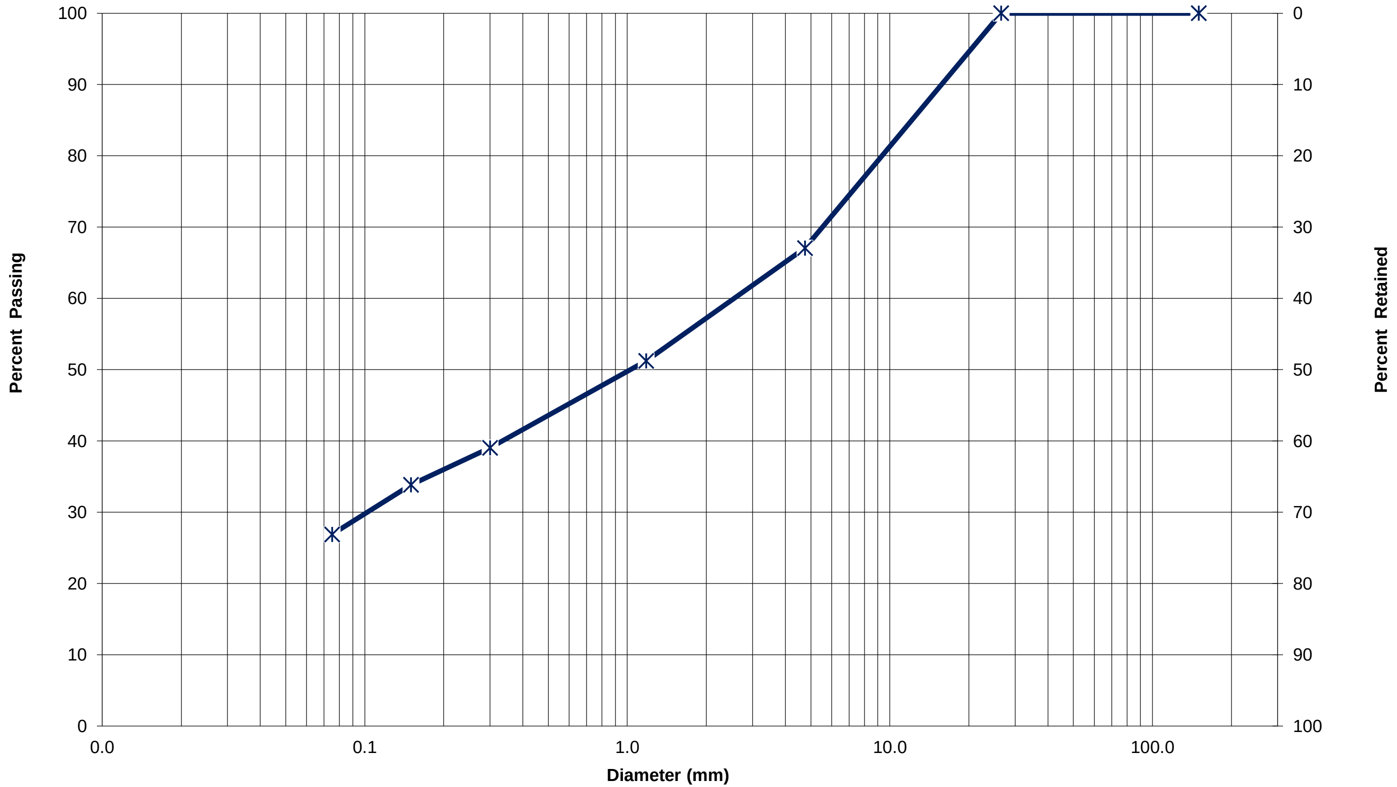


Clay & Silt	**Sand**			**Gravel**	
	Fine	**Medium**	**Coarse**	**Fine**	**Coarse**
Particle-Size Limits as per USCS (ASTM D-2487)					

SUMMARY			
Soil Description	**Gravel (%)**	**Sand (%)**	**Clay and Silt (%)**
SILTY SAND, some gravel (TILL)	28	49	24

GRAIN SIZE ANALYSIS

Ref. No.:	2101208.000	**Date Sampled:**	16-Feb-21
Project:	Proposed Commercial Subdivision	**Sampled By:**	Cameron Fischl
Client:	Dilworth Development Inc.	**Material Source:**	MW21-03, SS5
Project Location:	2095 Dilworth Road	**Sampling Location:**	3.2m-3.8m
Sample #:	21-042	**Material Description:**	GRAVELLY SILTY SAND (TILL)

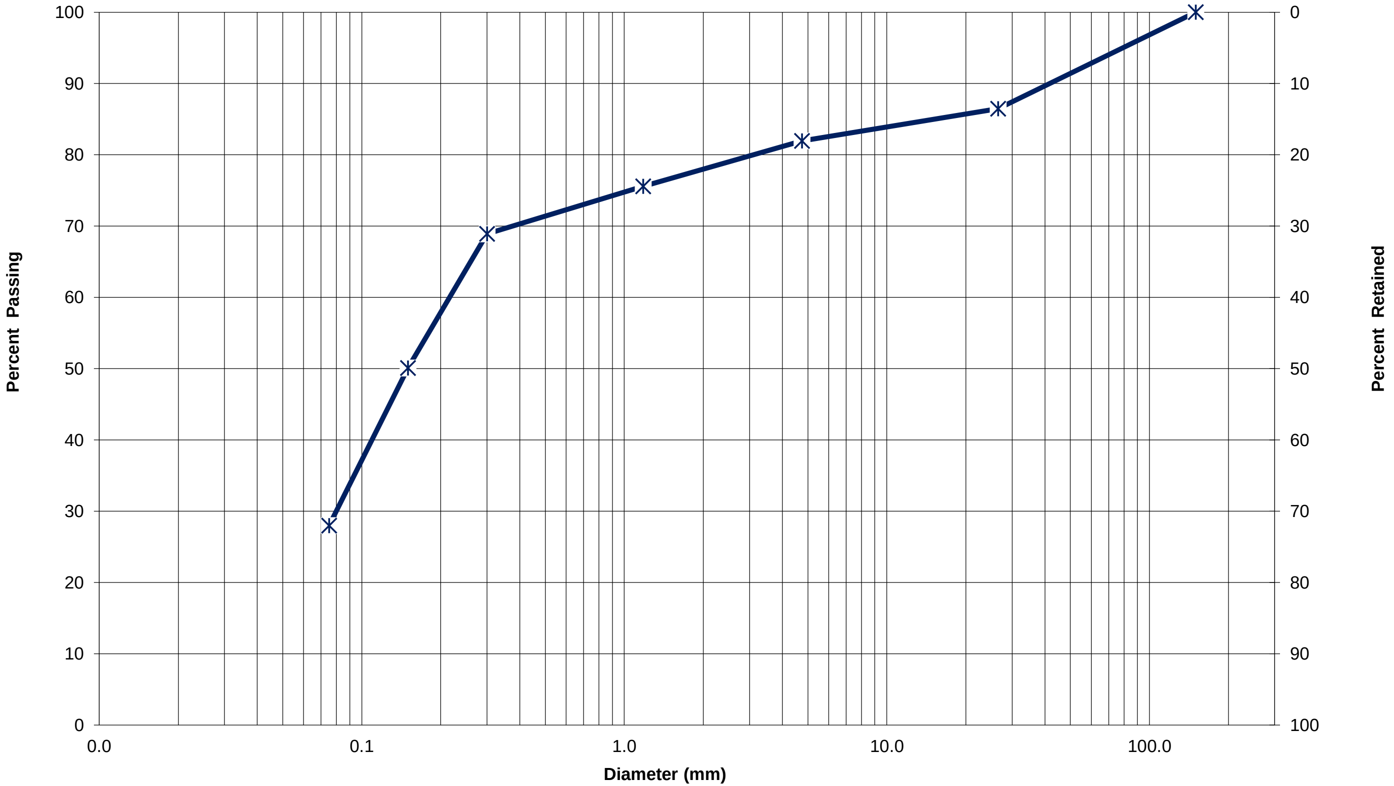


Clay & Silt	**Sand**			**Gravel**	
	Fine	**Medium**	**Coarse**	**Fine**	**Coarse**
Particle-Size Limits as per USCS (ASTM D-2487)					

SUMMARY			
Soil Description	**Gravel (%)**	**Sand (%)**	**Clay and Silt (%)**
GRAVELLY SILTY SAND (TILL)	33	40	27

GRAIN SIZE ANALYSIS

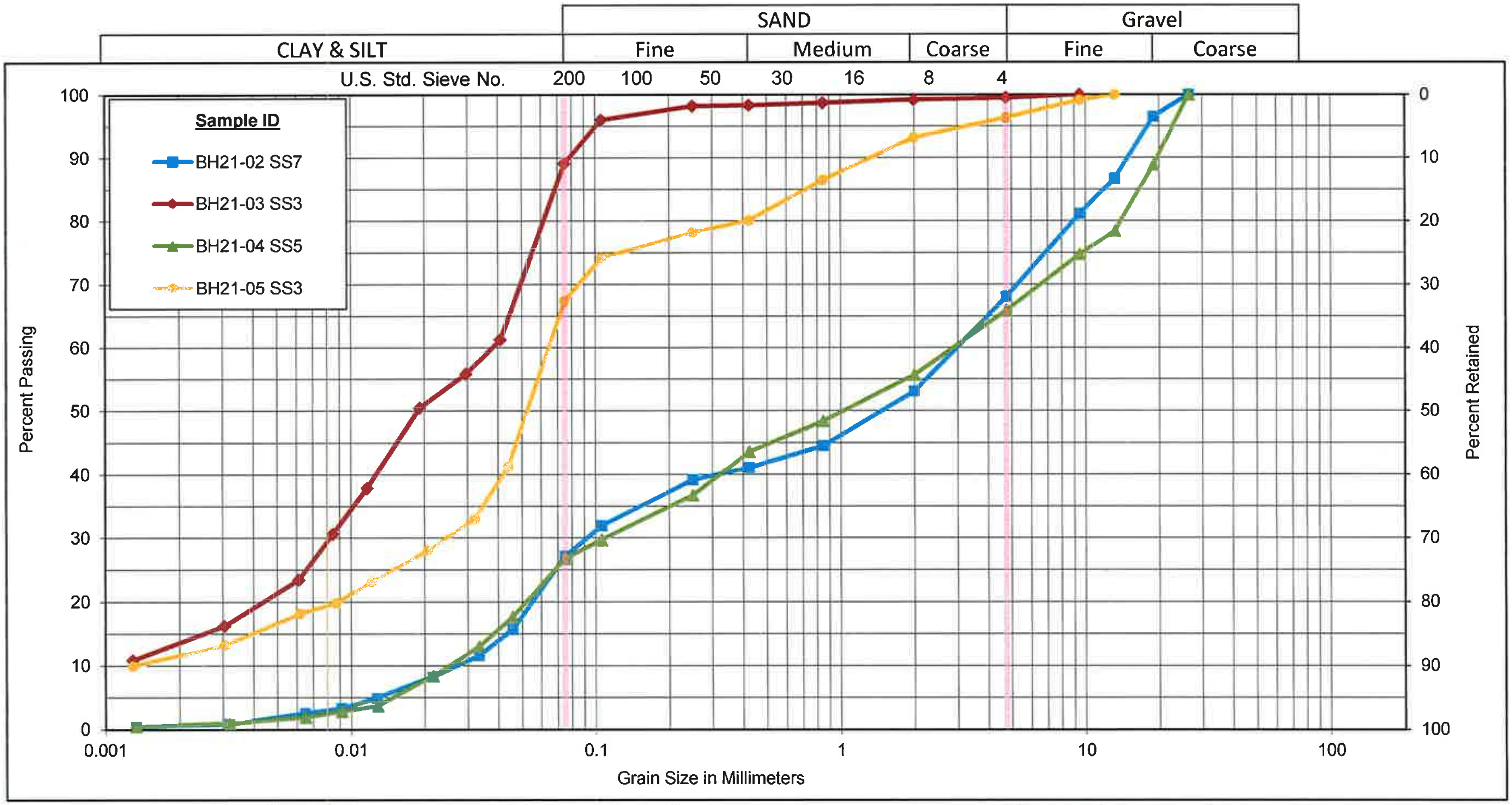
Ref. No.:	2101208.000	**Date Sampled:**	17-Feb-21
Project:	Proposed Commercial Subdivision	**Sampled By:**	Cameron Fischl
Client:	Dilworth Development Inc.	**Material Source:**	MW21-06, SS4
Project Location:	2095 Dilworth Road	**Sampling Location:**	2.4m-3.0m
Sample #:	21-042	**Material Description:**	SILTY SAND some gravel (TILL)



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Particle-Size Limits as per USCS (ASTM D-2487)					

SUMMARY			
Soil Description	**Gravel (%)**	**Sand (%)**	**Clay and Silt (%)**
SILTY SAND some gravel (TILL)	18	54	28

Unified Soil Classification System



Sample ID	Depth	% Gravel	% Sand	% Silt	% Clay
BH21-02 SS7	5.5-6.1 m	31.9	41.0	26.1	1.0
BH21-03 SS3	1.7-2.3 m	0.5	10.4	76.1	13.0
BH21-04 SS5	3.2-3.8 m	34.1	39.2	25.7	1.0
BH21-05 SS3	1.7-2.3 m	3.7	29.0	56.3	11.0

Stantec

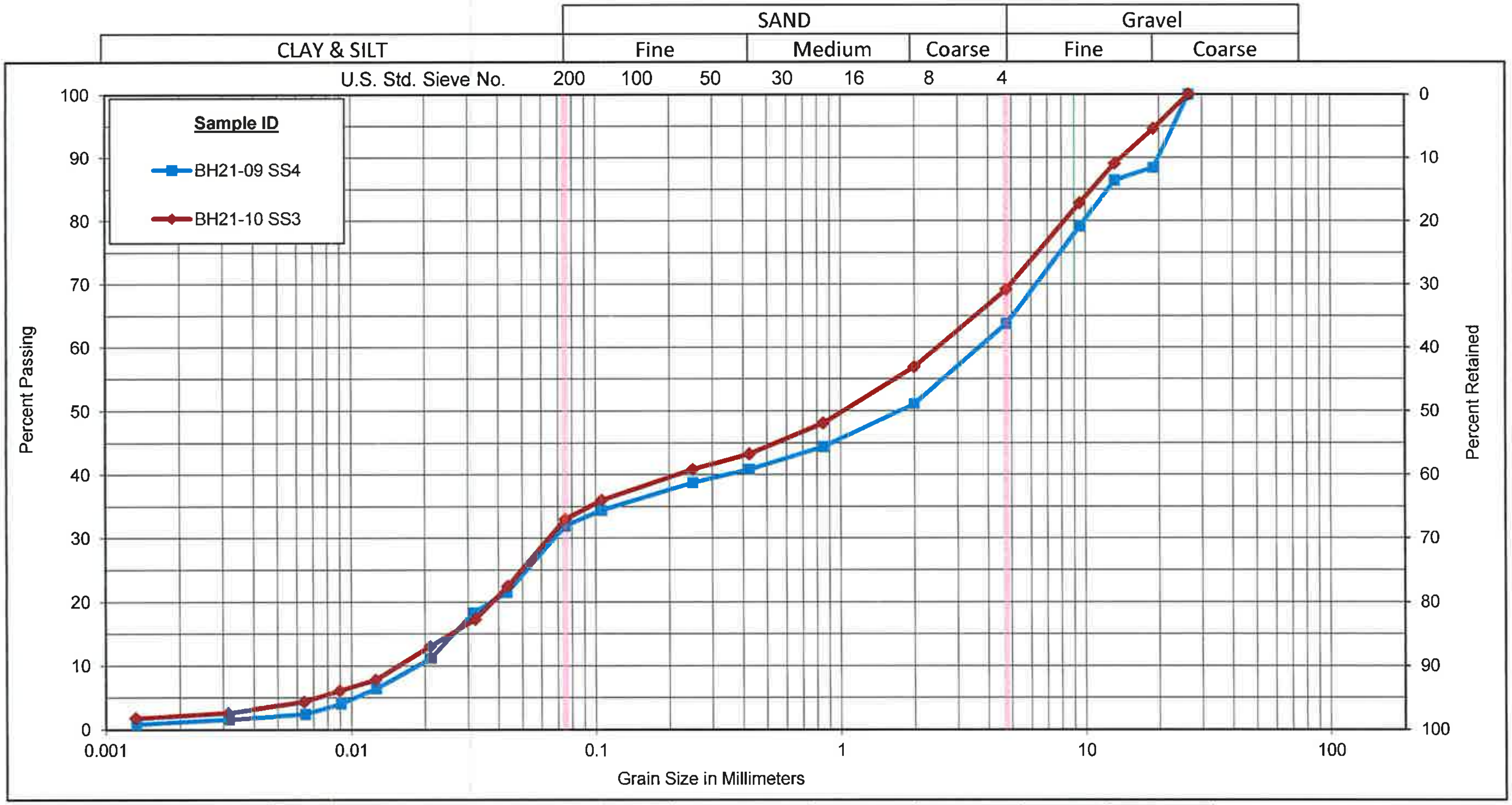
GRAIN SIZE DISTRIBUTION

DST Consulting Engineers Inc. File # 2101208
2095 Dilworth Road

Figure No.

Project No. 122411080

Unified Soil Classification System



Sample ID	Depth	% Gravel	% Sand	% Silt	% Clay
BH21-09 SS4	2.5-3.1 m	36.2	31.9	30.9	1.0
BH21-10 SS3	1.7-2.3 m	31.0	36.0	31.0	2.0

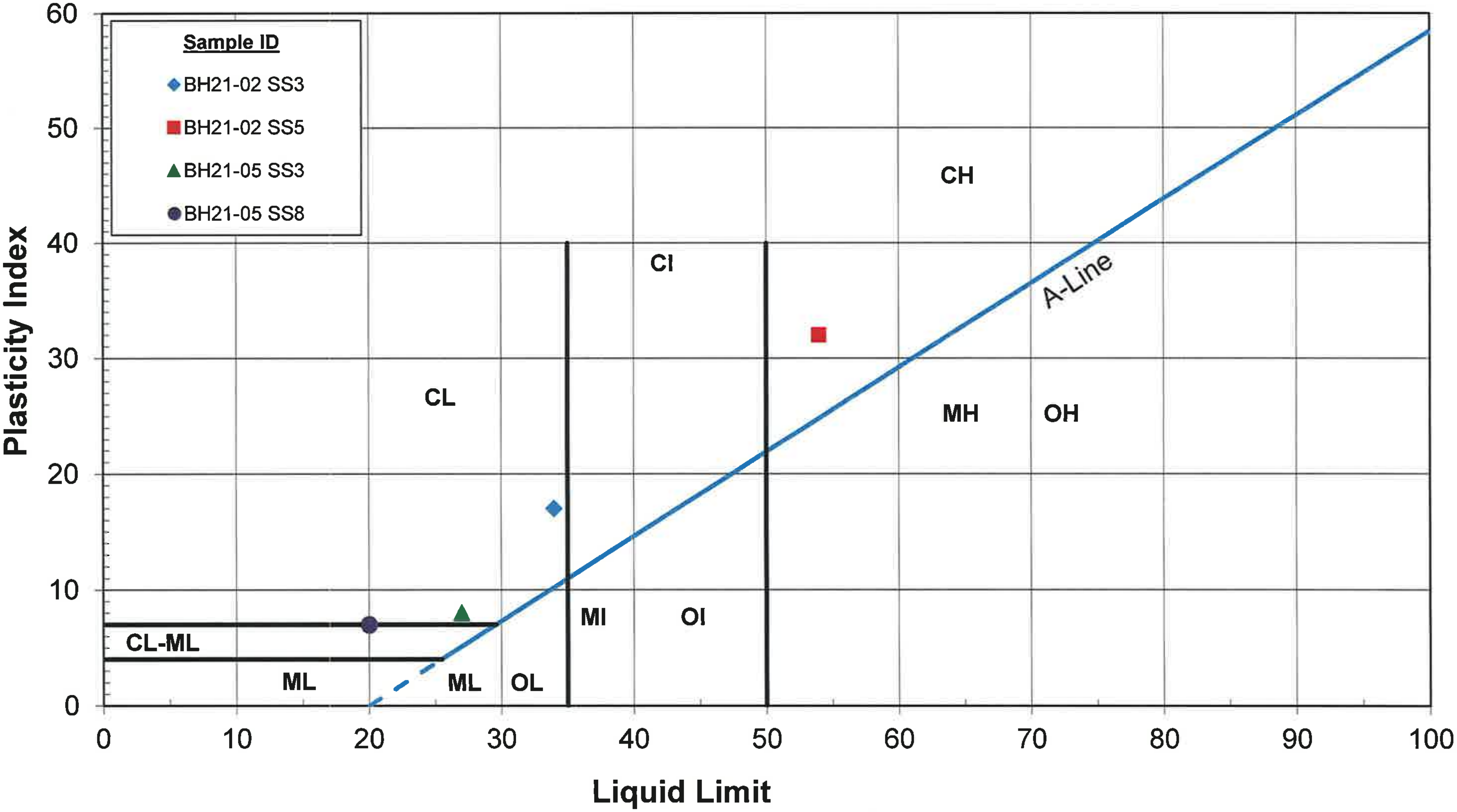
Stantec

GRAIN SIZE DISTRIBUTION

DST Consulting Engineers Inc. File # 2101208
2095 Dilworth Road

Figure No.

Project No. 122411080



Stantec

DST Consulting Engineers Inc. File # 2101208

2095 Dilworth Road.

PLASTICITY CHART

Figure No.

Project No. 122411080

Rockcore Photograph
2095 Dilworth Road, Kars, ON

Project No.: 0210208.000

Rock Core Photo No.: 1	Borehole: BH 21-05	Depth: 5.8 to 7.2 m

Appendix E

Chemical Laboratory Test Results

ENGLOBE

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Certificate of Analysis

DST Consulting Engineers Inc. (Ottawa)

203-2150 Thurston Dr.
Ottawa, ON K1G 5T9
Attn: Shanti Ratmono

Client PO:
Project: 2101208.00
Custody: 129454

Report Date: 8-Mar-2021
Order Date: 5-Mar-2021

Order #: 2110588

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

Paracel ID	Client ID
2110588-01	BH21-5, SS2
2110588-02	BH21-6, SS2

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	8-Mar-21	8-Mar-21
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	8-Mar-21	8-Mar-21
Resistivity	EPA 120.1 - probe, water extraction	8-Mar-21	8-Mar-21
Solids, %	Gravimetric, calculation	8-Mar-21	8-Mar-21

Certificate of Analysis

Client: **DST Consulting Engineers Inc. (Ottawa)**

Client PO:

Report Date: 08-Mar-2021

Order Date: 5-Mar-2021

Project Description: 2101208.00

	Client ID:	BH21-5, SS2	BH21-6, SS2	-	-
	Sample Date:	01-Mar-21 09:00	01-Mar-21 09:00	-	-
	Sample ID:	2110588-01	2110588-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	89.7	78.1	-	-
General Inorganics					
pH	0.05 pH Units	7.54	6.91	-	-
Resistivity	0.10 Ohm.m	36.2	63.5	-	-
Anions					
Chloride	5 ug/g dry	16	45	-	-
Sulphate	5 ug/g dry	122	15	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	452	5	ug/g dry	451			0.4	20	
Sulphate	187	5	ug/g dry	223			17.7	20	
General Inorganics									
pH	7.38	0.05	pH Units	7.35			0.4	2.3	
Physical Characteristics									
% Solids	88.8	0.1	% by Wt.	89.7			1.0	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	542	5	ug/g	451	91.5	82-118			
Sulphate	311	5	ug/g	223	88.6	80-120			

Qualifier Notes:

Login Qualifiers :

Sample not received in Paracel verified container / media

Applies to samples: BH21-5, SS2, BH21-6, SS2

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
NC: Not Calculated

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

PARACEL
LABORATORIES

Paracel ID: 2110588

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bs.com

Paracel Order Number
(Lab Use Only)
2110588

Chain Of Custody
(Lab Use Only)
№ 129454

Client Name: DST Group

Contact Name: Shanti Ratmono

Address: 2150 Thurston Drive
Ottawa, ON

Telephone: 613-402-0393

Project Ref: 2101208.00

Quote #:

PO #:

E-mail: sratmono@dstgroup.com
cfischl@dstgroup.com

Page 1 of 1

Turnaround Time

☐ 1 day ☐ 3 day
☐ 2 day ☒ Regular

Date Required:

Regulation 153/04

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

Other Regulation

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

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

					Sample Taken		Required Analysis												
	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Date	Time	PHCs F1-F4+BTEX	VOCs	PAHs	Metals by ICP	Hg	CrVI	B (HWS)	pH	Chloride	Sulphate	Resistivity	Redox Potential	Sulphide
1	BH21-5, SS2	S		2	2021/03/01	—								X	X	X	X	X	X
2	BH21-6, SS2	S		2	↓	—								X	X	X	X	X	X
3																			
4																			
5																			
6																			
7																			
8																			
9																			
10																			

Comments:

Method of Delivery: Drop Box

Relinquished By (Sign): CamFischl

Relinquished By (Print): Cam Fischl

Date/Time: 2021/03/05 1400

Received By Driver/Depot:

Date/Time:

Temperature: °C

Received at Lab: Sineepom Ndkmdi

Date/Time: Mar 05, 2021 02,31

Temperature: 16,4 °C

Verified By:

Date/Time: Mar 5 2021 1436

pH Verified: ☐ By:

Chain of Custody (Env.) xlsx

Revision 3.0

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Subcontracted Analysis

DST Consulting Engineers Inc. (Ottawa)
203-2150 Thurston Dr.
Ottawa, ON K1G 5T9
Attn: Shanti Ratmono

Tel: (343) 549-6678
Fax: (613) 748-1356

Paracel Report No **2110588**
Client Project(s): **2101208.00**
Client PO:
Reference: **Standing Offer**
CoC Number: **129454**

Order Date: 05-Mar-21
Report Date: 8-Mar-21

Sample(s) from this project were subcontracted for the listed parameters. A copy of the subcontractor's report is attached

Paracel ID	**Client ID**	**Analysis**
2110588-01	BH21-5, SS2	Redox potential, soil Sulphide, solid
2110588-02	BH21-6, SS2	Redox potential, soil Sulphide, solid

CERTIFICATE OF ANALYSIS

Client:	Dale Robertson	Work Order Number:	424718
Company:	Paracel Laboratories Ltd.- Ottawa	PO #:	
Address:	300-2319 St. Laurent Blvd. Ottawa, ON, K1G 4J8	Regulation:	O.Reg 153 Table 1 Soil Agricultural/Other
		Project #:	2110588
Phone/Fax:	(613) 731-9577 / (613) 731-9064	DWS #:	
Email:	drobertson@paracellabs.com	Sampled By:	
Date Order Received:	3/9/2021	Analysis Started:	3/16/2021
Arrival Temperature:	6.6 °C	Analysis Completed:	3/16/2021

WORK ORDER SUMMARY

ANALYSES WERE PERFORMED ON THE FOLLOWING SAMPLES. THE RESULTS RELATE ONLY TO THE ITEMS TESTED.

Sample Description	Lab ID	Matrix	Type	Comments	Date Collected	Time Collected
BH21-5, SS2	1623623	Soil	None		3/1/2021	
BH21-6, SS2	1623624	Soil	None		3/1/2021	

METHODS AND INSTRUMENTATION

THE FOLLOWING METHODS WERE USED FOR YOUR SAMPLE(S):

Method	Lab	Description	Reference
RedOx - Soil (T06)	Mississauga	Determination of RedOx Potential of Soil	Modified from APHA-2580B

REPORT COMMENTS

Non-Testmark containers submitted, samples received past hold time for redox potential, proceed with analysis as per client notes 03/09/2021 YH

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd.- Ottawa

Work Order Number: 424718

This report has been approved by:

Marc Creighton
Laboratory Director

CERTIFICATE OF ANALYSIS

Paracel Laboratories Ltd.- Ottawa

Work Order Number: 424718

WORK ORDER RESULTS

Sample Description	BH21 - 5, SS2		BH21 - 6, SS2			
Sample Date	3/1/2021 12:00 AM		3/1/2021 12:00 AM			
Lab ID	1623623		1623624			
General Chemistry	Result	MDL	Result	MDL	Units	Criteria: O.Reg 153 Table 1 Soil Agricultural/Other
RedOx (vs. S.H.E.)	330	N/A	286 [289]	N/A	mV	~

LEGEND

Dates: Dates are formatted as mm/dd/year throughout this report.

[rr]: After a parameter name indicates a re-run of that parameter. If multiple re-runs exist they are suffixed by a number. Sample may not have been handled according to the recommended temperature, hold time and head space requirements of the method after the initial analysis.

MDL: Method detection limit or minimum reporting limit.

[ ]: Results for laboratory replicates are shown in square brackets immediately below the associated sample result for ease of comparison.

~: In a criteria column indicates the criteria is not applicable for the parameter row.

Quality Control: All associated Quality Control data is available on request.

Field Data: Reports containing Field Parameters represent data that has been collected and provided by the client. Testmark is not responsible for the validity of this data which may be used in subsequent calculations.

Sample Condition Deviations: A noted sample condition deviation may affect the validity of the result. Results apply to the sample(s) as received.

SGS Canada Inc.
P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - KOL 2HO
Phone: 705-652-2000 FAX: 705-652-6365

19-March-2021

Paracel Laboratories
Attn : Dale Robertson

300-2319 St.Laurent Blvd.
Ottawa, ON
K1G 4K6, Canada

Phone: 613-731-9577
Fax:613-731-9064

Date Rec. : 09 March 2021
LR Report: **CA14245-MAR21**
Reference: Project#: 2110588

Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Sample ID	Sample Date & Time	Sulphide (Na2CO3) %
1: Analysis Start Date		19-Mar-21
2: Analysis Start Time		12:41
3: Analysis Completed Date		19-Mar-21
4: Analysis Completed Time		13:34
5: QC - Blank		< 0.04
6: QC - STD % Recovery		106%
7: QC - DUP % RPD		NV
8: RL		0.02
9: BH21-5, SS2	01-Mar-21	< 0.04
10: BH21-6, SS2	02-Mar-21	< 0.04

RL - SGS Reporting Limit
NV - No Value

Kimberley Didsbury
Project Specialist,
Environment, Health & Safety

OnLine LIMS

0002436797

Appendix F

2015 National Building Code Seismic Hazard Calculations

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 45.096N 75.648W **User File Reference:** 2095 Dilworth Road 2021-03-18 14:47 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.464	0.249	0.145	0.041
Sa (0.1)	0.541	0.302	0.183	0.057
Sa (0.2)	0.449	0.256	0.158	0.052
Sa (0.3)	0.339	0.195	0.122	0.042
Sa (0.5)	0.238	0.138	0.087	0.030
Sa (1.0)	0.117	0.069	0.044	0.015
Sa (2.0)	0.055	0.032	0.020	0.006
Sa (5.0)	0.015	0.008	0.005	0.001
Sa (10.0)	0.005	0.003	0.002	0.001
PGA (g)	0.288	0.163	0.100	0.030
PGV (m/s)	0.197	0.110	0.067	0.020

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Seismic Site Classification (Based on BH21-05)

Site Classification for Seismic Site Response Calculations (Commentary J)

Depth		Soil	Layer Thickness t	Corrected N-Value N_{60}	t/N_{60}
From	To				
(mbgs)	(mbgs)		(m)	( )	
1.5	2.4	Sandy Silt	0.9	3	0.3000
2.4	3.2	Sandy Silty Clay	0.8	1	0.8000
3.2	4.0	Glacial Till	0.8	36	0.0222
4.0	4.7		0.7	37	0.0189
4.7	5.5		0.8	36	0.0220
5.5	5.8		0.3	42	0.0071
5.8	30.0	Rock	24.2	100	0.2420
		TOTAL =	**28.5**	Sum t/N_{60} =	**1.4123**

NOTES:

(1) The founding depth is set as 1.5 mbgs based on anticipated depth of underside on pile cap in BH21-05.

(2) The N-Value bedrock conservatively taken as 100 (uncorrected).

The average standard penetration resistance is calculated using the following formula:
(as per OBC 2006 Table 4.1.8.4.A.):

$$\text{Avg}(N_{60)} = \frac{\text{Total Thickness of all Layers}}{\sum \frac{\text{Layer Thickness } (t)}{\text{Layer Corrected N-Value } (N_{60})}}$$

$$\text{Avg(N60)} = \frac{28.5}{1.4123}$$

Avg(N60) = 20.2

Average Standard Penetration Resistance for the Site is between 15 and 50.
∴ Seismic Site Class = 'D' based on average standard penetration resistance.

Appendix G

Grade Raise Memorandum

ENGLOBE

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July 18, 2024

Dilworth Development Inc.
92 Bentley Avenue
Ottawa, Ontario
K3E 6T9

Attention: **Mr. Dennis Collautti**

Subject: **Proposed Commercial Development 1.0 m Grade Raise**
2095 Dilworth Road, Kars, ON
Englobe Reference: 02101208.001

Englobe Corp. (Englobe) was retained by Dilworth Development Inc. (Client) to complete a preliminary geotechnical investigation for a proposed commercial development (Project) located at municipal address 2095 Dilworth Road in Kars, Ontario (Site). This memo should be read in conjunction with the original geotechnical investigation report entitled, “Preliminary Geotechnical Investigation Report, Proposed Commercial Development, 2095 Dilworth Road” (Ref No 202101208.000, dated May 1, 2024), prepared By Englobe.

The Site is located in a low-lying area in a former meander of the Rideau River. The soils beneath the Site have discontinuous and irregular deposits of firm to stiff clayey soils. Based on these conditions, within the Preliminary Geotechnical Investigation, Englobe had recommended grade raises of up to 0.5 m to avoid consolidation of the underlying clays and a corresponding long-term settlement. Furthermore, Englobe had presented three foundation options, which included removal of the underlying clays, or deep foundations. Since issuance of the Preliminary Geotechnical Investigation, Englobe has been requested to assess the possibility of increasing the permissible grade raise to 1.0 m to accommodate the civil grading design. The purpose of this memo is to present the potential localized settlement at the Site in relation to a global site grade raise of 1.0 m.

No consolidation testing was performed as part of the Preliminary Geotechnical Investigation to enable specific settlement analysis of the clayey deposits. However, Englobe has performed a desktop review of nearby consolidation testing parameters in an attempt to quantify the possible settlement. **Based on our review, approximately 40 mm of settlement could be expected across the Site from the grade raise alone where clay deposits exists. Any future foundation loads will increase this settlement in the vicinity of the structures.** It is important to emphasize that the clayey deposits are discontinuous, and not enough geotechnical data across the Site has been performed to delineate locations of deep clays (more susceptible to settlement) and no clays (less susceptible to settlement).

The following three foundations options were presented in our original Preliminary Geotechnical Investigation:

- **Option 1**: Sub-excavate the FILL and upper native silts, clays, and sand down to competent native till and replace with new Engineered Fill;
- **Option 2**: Sub-excavate down to competent native till and have deeper footings founded on till and longer foundation walls; or
- **Option 3**: Use deep foundations such as driven piles/caissons or micro piles driven to bedrock refusal.

If one of these three options are considered, then the settlement of structures is not a concern as they will not be founded on clayey soils. However, any future underground utilities and hard surface features, such as retaining walls or hard landscaping, will continue to be subject to settlement and will need to be designed with flexibility and tolerance to increased settlement.

If designers wish to consider a 1.0 m grade raise, and any future shallow foundations with footings founded on the clays, then additional investigation and consolidation testing is recommended at the specific building locations. The 40 mm settlement estimated above, as a result of the grade raise alone, is greater that the typical tolerable settlement of a lightly loaded slab on grade structure. Any new structures to be constructed after the grade raise has been placed would require their own localized Geotechnical Investigation.

We trust this report meets your present requirements. Should you have any questions, please do not hesitate to contact our office.

Yours very truly,

Englobe Corp.

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