Don Lita Lift Station

Sudbury, ON

Geotechnical Investigation

City of Greater Sudbury c/o AECOM Canada Ltd.

Final GI Report

February 5, 2024
02306796.000

ENGLOBE

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Proposed Lift Station Upgrades

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Revisions and publications log

REVISION No.	DATE	DESCRIPTION
0A	November 23, 2023	Preliminary version published for comments
0B	January 15, 2024	2nd Draft for Client Comment
01	February 5, 2024	Final Report

Distribution

1 PDF copy	Client

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APPENDICES

1 Introduction

As requested by Aecom Canada Ltd., the Client, Englobe Corp. (Englobe) has carried out the geotechnical investigation to assess the soil conditions at the location of the existing Don Lita Lift Station. The proposed upgrades to the lift station that is located at 2226 Hudson Street in Sudbury, Ontario at UTM coordinates 5151628 N, 506330 E, Zone 17T (see Key Plan, Drawing No. 1, Appendix A). We have completed the field and laboratory testing programs and submit the results in this report along with our comments and recommendations.

The purpose of the geotechnical investigation to assess the ground conditions throughout the project area and to support the design work and development. It will also include hydrogeology testing and analysis to evaluate the potential inflow to open excavations and to evaluate the need for groundwater taking permits.

It is understood that the proposed upgrades consist of a new concrete Valve Chamber with a footprint of 4.3 m by 2.9 m (drawings provided by the Client), and the Lift Station Upgrades will require a slab-on-grade generator equipment foundation concrete pad.

1.1 Site Conditions

The proposed Don Lita Lift Station upgrades is located beside a residential area on the west side, the remaining surrounding area is forested.

Underground utility service clearances were undertaken in advance of the investigation. No buried services were identified at the area of the proposed borehole locations. However, there is a force main, a sanitary sewer, gas service, and street lighting within the vicinity.

See Photo Essay Appendix D for existing site conditions.

2 Fieldwork

The fieldwork for this geotechnical investigation was supervised on September 28th & 29th, 2023 by Manuel Welke of Englobe Corp. The fieldwork consisted of two (2) sampled boreholes (BH No. 1 - BH No. 2) to depth ranges of 10.1 m - 10.4 m below grade. In addition, a borehole to complete Dynamic Cone Penetration Testing (DCPT) was undertaken from surface to a depth of 8.1 m below grade (DCPT No. 1).

The borehole locations were laid out on-site by Englobe field staff at the area of the Don Lita Lift Station. The locations of the boreholes are shown on the Borehole Location Plan, Drawing No. 2 in Appendix A.

The boreholes and DCPT were advanced using a drill rig and two-man crew supplied by Marathon Underground using continuous flight hollow stem auger equipment. The field work was under the full-time direction and supervision of an experienced member of our engineering field staff who was responsible for underground service locates, retrieving samples, field sample classification, and overall field/drill supervision. Samples at the borehole location were obtained at frequent intervals of depth by using the Standard Penetration Test (SPT) method. The SPT method of sampling involves advancing a 50 mm outside diameter split spoon sampler with the force of a 63.5 kg hammer, freely dropping 760 mm, mounted in a trip (automatic) hammer. The number of blows per 300 mm penetration is recorded as the “N” value.

A monitoring well was installed in the BH No. 1 to a depth of 3.35 m below grade. The monitoring well (MW1) was installed to perform a hydraulic recovery test for the hydraulic conductivity assessment explained in Section 4.8.

The DCPT method involves advancing a 50 mm outside diameter hardened conical tip with the force of a 63.5 kg hammer, freely dropping 760 mm, mounted in a trip (automatic) hammer. The number of blows per 300 mm penetration is recorded as the “N” value. The conical tip is continuously advanced to gather a profile of the substrata’s strength without the need of augers or obtaining samples.

All samples taken during this investigation were stored in labeled airtight containers for transport to our North Bay laboratory for visual examination and select laboratory testing. The routine laboratory testing consisted of natural moisture content determination, particle size analysis, and Atterberg limits determination on select samples. Samples remaining after testing will be stored for a period of three months following the date of this report and then discarded unless otherwise instructed.

To comply with the intent of Ontario Water Resources Act Regulation 903 amended to O. Reg. 128/03, the boreholes were sealed with reverse augering techniques for the full depth and, where appropriate, the surface was sealed with a bentonite plug.

The borehole and DCPT locations were surveyed using a Handheld GPS survey equipment. A local benchmark (BM) is described as the top manhole lid (centre) located northwest of BH No. 1, with an elevation of 269.28 m (NAD83, provided by client) (See Drawing 2, Appendix A and Photo Essay, Photo 3 in Appendix D). These elevations have not been confirmed by an Ontario Land Surveyor (OLS) and, as such, must be confirmed by an OLS prior to use in design.

Table 2-1: Summary of Borehole and Auger probe, Locations and Depths

Borehole ID	UTM Easting (1) (m)	UTM Northing (1) (m)	Elevation (m)(2)	Depth (m)	Refusal Depth (m)
BH No. 1 (MW)	506334	5151624	268.84	10.1	10.1
BH No. 2	506326	5151621	269.01	10.4	N/A
DCPT No. 1	506334	5151622	268.99	8.1	8.1

Notes:
(1) UTM Zone 17T
(2) Based on elevations provided by Client

The routine laboratory testing consisted of natural moisture content determination, grain size distribution, and Hydrometer testing on select samples. Samples remaining following testing will be stored for a period of three months following the date of this report and then discarded unless otherwise instructed. All measurements in this report are in Metric units (unless otherwise noted).

3 Subsurface Conditions

Soil conditions are confirmed at the boring locations only and may vary between borings. The boundaries between stratums indicated on the logs are inferred from non-continuous sampling, results of in-situ tests (e.g., SPT, DCPT, etc.), observations during the drilling operations, and/or the response of the drilling equipment. These boundaries are approximations only and should not be regarded as exact planes of geological change as the actual transition may be gradual from one soil type to another. The description of compactness of the granular subsoils, in part, was based on the results of the SPT, DCPT and/or the response of the drilling equipment. The consistency of very fine cohesive subsoils, if encountered, was based on in-situ vane tests. Refusal is defined as the point at which the augers can no longer be practically advanced with the equipment used in this investigation. Refusal, if encountered, to further advance of the augers, DCPT, and SPT may have been due to the presence of very dense soils, cobbles/boulders in the underlying soils, or possibly bedrock. Defining the nature of auger refusal with diamond drilling operations was outside the scope of work for this project.

The borehole and DCPT locations are shown in Drawing No. 2 included in Appendix A. The subsurface conditions in the geotechnical boreholes are presented in the individual Borehole Logs (presented in Appendix B) and summarized in the following paragraphs.

3.1 Subsurface Summary Description

BH No. 1, BH No. 2 and DCPT No. 1 were advanced in the area of the existing lift station. The subsurface conditions of each borehole are summarized in Table 3-1 below.

Table 3-1: Summary of Observed Stratigraphy at the Discrete Borehole and DCPT Locations

Borehole ID	Approximate Depth (m)						
	Sandy Gravel	Organics & Sandy Silt	Sand	Silty Clay	Bedrock	DCPT	Refusal (Bedrock)[1]
BH No. 1	0.0 - 0.8	-	0.8 - 2.3 7.3 - 7.9	2.3 - 7.3	7.9 - 10.1	-	10.1
BH No. 2	0.0 - 0.15	0.15 - 0.3	0.3 - 0.8	0.8 - 10.4	-	-	-
DCPT No. 1	-	-	-	-	-	0.0 - 8.1	8.1

(1) Inferred Bedrock.

3.1.1 Sandy Gravel Fill

At the surface of all boreholes, a layer of sandy gravel fill was observed. The thickness of this material ranged from 0.15 m - 0.8 m. The sandy gravel can be described as having a trace of silt, grey in colour and damp in saturation.

3.1.2 Organics & Sandy Silt Fill

Beneath the sandy gravel, a thin layer of organics and sandy silt fill was observed to a depth of 0.3 m below grade at BH No. 2. The material can be described as grey in colour, with damp saturation. The moisture content of this material was on the order of 21%.

3.1.3 Sand

Underlying the sandy gravel at BH No. 1 from the depth ranges of 0.8 m - 2.3 m below grade, underlying the silty clay at depth ranges of 7.3 m - 10.1 m and underlying the organics and sandy silt in BH No. 2 from the depth ranges of 0.3 m - 0.8 m, a layer of sand trace to some silt and clay, trace organics was encountered. The material can be described as grey to brown in colour, and moist to wet saturation. The SPT 'N'-value of the sample obtained in this borehole ranged was in the order of 9 to 24 blows per 300 mm of penetration, indicating a very loose to compact compactness. The natural moisture content of this material was in the order of 17 % to 46 %.

3.1.4 Silty Clay

Underlying the sands at BH Nos. 1 and 2, from the depth ranges of 0.8 m - 2.3 m below grade, and at depth ranges of 7.3 m - 10.4 m, a layer of silty clay, trace to with sand was encountered. The material can be described as brown in colour, and damp to moist saturation. The SPT 'N'-value of the sample obtained in this borehole ranged was in the order of 0 (weight of the hammer) to 9 blows per 300 mm of penetration, indicating a very soft to stiff consistency. The natural moisture content of this material was in the order of 17 %. The estimated undrained shear strength from field vane tests on the samples obtained in this borehole ranged from 14 to 28 kPa indicating a soft to firm consistency , and with a sensitivity level ranging from 2 to 6 (medium sensitive to sensitive clay).

Gradation (hydrometer) analysis was carried out on three (3) samples of this deposit. Atterberg Limits Testing was also carried out on one (1) sample of this deposit, the results of these tests are summarized in Section 3.1.7 below and also summarized in Appendix C - Laboratory Test Results. The results of the Atterberg Limits Testing indicate this material is type CI.

3.1.5 Dynamic Cone Penetration Testing (DCPT)

Beginning at surface to a depth of 8.1 m below grade at DCPT No. 1, the cone probe was advanced by means of Dynamic Cone Penetration Testing (DCPT) equipment. DCPT blow count numbers ranged from 6 - 46 blows per 300 mm of penetration, with the final interval having blow counts of 46 blows (for 150 mm) at a depth of 8.1 m below grade.

3.1.6 Refusal Depths

The auger refusal encountered in BH No. 1 was preceded by an auger slight deviation from its vertical direction during the drilling operation from 7.9 to 10.1 m, which might be attributed to large boulder or a slopping bedrock face, close to vertical (indication of this condition can be seen at the site with exposed bedrock - outcrops, see Photo 7, Appendix D). DCPT No. 1 encountered refusal at 8.1 m with 46 blow counts in 150 mm penetration.

3.1.7 Laboratory Test Results

The following summarizes the laboratory data results obtained from relevant samples collected during the geotechnical investigation. Samples were obtained from the investigation at BH Nos. 1 and 2 at frequent intervals of depth by using the Standard Penetration Test (SPT) method, or by collecting an auger sample. The SPT method of sampling involves advancing a 50 mm outside diameter split spoon sampler with the force of a 63.5 kg hammer, freely dropping 760 mm, mounted in a trip (automatic) hammer.

The following laboratory tests were conducted to determine relevant geotechnical information at select borehole locations:

- Gradation (hydrometer)
- Atterberg Limits Testing

The following Table 3-2 summarizes the gradation results (sieve and hydrometer) obtained from conducting laboratory testing on the following samples:

Table 3-2: Gradation Results - Sieve & Hydrometer

Borehole & Sample ID	Description	Depth (m)	Gradation			
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH No. 1, SS3	Silty Clay	2.3 - 2.9	0	21	55	24
BH No. 2, SS2	Silty Clay	0.8 - 1.35	0	16	52	32
BH No. 2, SS6	Silty Clay	6.1 - 6.7	0	0	75	25

The following summarizes the Atterberg Limits Testing results on the samples obtained from conducting laboratory testing on the following samples:

Table 3-3: Atterberg Limit Results for Silty Clay

Borehole & Sample ID	Depth (m)	Atterberg Limits		
		Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
BH No. 2, SS4	3.05 - 3.65	37	18	19

The results of the Atterberg Limits on the silty clay sample indicates that the soils are medium-plasticity clays and can be classified as type CI.

3.2 Groundwater Data

Groundwater in the boreholes were measured upon completion. It is noted that there may have been insufficient time for the groundwater levels to stabilize in the Boreholes prior to measuring. The groundwater levels are recorded on the individual Record of Borehole Log Sheets (Appendix B) and also summarized in Table 3-4 below:

Table 3-4: Groundwater Level Measurements

Boring ID	Ground Elevation (m) (1)	Groundwater Depth (m) (2)	Cave-In Depth (m)	Groundwater Elevation (m)
BH No. 1 (MW)	268.84	2.37	N/A	266.47
BH No. 2	269.01	4.01	4.16	265.0

Notes:

(1) Based on elevations provided by Client

(2) Groundwater elevation measured immediately after completion of borehole and it is noted that groundwater elevation may not have had time to stabilize. Groundwater measured in monitoring well at BH No. 1 on November 10, 2023 after groundwater stabilization at a depth of 2.33 m below grade.

Groundwater levels will fluctuate seasonally and/or yearly. As such, the groundwater level should be established in advance of the construction operations (i.e. at time of tender or following award, prior to starting site work) such that adequate groundwater control plans can be developed.

The groundwater level should be considered at the existing grade elevation as a worst case scenario for design.

4 Discussion and Recommendations

This section presents an interpretation of the geotechnical data presented above, and provides general geotechnical and foundation design recommendations, and general discussion for design and construction of the proposed Don Lita Lift Station Upgrades described in Section 1.

The surface and subsurface conditions described in Section 3 are generally suitable for the proposed development. Provided that the new structures are relatively lightly loaded, and foundations are placed on or slightly within the competent overburden, spread foundations (strip, square) are suitable for preliminary consideration for the development. Further information on shallow foundations is provided in Section 4.2.

The generalized stratigraphy across the Site consists of sandy silt, sandy gravel and sand deposits overlying silty clay layer, with thickness between 5.0 (BH No. 1) and greater than 9.6 m (BH No. 2) at the area. Refusal was encountered in BH No. 1 (Auger) and DCPT No. 1 (46 blow counts in 150 mm) at 10.1 m and 8.1 m respectively.

4.1 Frost Protection

The estimated frost depth penetration for the area (OPSD 3090.101 Rev#1 Nov 2010) of the subject site is:

- 2.0 m below exposed asphalt surfaces or for isolated, unheated foundations;
- 1.5 m for exterior footings in a heated structure below exposed surface (i.e. adjacent sidewalks, etc.);
- 1.3 m for naturally insulated (i.e., snow cover) exterior footings for a heated structure.

It is noted soil types that have a high susceptibility to frost heaving (silt or soil with high levels of silt) were encountered at this site.

All exterior footings/foundation elements and isolated footings supported on soil and subject to frost penetration must have frost protection (permanent and during construction) to the depths noted above.

If a sufficient depth of earth cover cannot be provided for frost protection, equivalent expanded extruded polystyrene (EEP) insulation may be used in conjunction with available soils cover to provide frost protection. If EEP is used for frost protection, precautions must be taken to protect the insulation from accidental spillage of hydrocarbons, solvents or other destructive products.

Foundations (including conventional shallow, pile cap/grade beams) can be founded at a higher elevation provided they are supported on approved subgrades and insulated. The following general insulation design can be used. The following insulation design was based on the generalized design curves (Robinsky and Bespflug, 1973) for minimum insulation requirements for heated structures founded on clayey soil, synthetic insulation (i.e. Styrofoam SM, HI-40, HI-60, HI-100, depending on loading, or equivalent), minimum 50 mm thick, should be placed down the face of the foundation wall to the top of footing/underside of pile cap/grade beam, and then extend outwards horizontally beyond the foundation edge a minimum of 1.2 m. For unheated structures founded on clayey soil profile within the frost depth, synthetic insulation (i.e. Styrofoam SM, HI-40, HI-60, HI-100, depending on loading, or equivalent), minimum 100 mm thick, should be placed down the face of the foundation wall to the top of footing/underside of pile cap/grade beam, and then extend outwards horizontally beyond the foundation edge a minimum of 2.44 m. Beyond the building footprint, the horizontal insulation should be sloped downwards slightly (i.e. 2 to 3%) to promote drainage away from the structure. The insulation should be overlapped (or step jointed) and pegged or spot glued together. The insulation must be unbroken and any damaged pieces must be replaced. The insulation should have a minimum of 300 mm of permanent soil cover. To reduce the risk of damage to the polystyrene insulation from an accidental hydrocarbon spill, it is recommended that the insulation be covered, where appropriate, with a layer of 6 mil polyethylene (i.e. maintenance areas, garage entrances, below parking lots, etc.).

Soils that are sensitive to frost heave may experience heave during the winter/spring months, only to settle back once thawed. As such, the founding subgrades for footings, slab on grade, services, etc. must be protected from frost penetration at all times during foundation excavation and construction operations. Should freezing temperatures occur during construction, the Contractor must undertake to prevent frost penetration into the natural soils (straw, insulated traps, etc.) until such a time that footings, slab on grade, services, etc. are adequately protected (soil cover, insulation, heat is supplied to the building, etc.).

At the locations where shallow bedrock was found and the footings are founded on sound, unweathered bedrock, full depth frost protection is not required. This is provided that the geometry of the bedrock is such that groundwater flows away from the footings (i.e. groundwater will not pool adjacent to or underneath the footings).

Concrete cannot be placed against materials with subzero temperatures.

All granular backfill must be free of frost, ice, and snow, and at an appropriate moisture content and temperature to allow compaction. Once a lift of engineered fill is placed, compacted, and accepted, it is considered acceptable to backfill overtop of this lift if the lift is unfrozen or if there is minimal frost within the surface of the lift. If the surface of a granular fill lift is frozen, the Contractor shall, in conjunction with an Englobe representative, confirm depth of frost prior to backfilling. It is noted that frost penetration can be reduced through the use of insulated tarps, with or without heat source (depending upon ambient temperatures), and by ensuring backfilling operations are continuous.

In addition, active monitoring of the subgrade temperatures may be warranted depending upon the time of year that construction is undertaken.

If winter construction is anticipated, a detailed winter construction plan shall be provided by the Contractor prior to the commencement of the project.

4.2 Foundations

Based on the results of in situ and laboratory tests conducted to date presented in Appendix C, the following parameters are suggested as design parameters for the soil type encountered in the boreholes. The geotechnical soildesign parameters are summarized in Table 4-1.

Table 4-1: Suggested Soil Parameters for Geotechnical Design Analyses

Soil type	Unit weight (kN/m^3)	Undrained Shear Strength, kPa	Angle of Internal Friction, ϕ (Degree)[2]	Interface Friction Angle,δ (Degree)[2,3]	Modulus of Subgrade Reaction Ks (kN/m^3)[1]
Engineered Fill	21	N/A	33	20	30,000
Fill Sandy Gravel and Fill Sandy Silt	20-21 (20)	N/A	29 to 33 (30)[4]	20	10,000 to 20,000
Native Sand	20 to 21 (20)	N/A	29 to 33 (30)[4]	20	10,000 to 20,000
Native Silty Clay	17 to 18 (17)	40	22 to 25 (22)[4]	18 to 20	3,000 to 15,000

(1) Recommended parameters have been estimated based on visual observation of the soil conditions, results of measured field testing, laboratory test results, correlation with published information (Terzaghi, Peck, and Mesri, Third Edition; Kenney, 1959; Ohsaki et al. 1959; CFEM, 4th Edition) and our previous experience with similar materials.
(2) Design values are in brackets
(3) Interface between soil and concrete
(4) Provided that all organic inclusions can be removed
(5) N/A - Not Applicable

Unless noted otherwise, preliminary foundation design parameters are given for static, vertically and concentrically loaded foundations in compression. Dynamic, lateral, eccentric and uplift design parameters can be provided in the detailed geotechnical report, if applicable and requested by the structural engineer. All foundation design recommendations presented in this report should be considered preliminary and subject to refinements and change during subsequent supplementary investigation during more detailed design stages of the project. In addition, all recommendations are based on the assumption that an adequate level of construction monitoring during foundation excavation and installation will be provided. An adequate level of construction monitoring is considered to include:

a) For shallow foundations, examination of all excavation surfaces before engineered fill placement to ensure the suitability of the subgrade; and

b) For earthwork, full-time monitoring and compaction testing or engineered fill below footings.

Where unsuitable (e.g., peat/organic silt and others) or unstable (e.g., disturbed during construction or this investigation) soils are encountered during construction; the foundation soils must be removed to firm or compact native soils and replaced with Engineered Fill to the foundation grade. The unsuitable material should be excavated under the direction of a geotechnical engineer to competent material and then backfilled either with Granular ‘B’ Type I or II, material should be placed in lifts of 300 mm and compacted to 100% SPMDD with the optimum moisture or with a lean concrete mix.

Bearing areas will require very careful preparation. Following excavation all bearing and fill material placement surfaces should be cleaned of all organic, existing fill, loose, disturbed, or slough material prior to concreting or placing compacted fill material. Fill should be placed and compacted immediately following excavation to design grades. The fill should be placed and compacted in an unfrozen condition. Bearing surfaces should be protected always from rain, freezing temperatures and the ingress of groundwater before, during and after construction. Backfill against foundation walls should consist of an Engineered Fill. All foundation excavations and bearing surfaces should be inspected by a qualified geotechnical engineer to confirm the integrity of the bearing surface. All constructed foundations should be placed on unfrozen soils, which should be always protected from frost penetration. Final foundation drawings for construction will be reviewed by englobe to confirm footing dimensions and geotechnical recommendations.

4.2.1 Pad Foundation on Grade (GENSET)

The foundation for the new GENSET generator for the new ERV Unit shall be founded on a concrete slab on engineered fill. Beneath the proposed concrete slab, minimum 600 mm of existing soil shall be removed and replaced with engineered fill. In order to allow for the area of influence as well as synthetic insulation, the excavation will need to extend a minimum of 2.5 - 3 m beyond the outside face of the new foundation. This is to allow for insulation placement minimum 2.44 m extending horizontally past the face of the new foundation slab in accordance with Figure 13.11 of the Canadian Foundation Engineering Manual.

The subgrade at that level shall be proofrolled to the maximum in-situ density achievable and shown by in-situ testing.

It is recommended that footing bear on a granular pad of engineered fill, consisting of Granular A or Granular B Type II, over the approved and proofrolled subgrade. The granular pad is to be a minimum thickness of 600 mm below the underside of the foundation and extend 0.6 m width beyond the sides of the footing. The imported engineered Granular Type A or B Type II is to be compacted to 100% SPMDD.

The geotechnical resistance of the proposed bearing areas can be estimated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm. The geotechnical resistance at ULS was calculated by applying load resistance factor of 0.5 according to the 2006 Canadian Foundation Engineering Manual (4th Edition).

Table 4-2: Geotechnical Resistances and Reactions

Bearing Material	Depth of the footing, D (m)	Ultimate Bearing Capacity (kPa)	Factored Resistance at ULS (kPa)	Reaction at SLS (kPa)
Minimum 600 mm of Engineered Fill over Improved Subgrade	At grade	400	200	125

4.2.2 Valve Chamber Slab Foundation

It is understood that the precast concrete chamber to be installed will require at least a bearing capacity of 120 kPa at the base of the foundation slab. According to the drawings supplied by the client (S102 -Structural/ Valve Chamber Plans and Section), the proposed foundation (concrete slab) depth is anticipated at elevation 266.29 masl.

The generalized soil profile below the foundation slab of the Valve Chamber consists of soft to firm silty clay from the depth ranges of 0.8 m – 2.3 m below (BH No.1), and at depth ranges of 7.3 m – 10.4 m (BH No.2) below grade. This soil profile cannot provide the required soil bearing capacity indicated above without soil improvement and/or soil replacement. In order to reach the required bearing capacity, it is recommended soil replacement under the foundation level with a 2 m thick engineered fill pad below the proposed mud slab shown in the structural drawing S102. This engineered fill pad will require the extension of the shoring system (design by Others) to a suitable embedment depth for the excavation and backfill works.

The subgrade at that level shall be proofrolled to the maximum in-situ density achievable and shown by in-situ testing.

The granular pad of engineered fill will consist of Granular A or Granular B Type II, over the approved and proofrolled subgrade. The granular pad is to be a minimum thickness of 2.0 m below the underside of the foundation and extend 0.6 m width beyond the sides of the footing or to the shoring system. The imported engineered Granular Type A or B Type II is to be compacted to 100% SPMDD.

The geotechnical resistance of the proposed bearing areas can be estimated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm. The geotechnical resistance at ULS was calculated by applying load resistance factor of 0.5 according to the 2006 Canadian Foundation Engineering Manual (4th Edition).

Table 4-3: Geotechnical Resistances and Reactions

Bearing Material	Depth of the footing, D (m)	Dimensions of the footing, B x L (m)	Ultimate Bearing Capacity (kPa)	Factored Resistance at ULS (kPa)	Reaction at SLS (kPa)
Minimum 2.0 m of Engineered Fill over proofrolled Subgrade	At Elevation 266.29m	2.0 m x 3.0 m	250	125	50

Bearing areas will require very careful preparation. Following excavation, all bearing surfaces should be cleaned of all organic, loose, disturbed, or sloughed material prior to concreting or placing compacted engineered fill. Bearing surfaces should be protected at all times from rain, freezing temperatures and the ingress of groundwater before, during, and after construction. Subgrade dewatering should be anticipated and planned for. All foundation excavations and bearing surfaces should be inspected by a qualified geotechnical engineer to confirm the suitability of bearing surfaces and to confirm that the resistances provided in this report are consistent with what is observed during construction inspection.

4.3 Lateral Earth Pressure and Sliding Resistance

Temporary bracing and shoring may be designed using the typical soil coefficients and parameters given in Table 4-4, however the designer/contractor should verify the appropriate soil parameters for the designs of a specific bracing and shoring system. The design should incorporate the effects of hydrostatic pressure, traffic surcharge and retained sloping earth conditions in the bracing design. The following parameters may be used for design. The parameters are based on general representative values for the various soil types, obtained through laboratory testing and tactile analysis and are presented in the table below.

Table 4-4: Lateral Pressure Coefficients

Parameter	Granular A	Granular B Type I	Granular B Type II	Sandy Gravel and Sandy Silt Fill	Sand	Silty Clay
Angle of Internal Friction	35°	32°	35°	30°	30°	22°
Coefficient of Active Earth Pressure (K_a)	0.27	0.32	0.27	0.33	0.33	0.45
Coefficient of Passive Earth Pressure (K_p)	3.69	3.12	3.69	3.00	3.00	2.20
Coefficient of Earth Pressure at Rest (K_o)	0.43	0.48	0.43	0.50	0.50	0.63

The sliding resistance can be calculated using the following formula.

$F_r = \Sigma W (\tan\delta)$

Where,

F_r = base resistance to sliding (ultimate)

δ = Interface friction angle

ΣW= Total weight of the of vertical forces acting on footing.

A resistance factor of 0.8 should be applied to the ultimate sliding resistance in accordance with Canadian Foundation Engineering Manual (4th Edition).

4.3.1 Seismic Condition

In accordance with Canadian Foundation Engineering Manual (CFEM, 2023), Section 18.7.1, seismic loads shall be considered in the design. The designs shall take into consideration:

- The wall should be designed to withstand the combined static lateral loads plus the earthquake induced loads.
- The horizontal seismic coefficient (k_h) used to calculate the seismic active pressure coefficient is taken as 1.0 times the PGA for structures that do not permit lateral yielding and 0.5 times PGA for structures that permit lateral yielding;
- The vertical seismic coefficient (k_v) is typically 2/3 of k_h, and can be assumed to be 0 (conservative assumption); and
- Where sloping backfill exists above the top of the wall, the weight of the backfill above the top of the wall should be treated as a surcharge when calculating the lateral earth pressure under seismic conditions.

The Mononobe-Okabe (M-O) method was used to calculate the active earth pressure coefficients for yielding and non-yielding walls assuming that the angle of friction between the wall and backfill material is 0.5 Φ. The seismic active earth pressure coefficients provided in the following table may be used for designs. Passive pressures will not be mobilized by the design displacements and thus it should be assumed that passive thrust on a retaining wall will not develop.

Table 4-5: Lateral Pressure Coefficients - Seismic Condition

Wall Condition	Seismic Active Earth Pressure Coefficients (K)	
	OPSS Granular A or OPSS Granular B Type II Φ = 35° ; δ = 17.5° γ = 22.8 kN/m^3	OPSS Granular B Type I Φ = 32°; δ = 16.0° γ = 21.2 kN/m^3
	Horizontal Surface Behind Wall	
K_{AE} (Yielding Wall)	0.32	0.36
K_{AE} (Non-Yielding Wall)	0.38	0.42

For dry cohesionless backfill, the total active thrust can be calculated using the equation below:

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v)$$

Where: P_{AE} = Total active thrust (kN)
H = total height of the wall (m)
k_v = vertical acceleration coefficient (use 0)
K_{AE} = seismic active earth pressure coefficient (use Table 4-5 values)

It should be noted that the total active thrust calculated using the above equation is unfactored. This total active thrust, P_{AE}, can be further divided into a static component, P_A, and a dynamic component, ΔP_{AE}, as follows:

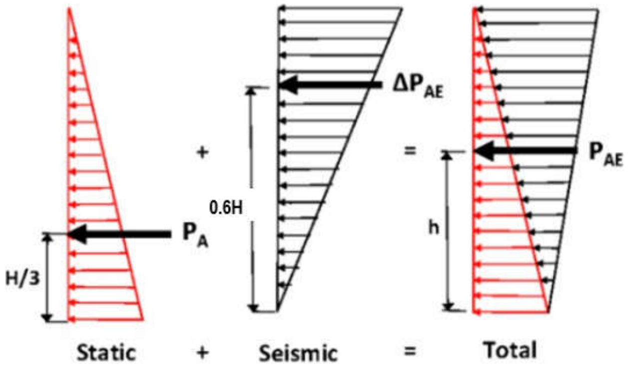
$$P_{AE} = P_A + \Delta P_{AE}$$

Where P_A = Static Active Earth Load (kN) = $P_A = \frac{1}{2} K_A \gamma H^2$

The total active thrust may be considered to act at a height h (to be calculated using below equation and stress distribution diagram), from the base of the wall as per CFEM:

$$h = \frac{P_A(H/3) + \Delta P_{AE}(0.6H)}{P_{AE}}$$

All the static earth pressure parameters for this equation are provided in Table 4-4.



4.4 Excavation, Dewatering, and Backfilling

Based on the Occupational Health and Safety Act Regulations for Construction Projects (O.Reg. 213/91, s. 226(4)), the existing fills and native soils are classified as follows:

a) Existing sandy silt and sandy gravel fills – Type 3
b) Native soils (sands) – Type 3
c) Native soils (clay) – Type 4

All excavations greater than 1.2 m in depth must be sloped or shored in accordance with the Occupational Health and Safety Act Regulations for Construction Projects. Short-term (i.e., day) open excavations will be stable above the groundwater table at a temporary angle of 1H:1V, however excavations established at this slope must not be left unattended at any time. Below the prevailing groundwater table, the slopes of open excavations will have to be flattened to 3H:1V or possibly shallower depending upon the method of dewatering employed, or possibly sheeted.

When approaching the founding soil subgrade surface, the excavating Contractor should use equipment that will not leave deep gouges in the bearing surface. If there are tooth gouges in the subgrade, these are indicative of disturbance and can collect water, further affecting the subgrade. It is strongly recommended that a ditching bucket or a bucket with a blade across the teeth be used to prepare a smooth subgrade surface.

Excavations must be maintained in a dewatered condition during excavation and foundation construction, and every reasonable effort must be made to prevent disturbing (piping/boiling) at the founding subgrade. At the time of the fieldwork, the groundwater table was encountered as shallow as 2.4 m below existing grade. Groundwater control, in accordance with OPSS.MUNI 517 and 518, will be required to maintain a stable subgrade during excavation and backfilling operations. The Contractor must undertake to establish the groundwater level in advance of the construction operations such that adequate groundwater control plans can be developed.

Where space limitations (from utility poles, existing underground services, above ground structures, etc.) do not permit overburden cut slopes at inclinations specified above, a steeper cut slope can be employed if supported by appropriately designed shoring, designed and constructed by a specialized shoring engineer/contractor. A support system comprising of a watertight caisson wall system may be suitable. Some movement/slumping of the cohesionless sandy soils should be expected.

An experienced contractor should be consulted during the design process to confirm the suitability of the vertical shoring method for the subsurface conditions encountered during drilling, specifically the ability to advance piles through boulders, and achieve an effective groundwater cut-off at the uneven bedrock surface. Also, the contractor selected for the work should be experienced and prepared to handle unexpected and/or difficult soil conditions.

Depending upon the final depth of foundation elements, typical localized dewatering during construction, such as installation of filtered sumps and pumping from sump holes in the base of the excavation may be required to maintain the excavation in a dewatered condition during subgrade preparation. The effectiveness of this method of groundwater control would be limited to conditions where excavations of less than 0.5 m below the prevailing groundwater table are anticipated, and the soil is such that the groundwater can be drawn down a minimum of 0.5 m below the working surface. If the excavation must penetrate to a greater depth below the prevailing groundwater table, a more effective groundwater control method such as a vacuum well point system with or without a sheet pile cut-off wall, should be considered by the contractor to maintain a stable excavation base. The Contractor's dewatering method must be designed to prevent piping.

The Environmental Protection Act (EPA) requires a person who is engaging in the prescribed water taking activities set out in *O. Reg. 63/16*, that meet the criteria set out in that regulation, to register those activities in the Environmental Activity and Sector Registry (EASR), and possibly obtain a Permit to Take Water (PTTW). An EASR or PTTW is required by the Ministry of Environment, Conservation and Parks if the daily taking of groundwater exceeds 50,000 L or 400,000 L per day, respectively.

Ultimately, the method of dewatering will be the choice of the contractor. The importance and benefits of maintaining a dry stable subgrade during excavation and foundation construction cannot be stressed enough. Failure by the contractor to adequately control the groundwater, and/or rainwater, surficial runoff, etc., can result in disturbance to the founding subgrades, which can result in having to carry out corrective measures (i.e., additional excavation, time delays, etc.) to improve the subgrade. Corrective measures required to improve subgrades where groundwater is not adequately controlled will be at the Contractors cost. As part of the Contractors proposed methodology of construction, the Contractor should be requested to submit a dewatering plan prior to commencement of the project that details how they will control groundwater. The plan should include all aspects from methodology (e.g., sump holes and pumps, drainage ditches, vacuum well points), to construction of system (sump hole details, placement, etc.), to operation of system, etc.

A geotextile separator (i.e. Terrafix 270R or equivalent) shall be placed between the engineered granular backfill material and any areas of finer grained soil (i.e. silt or clay).

The sand fills, upper silt and silty sand were not found to consistently meet any OPSS Form 1010 specification and can therefore only be used in areas of landscaping or elsewhere where movement of the ground surface is not of concern, unless a stockpiling and quality control segregation program is undertaken to identify those sands that may meet for Select Subgrade material.

Any soil to be removed from the Site will be considered excess soil and is subject to O. Reg. 406/19: On-Site and Excess Soil Management.

Any granular material to be used as engineered fill on this site must be tested and approved by this office prior to delivery to the site. It should be noted that engineered fill(s) should be placed in lifts of thickness less than the effective compaction depth of the equipment used to carry out the compaction operations (i.e., if using a heavy diesel Wacker, lifts should be a maximum of 300 mm thick, etc.).

4.5 Pipe Installations

Installation of services will occur mostly in the granular fill, organics and silty clay layers and bedrock based on borehole soil profiles. As previously noted, the fill and organic layers are not suitable for support (see below). Various installation methodologies will be required to accommodate the installation. The generalized stratigraphy of the site allows to place the pipes for various services at or below the estimated frost depth.

The possibility of bottom heave (earth pressure at the bottom of the excavation due to removal of trench soils) in the trench exists below the water table. Note that once heaved, a trench base would be considered unsuitable for pipe support. The base of the excavation should be closely monitored for vertical movements and disturbance. Backfilling of the trench should proceed as soon as possible after excavation.

4.5.1 Pipe Bedding

Pipe bedding should be in accordance with the following Ontario Provincial Standard Drawings (OPSD) design standards for the class and size of pipe being used as well as manufactures recommendations. Depending on the type of pipe, as well as on ground conditions (e.g., groundwater level, moisture content of the soils, etc.) at the time of construction, one or more of the following OPSD design standards may be applicable:

- OPSD 0802.010 Flexible Pipe Embedment and Backfill – Earth Excavation
- OPSD 0802.013 Flexible Pipe Embedment and Backfill – Rock Excavation
- OPSD 0802.030 Rigid Pipe Bedding, Cover and Backfill – Type 1 and 2 Soil - Earth Excavation
- OPSD 0802.031 Rigid Pipe Bedding, Cover and Backfill – Type 3 Soil - Earth Excavation
- OPSD 0802.033 Rigid Pipe Bedding, Cover and Backfill – Rock Excavation

Other OPSD standards or manufacturer requirements may apply to the construction of the buried services and the designer should consult these details as appropriate for the materials being selected for design.

Bedding pipe thickness shall follow recommendations the OPSD 802 series mentioned above. In the case of over-excavation, the material, required to bring the trench back to the required subgrade level, should consist of a well graded granular material compacted to 95% of standard Proctor maximum dry density (SPMDD) in accordance with OPSS 514.

The trench base should not be founded in organics. If organics are found at the base elevation of a trench, the trench should be extended through the organics (for a width equal to twice the depth plus the pipe diameter) and the grade restored, as noted above.

4.5.2 Trench Backfill and Compaction Standard

Compaction of the trench backfill will be necessary in some cases for the following reasons:

- To control anticipated settlement of the trench backfill;
- To provide lateral support to the trench sidewall; and
- To minimize soil loads on the pipe.

The granular fill, silt, sandy silt and silty clay soil layers at surface is unsatisfactory for backfill due to its high moisture and/or fines content. Stockpiled materials will be very susceptible to gaining moisture from rainfall which may render them unusable. In addition, stockpiles may require to be covered and protected from the effects of weather and moisture conditioning.

In general, trench backfill below roads should be compacted to 98% of standard Proctor maximum dry density. Where native soils are used below subgrade level, this may be 95% of standard Proctor maximum dry density at the natural moisture content for the full depth of the trench. This requirement may be waived where the above three criteria do not apply. Differential frost heave under a road will be minimized if the excavated soil is used as backfill below the road subgrade.

Heavy compaction equipment should not be used until at least 1 m of compacted backfill exists above the pipe. During backfilling, care should be taken to ensure the backfill proceeds in equal stages simultaneously on both sides of the pipe. Organic soils should be wasted. 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.

4.6 Site Drainage Recommendations

For those structures that do not have a basement, full perimeter footing drains and underslab drainage should not be necessary provided the top of floor slabs are a minimum of 300 mm above exterior grade.

For those structures where a basement is constructed, full perimeter footing drains and underslab drainage will be required. Drainage should consist of a minimum 100 mm diameter weeping tile or equivalent perforated pipe leading to a sump or other positive outlet. The weeping tile should be surrounded by an approved porous geotextile membrane to prevent the entry of fines into the system. For perimeter drainage, the invert of the pipe should be established a minimum of 300 mm below the top of the slab. Below the slab, the obvert of the drains should be a minimum of 100 mm below the bottom of the slab elevation. The pipes should be placed on a 100 mm thick bed of 19 mm clear stone and should also be surrounded on the sides and top with at least 100 to 150 mm of 19 mm clear stone or concrete sand. Perimeter drainage should be independent of underslab drainage.

The surface of the finished grade around the exterior of the building should be relatively impermeable and contouring of the perimeter exterior grade surface must direct all surface waters away from the structure.

4.7 Earthquake Parameters

The design peak ground acceleration (PGA) and Peak Ground Velocity (PGV) for the Lift Station area were calculated as 0.158 m/s and 0.189 m/s, respectively. The PGAs and PGVs were calculated with a 2%, 5 % and 10% probability of exceedance in 50 years based on the interpolation of the 2020 National Building Code Seismic Hazard calculation are attached in Appendix E.

Considering the geotechnical values, and based on 2020 Ontario Building Code, Table 4.1.8.4A, Site Classification for Seismic Site Response, the Lift Station area would have Site Class E (Site Designation X_E).

4.8 Hydraulic Conductivity and Inflow

4.8.1 Hydraulic Recovery Testing

Hydraulic recovery testing was performed on monitoring well MW1 (the well installed at BH No. 1) to estimate the in-situ hydraulic conductivity of the saturated formation that would be penetrated by excavations planned during construction. The effective screen intake of this monitoring well was set between 265.50 m and 267.00 m.

The groundwater level at the site was measured at MW1 on November 10, 2023, to be at an elevation of 266.50 m, or 2.33 m below the ground surface. From the borehole record, this indicates that groundwater was within the saturated silty clay horizon, below the uppermost sand horizon.

Hydraulic conductivity was tested by bailing the well vigorously to dryness, and subsequently measuring the water levels as they recovered with respect to time. The test results were then analysed using a method by Hazen, and the results are presented in Appendix F. In this method, a graphical analysis was made, utilizing the straight-line segment of data from the late recovery data to reduce the influence of sand pack recovery on the test results. The resulting hydraulic conductivity was calculated to be $2.33 \text{ X } 10^{-6}$ cm/s which is typical of the bulk conductivity of silty clay in the uppermost reaches of such a horizon.

4.8.2 Hydrogeology and Excavation Groundwater Inflow

The Don Lita lift station is situated at the edge of an urban residential subdivision. The surrounding landscape is a relatively level overburden (soil) covered area with occasional bedrock outcrops. The lift station is generally east of the subdivision with a rising bedrock outcrop terrain to the north and northeast. The station is generally at the same elevation as surrounding properties, with a slight grade to the south. Drainage is generally south and southeast, toward a generally low-lying area.

Hydrogeologic conditions were interpreted through geologic mapping (Northern Ontario Engineering Geology Terrain Study, or NOEGTS mapping), topographic and GIS mapping, and the borehole log stratigraphy developed as part of the borehole drilling. These conditions were reviewed for the purpose of developing estimates of groundwater impacts into the planned excavation at the Don Lita lift station.

NOEGTS mapping describes the site as a glaciolacustrine plain, with the primary surface material comprising sand, with a minor silty component. The landform is described as low relief, dry, with a suspected high-water table. Immediately southeast of the site, NOEGTS describes the terrain as being organic terrain with low local relief, peat and muck, and wet.

The borehole logs indicate that the upper soil profile is sand, with a thickness extending to 0.8 m metres below ground surface (m bgs) at BH No. 2 and to 2.3 m bgs at BH No. 1. Underlying the surficial sandy unit, the subsurface material is described as silty clay. The clay was found to be soft at the top, becoming softer with depth.

At BH No. 1, a 0.5 m thick layer of fine sand with some silt was observed immediately overlying bedrock, encountered at 7.9 m bgs. Bedrock was not encountered at BH No. 2 at a depth of 10.4 m bgs.

In terms of construction dewatering, it is understood that a 3m wide by 5m long pre-cast structure is to be set at a depth of 3 m bgs in the vicinity of BH No. 1, likely set on an imported granular pad overlying the clay unit. Some dewatering will be required to maintain dry subsurface conditions for placement of the pad and the structure. For the purpose of estimation, it was assumed that dewatering would be to reduce prevailing groundwater levels to 3.5 m bgs, and the overall excavation would be 5 m wide by 7 m long (allowing for 2H:1V slopes on granular pad edges). This configuration was used to estimate groundwater inflow.

For the purpose of calculation, the static groundwater elevation was set at 266.50 m, as measured on November 10, 2023. This would involve inflow from the clay unit only, estimated from the recovery testing to have a hydraulic conductivity of 2.33 X 10^{-6} cm/s. Additional calculation trials were run with the groundwater set at 0.5 m and 1.0 m higher than measured and contributing from within the upper sand unit. The upper sand was assumed to have a hydraulic conductivity of 5 X 10^{-3} cm/s.

Direct inflows to the pit were estimated using a method developed by Carslaw and Jaeger (1959), for unconfined flow of bank storage into an excavation. This solution results in an estimate of total inflow as a function of time. Using a conservative specific yield value of 15% for clay and 30% for sand, groundwater inflows were calculated and presented in Appendix F. The initial groundwater inflow to the dewatering area was 175 litres/day for the base case from the clay unit (groundwater at elevation 265.5 m, November 10, 2023) decreasing to 32 litres/day after 30 days. For the condition where prevailing groundwater is 0.5 m higher than measured, total initial inflows are estimated at 3,225 Litres/day on the first day, decreasing to 589 Litres/day on day 30. Further increases in groundwater to 1.0 m higher than measured result in an estimated inflow of 9,118 Litres/day on the first day, decreasing to 1,665 Litres/day after 30 days.

Based on the above estimates, subject to the assumptions and site-specific measurements made, it appears that groundwater inflows to the planned excavation will be below 50,000 Litres/day and no groundwater taking permit (i.e. no PTTW or EASR) will be required. Sufficient pumping capacity should be available to handle up to 10,000 Litres/day of groundwater inflow, plus the incidental rainfall that may occur during construction.

It is considered prudent to leave the groundwater monitoring well at BH No. 1 operational such that the groundwater table elevation can be measured as the construction date approaches. Observations of groundwater levels that are higher than the conditions observed and assumed above should be communicated to Englobe so that we may revise estimations of inflow and advise on the potential need for permits or the need to revise the construction schedule.

4.9 Pre-Construction Survey

It is noted that an adjacent City of Sudbury building is located approximately 44 m west of the work area at the site. For similar projects, the City of Sudbury has specified that pre-construction surveys be carried out on structures within 30 m of the work, however, as a precautionary measure, a pre-construction survey can be considered.

It is recommended that a pre-construction survey be carried out prior to the commencement of construction activities on the site which includes, at a minimum, the existing lift station and structures located on properties adjacent to the site.

Additionally, it is recommended a vibration monitoring program be implemented during all sheet piling and excavation/backfilling activities which includes, at a minimum, monitoring of the existing lift station and closest structure to the work.

4.10 Monitoring During Construction

All foundation design 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. An adequate level of construction monitoring is considered to be: a) for deep and shallow foundations: full-time monitoring and design review during construction; and b) for earthworks: full-time quality control and compaction testing.

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

In order to provide an adequate level of construction monitoring, qualified geotechnical personnel should manage and supervise the following tasks during construction:

Shallow and Deep foundations:

- Confirm that materials and methods meet specifications.
- Inspect foundation subgrades.
- Inspect excavation.
- Review shallow foundation installation/testing methods.
- Review compaction testing records.
- Provide review comments, including any discrepancies found with respect to specifications as well as this report, and the need for any modifications to the design or methods.

Earthworks:

- Confirm that materials and methods meet specifications.
- Inspect subgrade prior to any fill placement.
- Quality control of granular and select fill material.
- Review compaction testing records.

Pavement:

- Inspection of roadway subgrades prior to placement of pavement structures as per OPSS;
- Compaction testing and backfill monitoring;

An adequate level of construction monitoring for granular pavement materials is considered to be inspection of the subgrade and compaction testing. An adequate level of construction monitoring for placement of pavement structure is considered full-time field monitoring and compaction testing, as well as laboratory gradation and Proctor compaction testing of backfill material.

5 Limitations

The design recommendations given in this geotechnical report are applicable only to the project described in the text and only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known, in our analysis certain assumptions had to be made. The actual conditions may however, vary from those assumed, in which case changes and modifications may be required to our geotechnical recommendations. We recommend, therefore, that we be retained and provided the opportunity during the design stage to review the design drawings, site survey information, proposed elevations, etc. to verify that they are consistent with our recommendations or the assumptions made in our analysis. It is further recommended that we be retained to review the final design drawings and specifications relative to the geotechnical recommendations. If, during construction, conditions in the field vary from those assumed at the design stage, an engineer from this office must be notified immediately.

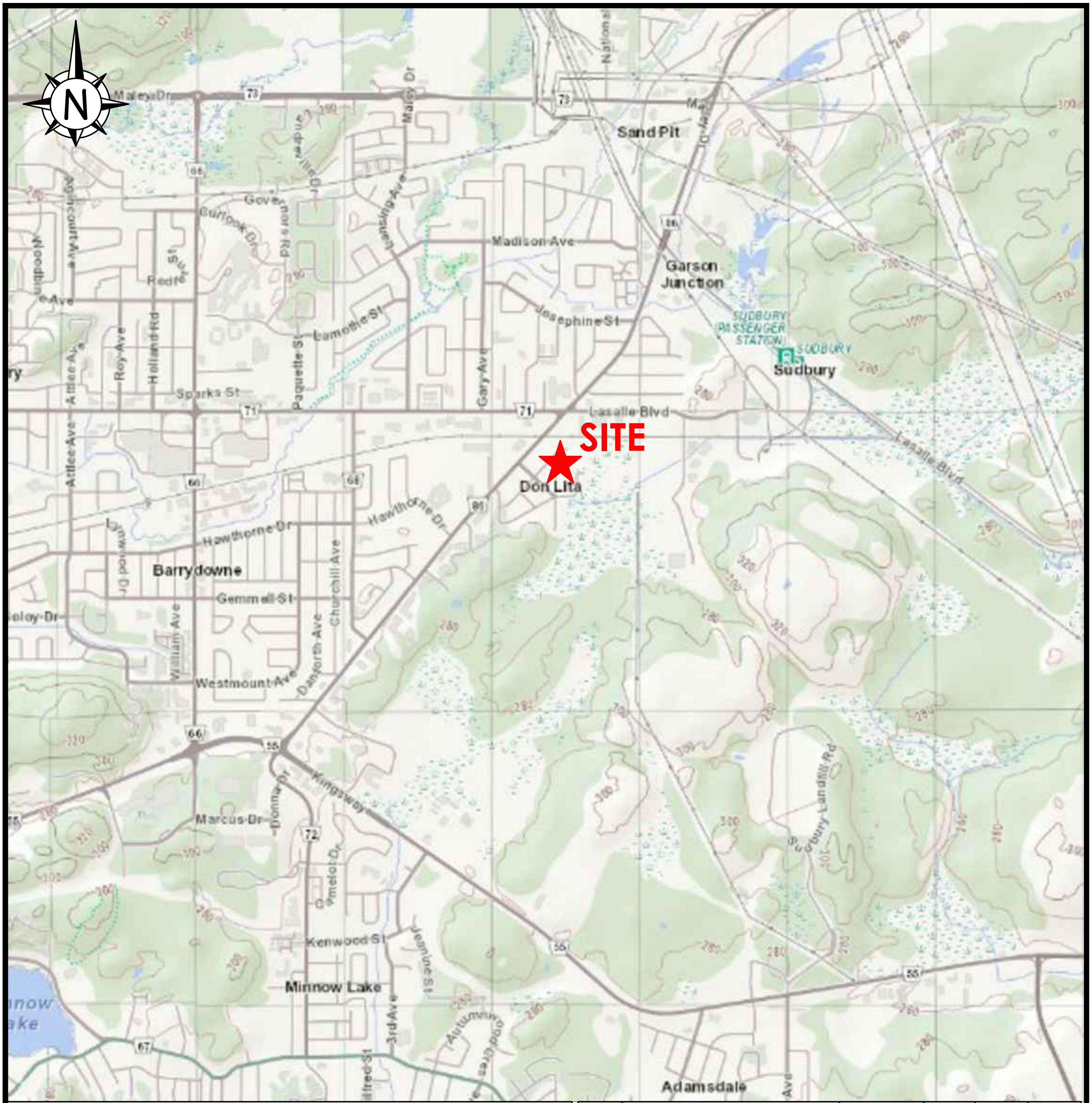
Proper subgrade preparation, groundwater control, compaction, etc. are all critical aspects of the bearing capacity of native soils. It must be noted that different aspects of the geotechnical design are based on the assumption that Englobe will be retained during site preparation and construction of the proposed works to ensure that both the geotechnical site characteristics and the construction operations/techniques are consistent with our recommendations. Should Englobe not be involved during the full construction phase, our liability is strictly limited to the factual information contained herein only.

The comments in this report are intended solely for the guidance of the design team and address the geotechnical conditions only. The number of boreholes required to determine the localized conditions between boreholes directly affecting construction costs, equipment, scheduling, etc. would in fact be greater than what has been carried out for design purposes. Inclusion of the factual information (Sections 1 to 3 inclusive) in the tender documents is furnished merely for the general information of bidders and is not in any way warranted or guaranteed by or on behalf of the owner or the owner's consultants and its subconsultants or the consultants' or subconsultants' employees, and neither the owner nor its consultants or its employees shall be liable for any representations negligent or otherwise contained in the documents. Therefore, contractors bidding on this project or undertaking this work should make their own interpretations of the factual borehole results and carry out further work as they deem necessary to assess the scope of the project.

Section 4 of this report is intended solely for the use of the client and the design team. If this section is provided to the Contractor, it is solely to provide an understanding of the geotechnical aspects of the site, and alternatives presented are not to be considered potential substitutes of the final design. If there is a discrepancy between this report and the tender documents and/or construction drawings, the latter shall govern and the discrepancy must be immediately brought to the attention of the design team.

Appendix A
Drawings

Drawing No. 1a and 1b	Key Plans
Drawing No. 2	Borehole Location Plan



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01	Final	2024/02/05	DMc	JMM	BV
No.	Version	Date	By	Verif	Appr.

City of Greater Sudbury c/o Aecom Canada Ltd.

Geotechnical Investigation
Proposed Upgrades to Don Lota Lift Station

2226 Hudson Street, Greater Sudbury, Ontario

Key Plan (Macro)

ENGLOBE

2–120 Progress Court
North Bay, Ontario, P1A 0C2
705-476-2550

Discipline: Geotechnical		Prepare by: DMc	Verify by: JMM
Scale: Not To Scale		Drawn by: DMc	Approval by: BV
Date: 2024/02/05		Drawing no: 1a	
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Man.	Project	Otp	Project	Phase			Electronic ref.	Rev.
EL	02306796	---	GE	-	--	--	----	01

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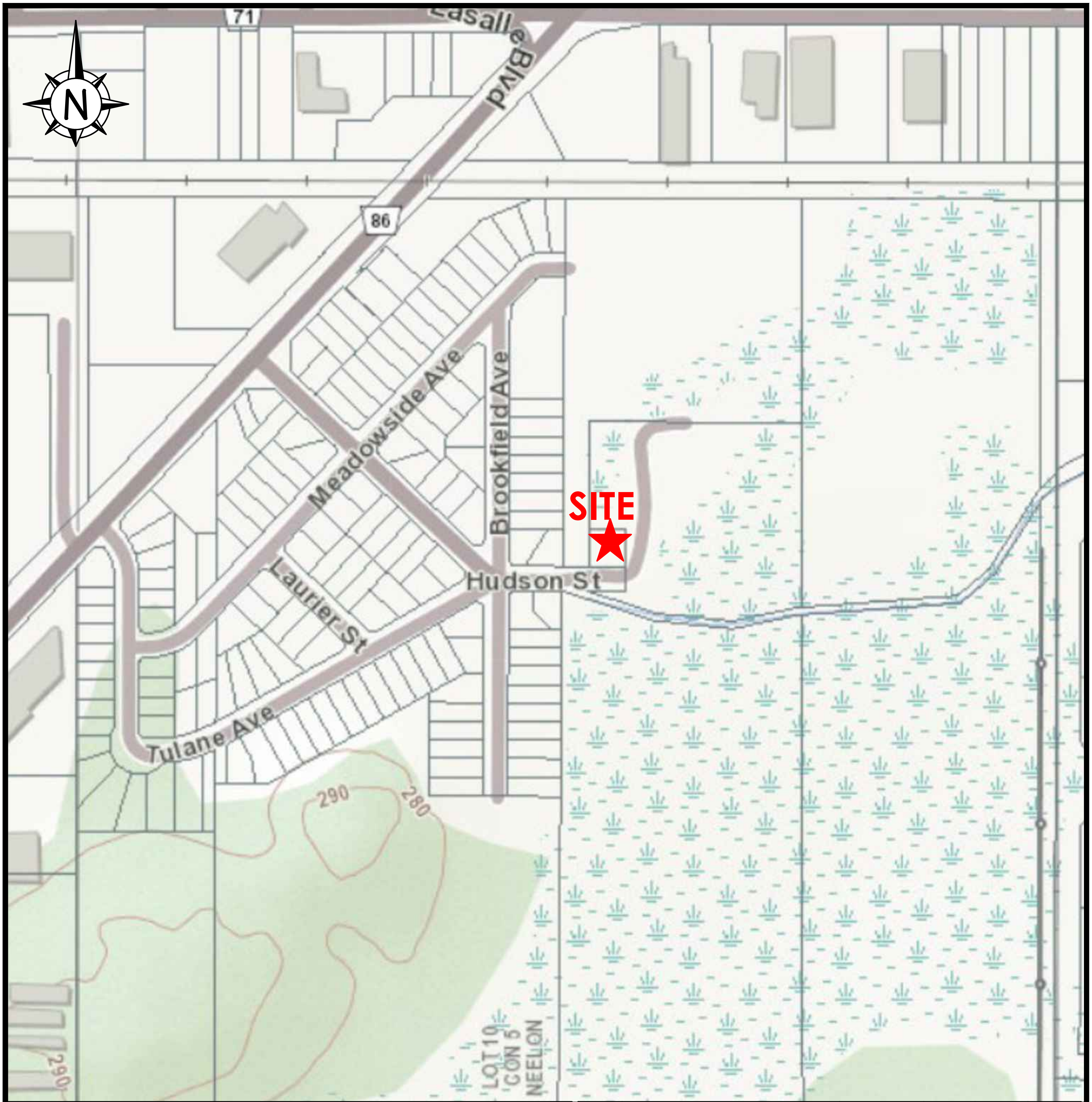

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01	Final	2024/02/05	DMc	JMM	BV
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City of Greater Sudbury c/o Aecom Canada Ltd.

Geotechnical Investigation
Proposed Upgrades to Don Lita Lift Station

Hudson Street, Greater Sudbury, Ontario

Key Plan (Micro)

ENGLOBE

2–120 Progress Court
North Bay, Ontario, P1A 0C2
705-476-2550

Discipline: Geotechnical
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Date: 2024/02/05
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Verify by: JMM
Drawn by: DMc
Approval by: BV
Drawing no: 1b
Register no.: ---------

Man.	Project	Otp	Project	Phase			Electronic ref.	Rev.
EL	02306796	---	GE	-	--	--	----	01

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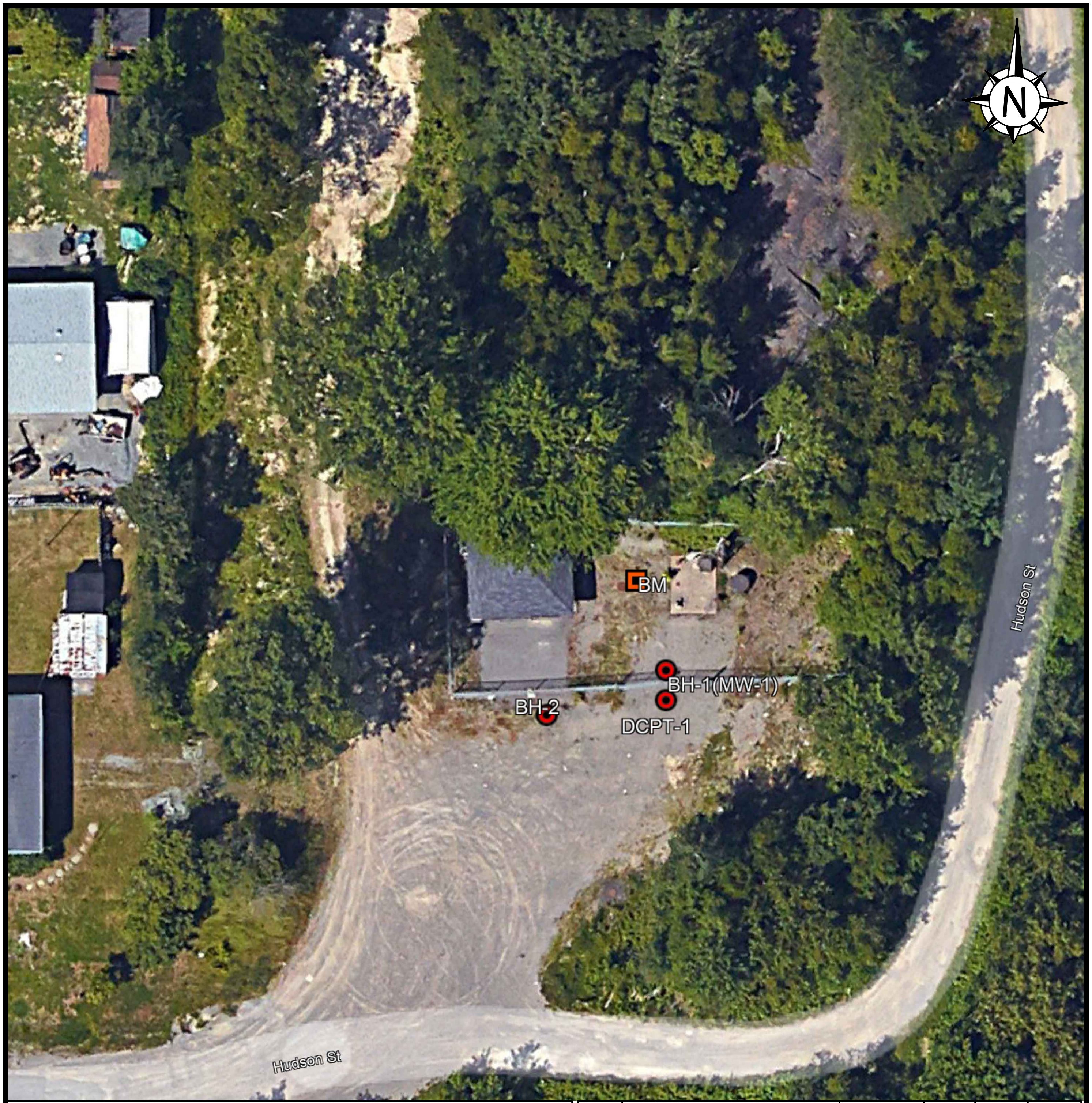

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No.	Version	Date	By	Verif	Appr.
01	Final	2024/02/05	DMc	JMM	BV

City of Greater Sudbury c/o Aecom Canada Ltd.

Geotechnical Investigation
Proposed Upgrades to Don Lita Lift Station

2226 Hudson Street, Greater Sudbury, Ontario

Borehole & DCPT Location Plan

ENGLOBE
2–120 Progress Court
North Bay, Ontario, P1A 0C2
705-476-2550

Discipline: Geotechnical
Scale: Not To Scale
Date: 2024/02/05
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Verify by: JMM
Drawn by: DMc
Approval by: BV
Drawing no: 2
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Appendix B
Borehole Logs

Enclosure No. 1	List of Abbreviations and Symbols
Enclosure No. 2-4	Record of Borehole Sheets

LIST OF ABBREVIATIONS & DESCRIPTION OF TERMS

The abbreviations and terms, used to describe retrieved samples and commonly employed on the borehole logs, on the figures and in the report are as follows:

1. ABBREVIATIONS

AS	Auger Sample
CS	Chunk Sample
DS	Denison type sample
FS	Foil Sample
NFP	No Further Progress
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
RC	Rock core with size & percentage of recovery
SS	Split Spoon
ST	Slotted Tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash Sample

2. PENETRATION RESISTANCE/"N"

Dynamic Cone Penetration Test (DCPT):

A continuous profile showing the number of blows for each 300 mm of penetration of a 50 mm diameter 60° cone attached to AW rod driven by a 63 kg hammer falling 760 mm.

Plotted as ——●——●——●——●

Standard Penetration Test (SPT) or "N" Values

The number of blows of a 63 kg hammer falling 760 mm required to advance a 50 mm O.D. drive open sampler 300 mm.

3. SOIL DESCRIPTION

a) *Cohesionless Soils:*

"N" (blows/0.3 m)	*Compactness Condition*
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

3. SOIL DESCRIPTION (Cont'd)

b) *Cohesive Soils:*

Undrained Shear Strength (kPa)	*Consistency*
Less than 12	very soft
12 to 25	soft
25 to 50	firm
50 to 100	stiff
100 to 200	very stiff
over 200	hard

c) *Method of Determination of Undrained Shear Strength of Cohesive Soils:*

+ 3.2 - Field Vane test in borehole. The number denotes the sensitivity to remoulding.

D - Laboratory Vane Test

¨ - Compression test in laboratory

For a saturated cohesive soil the undrained shear strength is taken as one-half of the undrained compressive strength.

4. TERMINOLOGY

Terminology used for describing soil strata is based on the proportion of individual particle sizes present in the samples (please note that, with the exception of those samples subject to a grain-size analysis, all samples were classified visually and the accuracy of visual examination is not sufficient to determine exact grain sizing):

Trace, or occasional	Less than 10%
Some	10 to 20%
With	20 to 30%
Adjective (i.e. silty or sandy)	30 to 40%
And (i.e. sand and gravel)	40 to 60%

5. LABORATORY TESTS

P	Standard Proctor Test
A	Atterberg Limit Test
GS	Grain Size Analysis
H	Hydrometer Analysis
C	Consolidation

SAMPLE DESCRIPTION NOTES:

1. **FILL:** The term fill is used to designate all man-made deposits of natural soil and/or waste materials. The reader is cautioned that fill materials can be very heterogeneous in nature and variable in depth, density and degree of compaction. Fill materials can be expected to contain organics, waste materials, construction materials, shot rock, rip-rap, and/or larger obstructions such as boulders, concrete foundations, slabs, abandoned tanks, etc.; none of which may have been encountered in the borehole. The description of the material penetrated in the borehole therefore may not be applicable as a general description of the fill material on the site as boreholes cannot accurately define the nature of fill material. During the boring and sampling process, retrieved samples may have certain characteristics that identify them as ‘fill’. Fill materials (or possible fill materials) will be designated on the Borehole Logs. If fill material is identified on the site, it is highly recommended that testpits be put down to delineate the nature of the fill material. However, even through the use of testpits defining the true nature and composition of the fill material cannot be guaranteed. Fill deposits often contain pockets or seams of organics, organically contaminated soils or other deleterious material that can cause settlement or result in the production of methane gas. It should be noted that the origins and history of fill material is frequently very vague or non-existent. Often fill material may be contaminated beyond environmental guidelines and the material will have to be disposed of at a designated site (i.e. registered landfill). Unless requested or stated otherwise in this report, fill material on this site has not been tested for contaminants however, environmental testing of the fill material can be carried out at your request. Detection of underground storage tanks cannot be determined with conventional geotechnical procedures.

2. **TILL:** The term till indicates a material that is an unstratified, glacial deposit, heterogeneous in nature and, as such, may consist of mixtures and pockets of clay, silt, sand, gravel, cobbles and/or boulders. These heterogeneous deposits originate from a geological process associated with glaciation. It must be noted that due to the highly heterogeneous nature of till deposits, the description of the deposit on the borehole log may only be applicable to a very limited area and therefore, caution must be exercised when dealing with a till deposit. When excavating in till, contractors may encounter cobbles/boulders or possibly bedrock even if they are not indicated on the borehole logs. It must be appreciated that conventional geotechnical sampling equipment does not identify the nature or size of any obstruction.

3. **BEDROCK:** Auger refusal may be due to the presence of bedrock, but possibly could also be due to the presence of very dense underlying deposits, boulders or other large obstructions. Auger refusal is defined as the point at which an auger can no longer be practically advanced. It must be appreciated that conventional geotechnical sampling equipment does not differentiate between nature and size of obstructions that prevent further penetration of the boring below grade. Bedrock indicated on the borehole logs will be labeled ‘possibly’ or ‘probable’ etc. based on the response of the boring and sampling equipment, surrounding topography, etc. Bedrock can be proven at individual borehole locations, at your request, by diamond core drilling operations or, possibly, by testpits. It must also be appreciated that bedrock surfaces can be, and most times are, very erratic in nature (i.e. sheer drops, isolated rock knobs, etc.) and caution must be used when interpreting subsurface conditions between boreholes. A bedrock profile can be more accurately estimated, at the clients’ request, through a series of closely positioned unsampled auger probes combined with core drilling.

4. **GROUNDWATER:** Although the groundwater table may have been encountered during this investigation and the elevation noted in the report and/or on the record of boreholes, it must be appreciated that the elevation of the groundwater table will fluctuate based upon seasonal conditions, localized changes, erratic changes in the underlying soil profile between boreholes, underlying soil layers with highly variable permeabilities, etc. These conditions may affect the design and type and nature of dewatering procedures. Cave-in levels recorded in borings give a general indication of the groundwater level in cohesionless soils however, it must be noted that cave-in levels may also be due to the relative density of the deposit, drilling operations etc.

ENCLOSURE NO.:2 (Pg. 1 of 1)

METRIC

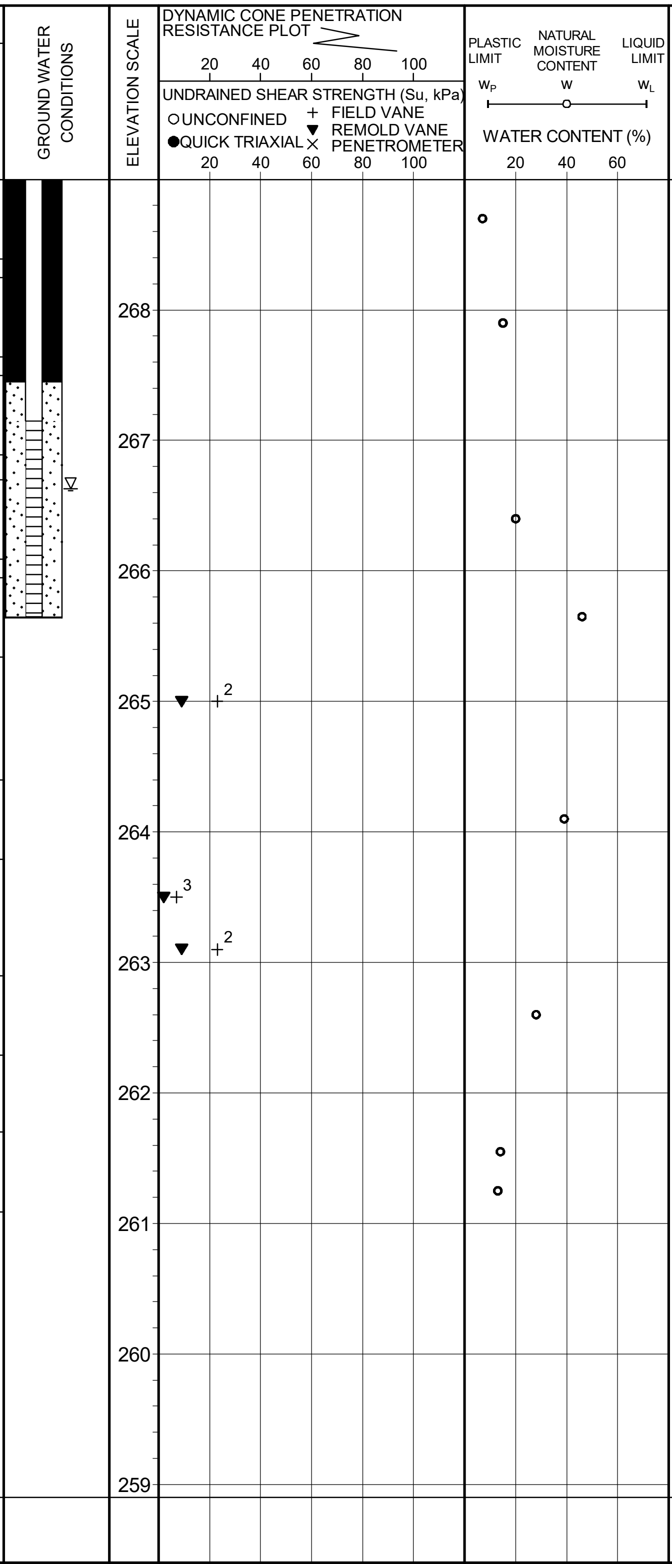
RECORD OF BOREHOLE/MONITORING WELL NO. 1

REFERENCE 02306796.000 DATUM Geodetic LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2 ORIGINATED BY MW

PROJECT Don Lita Lift Station BOREHOLE TYPE CME 850 Track Mounted - Hollow Stem Auger COMPILED BY DMc

CLIENT City of Greater Sudbury c/o Aecom Canada Ltd. DATE (Started) September 28, 2023 DATE (Completed) September 28, 2023 TIME (Completed) CHECKED BY JMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION (see Enclosure No. 1)	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
269.0					
0.0	FILL - SANDY GRAVEL - trace silt, grey, damp, compact	1	AS		
268.3 0.8	SAND - some clay, silt, trace organics, brown, damp, loose to compact	2	SS	9	
	- damp to moist		NR	12	
266.7 2.3	SILTY CLAY - with sand, grey, moist to wet, very soft to soft - sand & gravel seam	3	SS	1	0 21 55 24
		4	SS	WH	
	- trace to some fine grained sand, wet	5	SS	WH	
	- some fine grained sand	6	SS	WH	
261.7 261.6 7.5	SAND - coarse grained, some silt, trace clay, gravel, grey, wet, compact SAND - fine to coarse grained, some silt, trace clay, gravel, grey, wet, compact	7A 7B	SS	24	
			NR		
258.9 10.1	Auger Refusal on Possible Bedrock at 10.1 m bgs				



COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

+³, ×³ : Numbers on right refer to Sensitivity
Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 9/29/23 5:00:00 PM	2.37 ∇	-
2)	-	-
3)	-	-

MEL-GEO 2306796 - DON LITA LIFT STATION.GPJ MEL-GEO.GDT 1/12/24

ENCLOSURE NO.:3 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 2

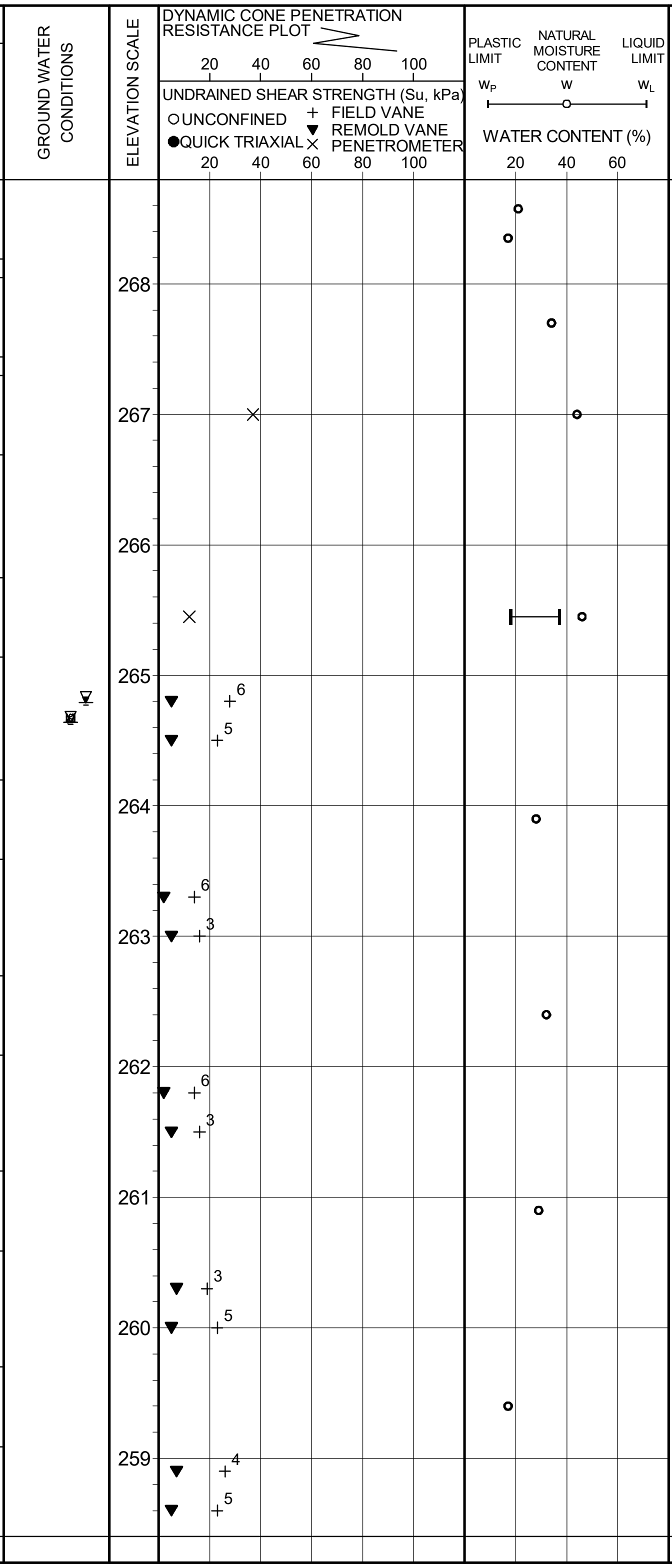
REFERENCE 02306796.000 DATUM Geodetic LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2 ORIGINATED BY MW
PROJECT Don Lita Lift Station BOREHOLE TYPE CME 850 Track Mounted - Hollow Stem Auger COMPILED BY DMc
CLIENT City of Greater Sudbury c/o Aecom Canada Ltd. DATE (Started) September 28, 2023 DATE (Completed) September 28, 2023 TIME (Completed) CHECKED BY JMM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)
268.8	
268.7	FILL - SANDY GRAVEL
268.5 0.3	FILL - ORGANICS & SANDY SILT - grey, damp, loose
	SAND - trace silt, brown, damp, loose
268.1 0.8	SILTY CLAY - some fine grained sand, brown, damp to moist, firm to stiff
	- grey, moist to wet
	- wet
258.4 10.4	End of Borehole at 10.4 m bgs

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	(SI	CL)
1A	SS	8				
1B						
2	SS	9	0	16	52	32
3	SS	5				
4	SS	WH				
5	SS	WH				
6	SS	WH	0	0	75	25
7	SS	WH				
8	SS	WH				



COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

+ ^{3}, × ^{3} : Numbers on right refer to Sensitivity
Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 9/28/23 12:25:00 PM	4.16	4.16
2) 9/28/23 1:00:00 PM	4.01	-
3)	-	-

Englobe Corp.
120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@englobecorp.com

ENCLOSURE NO.:4 (Pg. 1 of 1)

METRIC

RECORD OF DCPT NO. 1

REFERENCE 02306796.000 DATUM Geodetic

LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2

ORIGINATED BY MW

PROJECT Don Lita Lift Station

BOREHOLE TYPE CME 850 Track Mounted - Hollow Stem Auger

COMPILED BY DMc

CLIENT City of Greater Sudbury c/o Aecom Canada Ltd.

DATE (Started) September 29, 2023

DATE (Completed) September 29, 2023

TIME (Completed)

CHECKED BY JMM

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 UNDRAINED SHEAR STRENGTH (Su, kPa) ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL ▼ REMOLD VANE × PENETROMETER 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
269.0											
0.0	Dynamic Cone Penetration Test										
260.9 8.1	Dynamic Cone Refusal on Possible Bedrock 46 blow counts in 150 mm										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3$, $\times^3$: Numbers on right refer to Sensitivity
Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

Englobe Corp.
120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@englobecorp.com

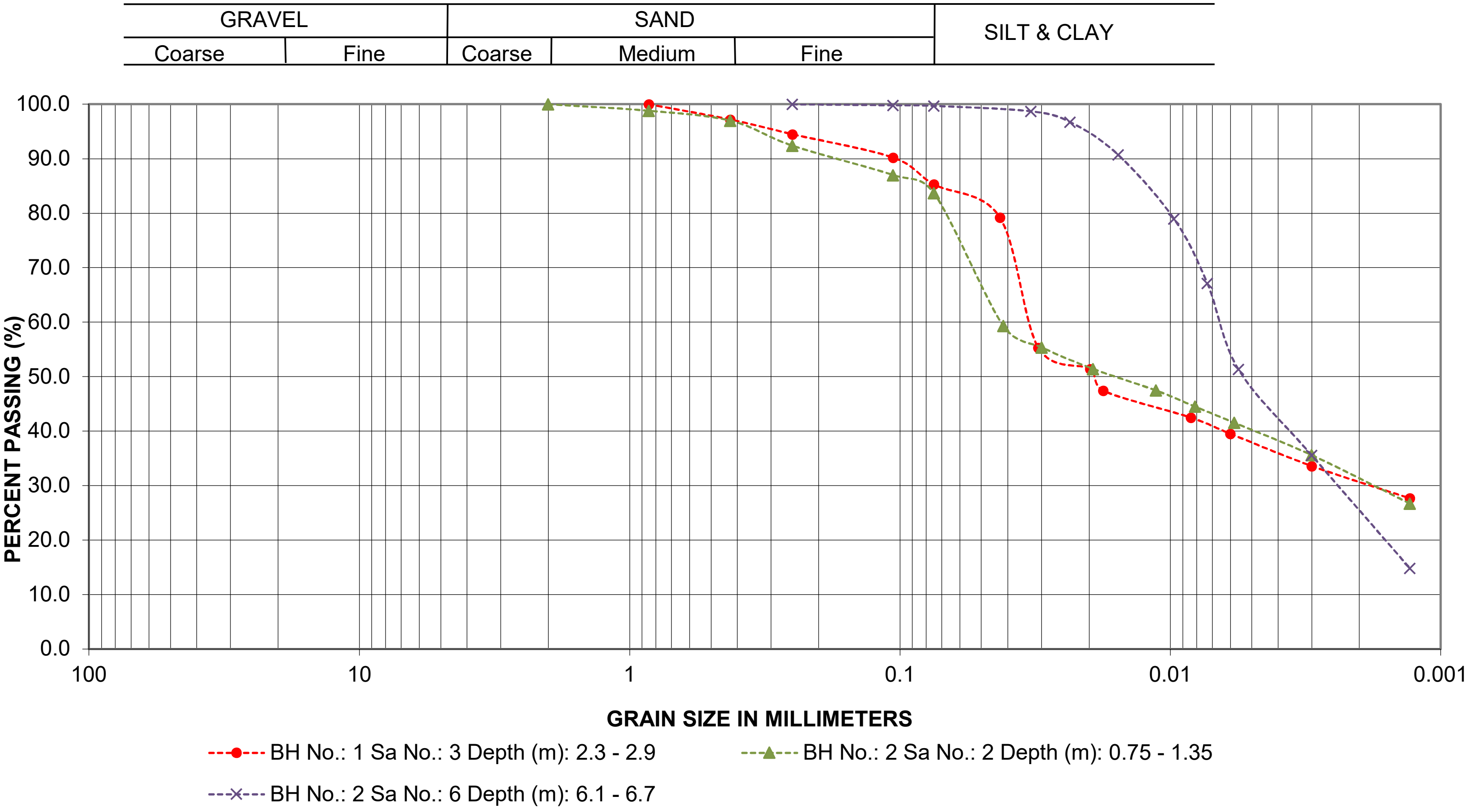
Appendix C
Laboratory Test Results

Lab Data

ENGLOBE

Reference No.: 02306796
Date: November 2023

GRAIN SIZE ANALYSIS



SILTS CLAYS

PROJECT: Upgrades to Don Lita Lift Station
LOCATION: 2226 Hudson Street, Greater Sudbury, Ontario

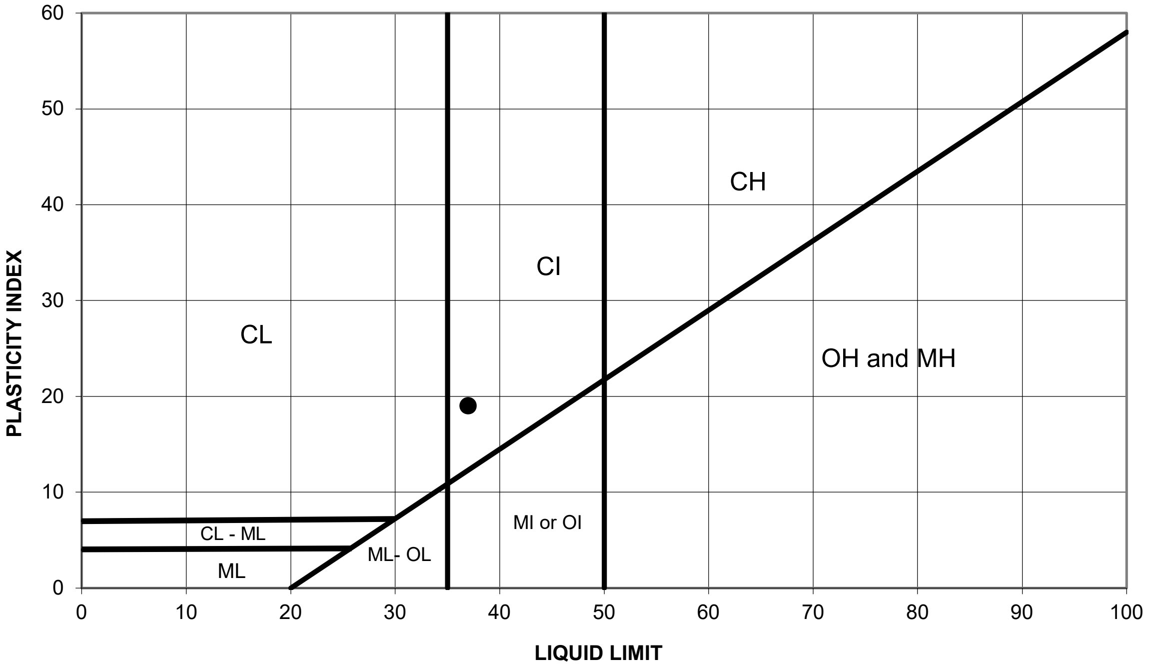
Englobe Corp.

FIGURE L-1

ATTERBERG LIMITS TEST RESULTS

FIGURE L-2

ATTERBERG INDICES



SYMBOL	BH No.	Sa No.	Depth (m)	Plasticity Index	Plastic Limit	Liquid Limit	NMC %
●	2	4	3.05 - 3.65	19.0	18.0	37.0	46.0

Date: 11/1/2023

Prepared By: DMc

Project: Upgrades to Don Lita Lift Station, 2226 Hudson Street, Greater Sudbury, Ontario

ENGLOBE CORP.

Appendix D
Photo Essay

Photo Essay

ENGLOBE

Drill rig used for geotechnical investigation	Photo: 1.

Project: GI, Don Lita Lift Station	Photos by: Englobe Date: September, 2023

Site overview (1 of 2)	Photo: 2.

Project: GI, Don Lita Lift Station	Photos by: Englobe Date: September, 2023

Site overview (2 of 2)	Photo: 3.

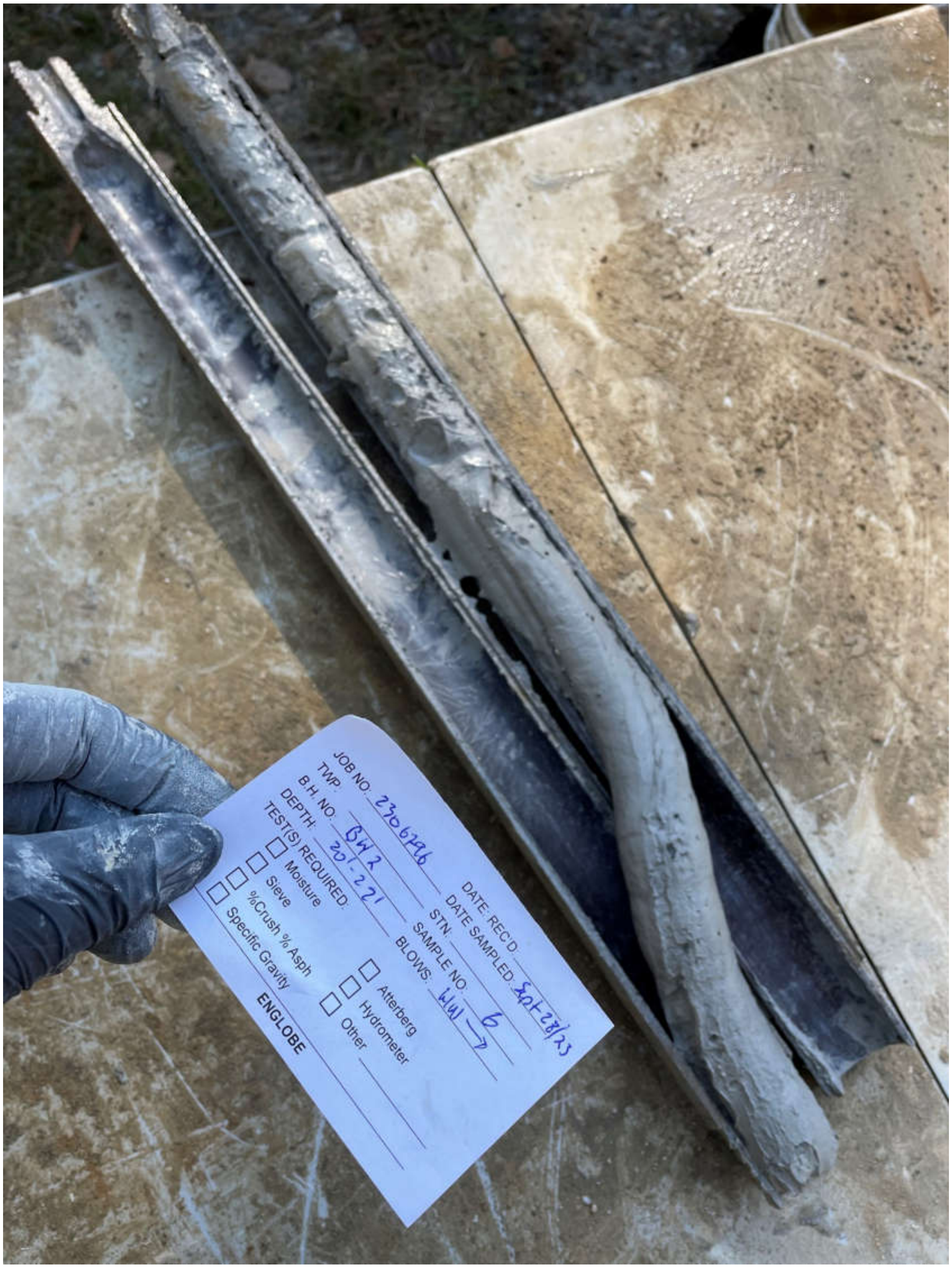
Project: GI, Don Lita Lift Station	Photos by: Englobe Date: September, 2023

Typical sandy soil encountered during investigation	Photo: 4.

JOB NO. 2306796 DATE REC'D
TYP. DATE SAMPLED Sep/18/23
B.H. NO. BH2 STN
DEPTH 0-2' SAMPLE NO. 1
BLOWS
TESTS REQUIRED
☐ Moisture ☐ Atterberg
☐ Sieve ☐ Hydrometer
☐ %Crush % Asph ☐ Other
☐ Specific Gravity
ENGLOBE

Project: GI, Don Lita Lift Station	Photos by: Englobe Date: September, 2023

Typical clay soil encountered during investigation	Photo: 5.



JOB NO.: 2306796
TWP.:
B.H. NO.: BW2
DEPTH: 20'-22'
TEST(S) REQUIRED:
☐ Moisture
☐ Sieve
☐ %Crush % Asph
☐ Specific Gravity
DATE. REC'D.:
DATE SAMPLED: Sept 28/23
STN.:
SAMPLE NO.: 6
BLOWS: WH →
☐ Atterberg
☐ Hydrometer
☐ Other

ENGLOBE

Project: GI, Don Lita Lift Station	Photos by: Englobe Date: September, 2023

Typical silt soil encountered during investigation	Photo: 6.

Project: GI, Don Lita Lift Station	Photos by: Englobe Date: September, 2023

Bedrock outcrop observed beyond fence line	Photo: 7.

Project: GI, Don Lita Lift Station	Photos by: Englobe Date: September, 2023

At-grade monitoring well installed at BH 1	Photo: 8.

Project: GI, Don Lita Lift Station	Photos by: Englobe Date: September, 2023

Typical patched borehole	Photo: 9.

Project: GI, Don Lita Lift Station	Photos by: Englobe Date: September, 2023

Appendix E
Seismic Hazard

2020 National Building Code Seismic Hazard Calculations

ENGLOBE

2020 National Building Code of Canada Seismic Hazard Tool

This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_E
Latitude (°)	46.518
Longitude (°)	-80.917

Please select one of the tabs below.

NBC 2020 | Additional Values | Plots | API | Background Information

The 5%-damped spectral acceleration ($S_a(T,X)$, where T is the period, in s, and X is the site designation) and peak ground acceleration (PGA(X)) values are given in units of acceleration due to gravity (g, 9.81 m/s^2). Peak ground velocity (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.281	0.303	0.186	0.0896	0.0236	0.00724	0.158	0.189

The log-log interpolated 2%/50 year $S_a(4.0, X_E)$ value is : **0.0327**

▼ Tables for 5% and 10% in 50 year values

NBC 2020 - 5%/50 years (0.001 per annum) probability

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.17	0.182	0.109	0.0501	0.0122	0.00376	0.0953	0.105

The log-log interpolated 5%/50 year $S_a(4.0, X_E)$ value is : **0.0172**

NBC 2020 - 10%/50 years (0.0021 per annum) probability

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.107	0.115	0.0663	0.0294	0.00663	0.00205	0.0597	0.0622

The log-log interpolated 10%/50 year $S_a(4.0, X_E)$ value is : **0.0095**

Appendix F
Hydraulic Conductivity Test Results

ENGLOBE

Groundwater Inflow Calculations

Clay Unit - Below 266.58m

Hydraulic Conductivity	2.33E-06	(cm/s)
Specific Yield	0.15	
Perimeter of Excavation	24	(m)
Drain Elevation (Excavation Base)	265.34	(m)

Sand Unit - Above 266.58m

Hydraulic Conductivity	5.00E-03	(cm/s)
Specific Yield	0.3	
Perimeter of Excavation	24	(m)
Drain Elevation (base of sand)	265.34	(m)

Inflow Calcs

GW at Nov 10 2023 level

	Clay Horizon
Groundwater El. (m):	266.51
Initial Transmissivity	2.36E-03
Ave. Charni Transmissivity	0.000817307

Time (days)	Inflow (L/day)
1	175
2	124
3	101
5	78
10	55
30	32
60	23

Based on Carslaw and Jaeger (1959) and Charni (1951)

Groundwater Inflow Calculations

Clay Unit - Below 266.58m

Hydraulic Conductivity	2.33E-06	(cm/s)
Specific Yield	0.15	
Perimeter of Excavation	24	(m)
Drain Elevation (Excavation Base)	265.38	(m)

Sand Unit - Above 97.3m

Hydraulic Conductivity	5.00E-03	(cm/s)
Specific Yield	0.3	
Perimeter of Excavation	24	(m)
Drain Elevation (base of sand)	266.58	(m)

Inflow Calcs

GW at Nov 10 2023 level + 0.5m

	Clay Horizon	Sand Horizon
Groundwater El. (m):	267.05	267.05
Initial Transmissivity	0.00336191	2.03E+00
Ave. Charni Transmissivity	0.00	0.70

Time (days)	Inflow (L/day)	Inflow (L/day)	Total Inflow (L/day)
1	299	2926	3225
2	212	2069	2280
3	173	1689	1862
5	134	1308	1442
10	95	925	1020
30	55	534	589
60	39	378	416

Based on Carslaw and Jaeger (1959) and Charni (1951)

Groundwater Inflow Calculations

Clay Unit - Below 266.58m

Hydraulic Conductivity	2.33E-06	(cm/s)
Specific Yield	0.15	
Perimeter of Excavation	24	(m)
Drain Elevation (Excavation Base)	265.38	(m)

Sand Unit - Above 266.58m

Hydraulic Conductivity	5.00E-03	(cm/s)
Specific Yield	0.3	
Perimeter of Excavation	24	(m)
Drain Elevation (base of sand)	266.58	(m)

Inflow Calcs

GW at Nov 10 2023 level + 1.0m

	Clay Horizon	Sand Horizon
Groundwater El. (m):	267.55	267.55
Initial Transmissivity	0.00436847	4.19E+00
Ave. Charni Transmissivity	0.00	1.45

Time (days)	Inflow (L/day)	Inflow (L/day)	Total Inflow (L/day)
1	443	8675	9118
2	313	6134	6447
3	256	5008	5264
5	198	3880	4078
10	140	2743	2883
30	81	1584	1665
60	57	1120	1177

Based on Carslaw and Jaeger (1959) and Charni (1951)

Hydraulic Conductivity Calculations - Hvorslev

JOB #:	2306796	
WELL	MW-01	
Test Date	10-Nov-23	
STATIC LEVEL (m top)		2.3300
BOREHOLE DIAMETER (m)		0.1000
RISER DIAMETER (m)		0.0510
FILTER PACK LENGTH (m)		1.5240

Time (Minutes)	Time (seconds)	Water Level (m)	Drawdown (m)	Hydraulic Conductivity (cm/s)
3	180	3.15	0.82	
7	420	3.13	0.80	5.642E-06
10	600	3.12	0.79	5.094E-06
18	1080	3.10	0.77	3.894E-06
23	1380	3.09	0.76	3.176E-06
28	1680	3.08	0.75	3.219E-06
33	1980	3.07	0.74	3.262E-06
44	2640	3.05	0.72	3.026E-06
62	3720	3.01	0.68	3.858E-06
78	4680	3.00	0.67	1.125E-06
92	5520	2.98	0.65	2.630E-06
103	6180	2.97	0.64	1.712E-06
Graph	1000		0.80	
	10000		0.60	**2.330E-06**