



Subsurface Conditions Baseline Report

Sanitary Sewer Installation at Station 15+850, QEW Widening from West of Mississauga Road to West of Hurontario Street, City of Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00

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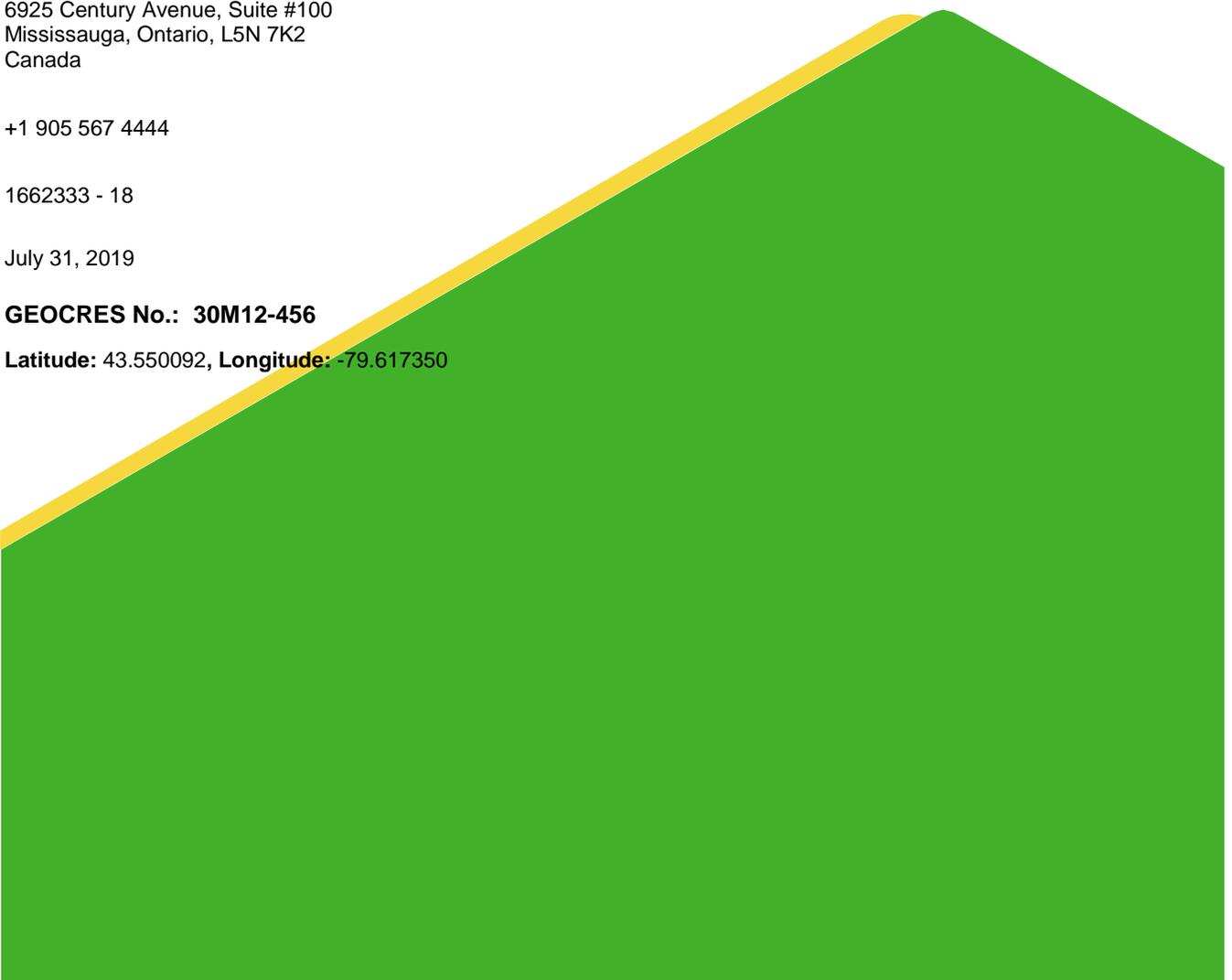
+1 905 567 4444

1662333 - 18

July 31, 2019

GEOCREs No.: 30M12-456

Latitude: 43.550092, **Longitude:** -79.617350



Distribution List

1 PDF & 1 Copy - Ministry of Transportation, Ontario (Central Region)

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to prepare this Subsurface Conditions Baseline Report (SCBR) for the sanitary sewer installation crossing the Queen Elizabeth Way (QEW) at about Station 15+850, associated with the widening of the QEW from west of Mississauga Road to west of Hurontario Street in the City of Mississauga, Ontario. This report consolidates and summarizes the results of geotechnical explorations and testing carried out at the site. This report is to be read together with the Contract Drawings and Specifications (Contract Documents) prepared by the project designers (MH) and the MTO. Bidders shall refer to the Contract Documents for the order of precedence in the event of conflicting information.

The purpose of this report is to describe and summarize the subsurface conditions anticipated at the crossing site and to establish the baseline subsurface conditions for the Contract. It forms the basis on which to judge whether the conditions encountered during construction are materially different from those anticipated at the time of bidding.

This report provides figures that summarize data and presents baseline subsurface conditions and geotechnical engineering parameters. For individual test results, the bidder is to refer to the Foundation Investigation Report for this sanitary sewer installation site, prepared by Golder (GEOCREC No. 30M12-448 dated July 30, 2019). Where alignments and stations are shown on the figures or referenced in the text, they are based on the General Arrangement drawings provided by MH. The stations referred to in this SCBR are approximate and the Contractor is expected to refer to the Contract Documents for the exact station, coordinates and details of existing and proposed features, structures and buried utilities.

This SCBR is intended to:

- provide a subsurface conditions baseline for bidding the work;
- assist the project Owner in reviewing the Contractor's submittals; and
- establish a subsurface conditions baseline that will be used to resolve disputes and claims related to subsurface conditions.

This SCBR has been prepared for a Design-Bid-Build construction contract for the installation of the primary liner and the sanitary sewer at the above-noted site and does not provide discussions on the anticipated ground behaviour in relation to specific construction means and methods because the Contractor is responsible for and will select the construction means and methods. This report does describe anticipated natural soil behaviour in the absence of any support or modification provided by the Contractor's means and methods. Therefore, the content of this report departs from typical practice, and this SCBR is not to be considered the equivalent of a Geotechnical Baseline Report, as defined in ASCE (2007).

The provision of baseline conditions in the Contract is not a warranty that the baseline conditions will be encountered; rather, the baseline conditions represent a contractual basis for the Owner and the Contractor to use when interpreting the differing site conditions clause in the General Conditions and Special Conditions of the Contract. The Contractor is to rely on this report for bidding and construction planning purposes related to anticipated ground and groundwater conditions and the Contractor is to plan construction and select equipment to fully address the expected baseline conditions identified in this report.

This SCBR is applicable only to the sanitary sewer installation section that will cross beneath the QEW using tunnelling methods and is not applicable to other elements of the QEW Widening from west of Mississauga Road to west of Hurontario Street or other sections of the sanitary sewer.

2.0 SITE AND PROJECT DESCRIPTION

The proposed sanitary sewer installation at Station 15+850 is located approximately 10 m west of the intersection of Sheridan Way and Indian Grove Avenue south of the QEW and extends from the grassy area between Mississauga Road and the Mississauga Road E – QEW W On-Ramp southwards to just north of South Sheridan Way, located south of the QEW, in the City of Mississauga, Ontario (see Drawing 1). The QEW is oriented in a northeast-southwest direction which at this location and for the purpose of this report, is referred to as west-east orientation.

The QEW consists of three eastbound lanes (to Toronto) and three westbound lanes (to Hamilton), while South Sheridan Way consists of one lane in each direction. Residential areas are located on the south side of South Sheridan Way and a golf course is located north of Mississauga Road. The existing ground surface along the sanitary sewer alignment varies from about Elevation 100.8 m at the north end of the sanitary sewer alignment, to about Elevation 101.2 m on the pavement surface of the QEW (westbound lanes), to about Elevation 100.5 m at the south end of the alignment.

3.0 SOURCES OF INFORMATION

The documents listed in this section have been used in developing the SCBR, but these are not to be considered part of the SCBR and publications are referenced herein for information purposes only.

Where precise determination of deposit boundaries or geotechnical engineering parameters are necessary for the safety and stability of the works, or for other construction concerns, or in instances where specialized geotechnical engineering properties of soils or bedrock are required but are not presented in the SCBR, these boundaries and parameters are to be identified and determined by supplementary investigations and testing by the Contractor prior to construction. This SCBR provides baseline conditions only for the physical subsurface conditions and does not provide baseline conditions for the chemistry of the soil, bedrock or groundwater.

3.1 Subsurface Data

Subsurface data gathered from multiple sources have been used in development of this report. The principal source of data is the Foundation Investigation Report, referenced below. The subsurface materials as characterized in the Foundation Investigation Report were defined at specific sample locations within the boreholes, and the Contractor is expected to review the specific subsurface data available in the Foundation Investigation Report. However, the interpretation of geotechnical engineering properties and parameters for the deposits and the stratigraphy as interpreted between samples provided in this SCBR are the baselines for this project. In the event of conflict between the Foundation Investigation Report and the SCBR, the SCBR shall be given precedence for the purpose of tendering and evaluating claims related to ground conditions.

- Golder Associates Ltd. "Foundation Investigation Report, Sanitary Sewer Installation at Station 15+850, QEW Widening from West of Mississauga Road to West of Hurontario Street, City of Mississauga, Ministry of Transportation, Ontario, GWP 2002-13-00", dated July 30, 2019, GEOCREs No. 30M12-448.

3.2 Geological References

The geological publications referenced in this document and listed below are for general information purposes only.

- Brennand, T. A. "Urban Geology of Toronto and Surrounding Area" in *Urban Geology of Canadian Cities*. GAC Special Paper 42, pp. 323-352. Karrow, P.F., and White O.L., Editors, Geological Association of Canada Special Paper 42, Geological Association of Canada, Newfoundland, 1998.
- Chapman, L.J. and Putnam, D.F. *The Physiography of Southern Ontario*. 3rd Edition, Ontario Geological Survey, Special Volume 2, 1984. Ontario Ministry of Natural Resources.

3.3 Publications

The publications referenced in this document, as listed below, are for general information purposes only.

- ASCE (2007). *Geotechnical Baseline Reports for Construction: Suggested Guidelines*. The Technical Committee on Geotechnical Reports of the Underground Technology Research Council, R.J. Essex, chairman, ASCE, Reston, VA, 62 pp.
- Boone, S.J., Westland, J., Busbridge, J.R., and Garrod, B. (1998). "Prediction of Boulder Obstructions", In *Tunnels and Metropolises, Proceedings of World Tunnel Congress 1998*, Sao Paulo, Brazil. A. Negro and A. Ferreira, Editors, Balkema, Rotterdam, pp. 817-822.
- Brown, 1981, "Suggested Methods for Rock Characterization Testing and Monitoring", International Society for Rock Mechanics.
- Canadian Geotechnical Society (2006). *Canadian Foundation Engineering Manual*, 4th Edition. BiTech Publishers Ltd., Richmond, British Columbia.
- Heuer, R. E. (1974). "Important Ground Parameters in Soft Ground Tunneling", in *Proceedings Specialty Conference on Subsurface Explorations for Underground Excavations and Heavy Construction*, ASCE, Reston, VA., pp. 152-167.
- Kulhawy, F.H. and P.W. Mayne. (1990). *Manual on Estimating Soil Properties for Foundation Design*. Report EPRI-EL6800. Palo Alto, CA, Electric Power Research Institute.
- Ministry of the Environment Ontario (2005). Water Well Information System, Version 2.01. Hydrogeology of Southern Ontario, Second Edition. http://www.ene.gov.on.ca/envision/techdocs/4800e_index.htm
- Poot, S., Boone, S.J., Westland, J., and Pennington, B. (2000). "Predicted Boulder Frequency Compared to Field Observations During Construction", in *Proceedings of the 50th Canadian Geotechnical Conference*, Montreal, pp. 47-54.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The interpreted and simplified stratigraphic profile shown on Drawing 2 is the baseline stratigraphy for this project and is a simplification of the subsurface conditions encountered at and between the borehole locations.

Although interpreted strata boundaries are illustrated on Drawing 2, it must be understood that the stratigraphic boundaries illustrated on Drawing 2 are inferred from non-continuous sampling, observations of drilling progress and results of Standard Penetration Tests, and, therefore represent transitions between soil types rather than exact planes of geological change; actual contacts between deposits will typically be gradational as a result of natural geologic processes. Further, the boundaries shown on Drawing 2 are illustrated for the indicated section line and are based on projection of the subsurface data onto this line. Variations in the deposit boundaries and the boundaries of major intra-deposit zones from those illustrated must be anticipated both along and perpendicular to the profile line. Therefore, selection of construction equipment and procedures must be made to accommodate variations in the deposit boundaries as described in this SCBR. Where precise determination of deposit boundaries is necessary for the safety and stability of the works, or for other construction concerns, they are to be verified by supplementary investigations and testing by the Contractor prior to construction.

In summary, the stratigraphy encountered at the various borehole locations typically consists of surficial layers of asphalt / concrete pavement and fill. The fill is underlain by a cohesive clayey silt to silty clay deposit, which in turn is underlain by shale bedrock.

Within this SCBR, the stratigraphy is defined and described based on the likely geologic origin, grain size distribution, plasticity characteristics and relative elevation. This approach is used to avoid geologic unit classifications based on geologic age or stage of glacial advance. In some instances, geologic nomenclature, although correct in defining the geologic origin and age of a particular layer, does not necessarily convey indications of material type or potential engineering behaviour. Precedence in this SCBR has therefore been given to naming the different soil layers based on relative elevation, grain size distribution and plasticity characteristics.

4.2 Baseline Engineering Characteristics of Soil and Rock Materials

This section of the SCBR provides baseline geotechnical engineering parameters to be used for design of temporary works and for selection of equipment and construction methods. The baseline geotechnical engineering parameters presented are those considered relevant for the proposed installation of a reinforced concrete pipe primary liner for a sanitary sewer. Baseline values are provided consistent with 10th, 50th, and 90th percentiles, as a means for quantitatively describing the statistical distribution of the parameter values and their natural variability. In some cases, the percentiles are based directly on statistical evaluation of available data and, in other cases, these values are supplemented by judgement based on local and regional experience with these soil and rock types. While the 50th percentile value can be used for some design purposes, the range represented by the 10th to 90th percentiles must also be considered as variability in physical properties is intrinsic to the nature of earth materials and is to be taken into account for estimating quantities, selection of equipment, and selection of construction means and methods.

When comparing actual encountered conditions to the baselines described in this report, where additional sampling and testing is carried out by the Contractor, any new data must be compared to the baselines presented in this report, using appropriate statistical methods that account for the numbers of tests completed, in order to determine whether the new data represents a materially different subsurface condition.

4.2.1 Baseline Engineering Characteristics of Soil

Engineering Classes A and B, specific to this report and identified with colours on the baseline stratigraphic profile, group soil types in relation to anticipated natural behaviour during construction if exposed and in the absence of support or other modification provided through the Contractor's means and methods. The Engineering

Classes used in this report are described in Table A following the text of this SCBR and in subsequent sections of this SCBR.

4.2.1.1 Asphalt / Concrete Pavement

Where boreholes were advanced through the existing pavement structures, an approximately 100 to 150 millimetre (mm) thick layer of asphalt pavement was encountered at ground surface in Boreholes C2-2, C2-3 and NW4-5. A 200 mm and 240 mm thick layer of concrete was encountered underlying the asphalt pavement in Boreholes C2-2 and C2-3, respectively. While asphalt pavement materials were encountered within some of the boreholes, this SCBR does not provide baseline characterizations of thicknesses, extents or locations of pavement materials.

4.2.1.2 Topsoil

No layers of topsoil were encountered at ground surface in any of the boreholes. Testing of topsoil that may be found within the contract limits for organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

4.2.1.3 Fill (Class A)

Approximately 0.6 m of Fill, comprised of clayey silt with varying proportions of sand and organic matter, was encountered at ground surface outside of the limits of the roadways. Approximately 0.5 m to 0.9 m of granular fill comprised of gravelly sand to sand and gravel was encountered underlying the asphalt / concrete within the limits of the roadway and underlying the cohesive fill outside the limits of the roadways at depths of between 0.2 m to 0.6 m. Fill is considered to be associated with the local road structure, QEW construction, nearby utility trenches and development of the adjacent residential lands. Grain size distribution of Fill will vary widely from primarily cohesive soils to sand and gravel materials used for utility bedding and roadway construction.

A baseline grain size distribution envelope of Fill (Class A) is presented on Figure 1. Baseline classification and engineering parameters for Fill (Class A) are provided in Table 1 below. SPT “N”-values, water content, and grain size distribution percentages are based on field sampling and lab testing. However, no Atterberg limits tests were performed on the Fill. Baseline SPT “N”-values provided within Table 1 are for samples where a full 457 mm of penetration could be accomplished and appropriate hammer blows summed for the last 300 mm of penetration. The baseline SPT “N”-values provided in Table 1 below are, therefore, considered representative of the soil matrix. For baseline purposes, it is expected that within the Fill Deposits approximately 6 per cent of all SPT “N”-values will not achieve full penetration and exhibit more than 50 hammer blows for less than 0.15 m of penetration, considered representative of driving the sampler on very dense soil, gravel, cobbles and/or boulders.

Table 1: Baseline Geotechnical Parameters for Fill (Class A)

Parameter	10 th Percentile ¹ .	50 th Percentile ¹ .	90 th Percentile ¹ .
SPT “N”-Value ² .	8	20	47
Water Content, w _n (%)	3	6	12
Gravel (%) ³ .	21	32	45
Sand (%) ³ .	36	54	65
Fines (%) ⁴ .	10	16	33

Parameter	10 th Percentile ¹ .	50 th Percentile ¹ .	90 th Percentile ¹ .
D ₁₀ (mm)	0.02	0.04	0.09
D ₆₀ (mm)	0.40	3.0	5.76
Coefficient of Uniformity, Cu	22	71	235
Wet Unit Weight, kN/m ³	19	21	23
Effective Angle of Internal Friction, ϕ' (degrees)	30	33	36
Estimated Permeability, k (m/s)	3x10 ⁻⁴	1x10 ⁻³	7x10 ⁻³

- 1) While for any given sample, percentages of gradation categories (e.g., gravel, sand, fines) will add to 100 per cent, percentile values do not necessarily add to 100 per cent because these statistics represent ranges of separate measurement parameters.
- 2) Baseline blow counts are based on SPT "N"-values measured for 0.3 m of penetration; for tests that did not penetrate a full 0.3 m, the blow count values were increased in proportion to the fraction of a standard 0.3 m penetration actually achieved, to a maximum equivalent SPT "N"-value of 150 blows per 0.3 m. Where a value of 150 blows per 0.3 m penetration is indicated within this report, this must also be interpreted that these values represent effective refusal to penetration of SPT equipment.
- 3) Total weight of particles as compared to the total sample weight.
- 4) "Fines" content refers to the total weight of particles passing the 0.075 mm sieve opening size as compared to the total sample weight.

4.2.1.4 Cohesive Deposits (Class B)

Cohesive Deposits consisting of clayey silt to silty clay was encountered at depths of about 0.7 m and 1.0 m below ground surface, respectively.

The baseline grain size distribution envelope for the cohesive deposits (Class B) is presented on Figure 2. The baseline envelope for Atterberg Limits is presented on Figure 3, which indicates that these materials have medium plasticity. Baseline values for other geotechnical engineering parameters for the cohesive deposits are provided in Table 2 below.

Table 2: Baseline Geotechnical Parameters for Cohesive Deposits (Class B)

Parameter	10 th Percentile ¹ .	50 th Percentile ¹ .	90 th Percentile ¹ .
SPT "N"-Value ² .	3	13	64
Water Content, w _n (%)	12	20	30
Gravel (%) ³ .	1	3	17
Sand (%) ³ .	2	13	31
Fines (%) ⁴ .	68	82	98
Plastic Limit, PL (%)	14	19	21
Liquid Limit, LL (%)	22	32	45
Plasticity Index, PI	5	15	24

Parameter	10 th Percentile ¹ .	50 th Percentile ¹ .	90 th Percentile ¹ .
D ₁₀ (mm)	0.0001	0.0005	0.004
D ₆₀ (mm)	0.007	0.03	0.09
Coefficient of Uniformity, Cu	10	64	225
Wet Unit Weight, kN/m ³	17	19	22
Effective Angle of Internal Friction, ϕ' (degrees)	30	32	38
Undrained Shear Strength S _u (kPa)	15	60	400
Estimated Permeability, k (m/s)	4.0x10 ⁻¹⁰	2.5 x10 ⁻⁹	1.0 x10 ⁻⁸

- 1) While for any given sample, percentages of gradation categories (e.g., gravel, sand, fines) will add to 100 per cent, percentile values do not necessarily add to 100 per cent because these statistics represent ranges of separate measurement parameters.
- 2) Baseline blow counts are based on SPT "N"-values measured for 0.3 m of penetration.
- 3) Total weight of particles as compared to the total sample weight.
- 4) "Fines" content refers to the total weight of particles passing the 0.075 mm sieve opening size as compared to the total sample weight.

4.2.2 Baseline Engineering Characteristics of Bedrock

The upper portion of the bedrock was sampled by split-spoon tools and the bedrock was confirmed by coring in Boreholes C2-1 to C2-3 and NW4-5.

4.2.2.1 Bedrock Contact

For baseline purposes the inferred bedrock contact line shown in profile on Drawing 2 shall define a surface at the elevation shown which can be projected across the tunnel excavation and the top of bedrock can vary vertically between 2 m above to 2 m below the top of bedrock defined by the inferred bedrock contact line. Boulders, rock fragments or residual soil situated above or adjacent to the parent bedrock shall not be considered bedrock for baseline purposes.

4.2.2.2 Weathering

For baseline purposes weathering of the bedrock is described as completely weathered to moderately weathered (Brown, 1981) within the first 2 m vertically below the inferred bedrock contact. The rock then improves to slightly weathered to fresh bedrock as shown on the profile on Drawing 2. For baseline purposes the top of bedrock is defined as inferred completely to moderately weathered shale bedrock which though weak, is a part of the parent rock mass due to the observed bedding structure.

4.2.2.3 Completely to Moderately Weathered Shale

Completely to moderately weathered (Brown, 1981) shale bedrock was inferred at the boreholes locations at depths ranging from 1.2 m to 1.5 m below ground surface (Elevations 99.9 m to 98.8 m) based on drilling behavior, observations of drilling cuttings and split-spoon sampling. The thickness of the completely to moderately weathered bedrock along the trenchless crossing is inferred to range from about 2.1 m to 2.5 m.

The baseline grain size distribution envelope of the soil-like component of the completely to moderately weathered shale is presented on Figure 4. The baseline envelope for Atterberg Limits is presented on Figure 5 and indicates that these materials recovered by split-spoon sampling would ordinarily be classified as generally low to medium plasticity; subject to the limitations associated with sampling and laboratory testing as described below. Baseline classification and engineering parameters for the soil-like component of the completely to moderately weathered shale are provided in Table 3 below, based on the site-specific data. Baseline SPT “N”-values provided within Table 3 are for samples where a full 457 mm of penetration could be accomplished and appropriate hammer blows summed for the last 300 mm of penetration. The baseline SPT “N”-values provided in Table 3 below are, therefore, considered representative of the soil matrix. For baseline purposes, it is expected that within the weathered rock approximately 80 per cent of all SPT “N”-values will not achieve full penetration and exhibit more than 100 hammer blows for less than 0.3 m of penetration, considered representative of driving the sampler on very dense soil, gravel, cobbles and/or boulders.

Table 3: Baseline Geotechnical Parameters for Split-Spoon Samples of Completely to Moderately Weathered Shale

Parameter	10 th Percentile ¹ .	50 th Percentile ¹ .	90 th Percentile ¹ .
SPT “N”-Value ² .	17	49	89
Water Content, w_n (%)	6	10	15
Gravel (%) ³ .	6	14	35
Sand (%) ³ .	7	14	34
Fines (%) ⁴ .	42	70	82
Plastic Limit, PL (%)	18	21	23
Liquid Limit, LL (%)	32	33	39
Plasticity Index, PI (%)	11	13	18
D_{10} (mm)	0.0005	0.0007	0.0008
D_{60} (mm)	0.007	0.018	3.16
Coefficient of Uniformity, C_u	14	25	4240
Wet Unit Weight, kN/m^3	20	23	25
Effective Angle of Internal Friction, ϕ' (degrees)	36	38	42

- 1) While for any given sample, percentages of gradation categories (e.g., gravel, sand, fines) will add to 100 per cent, percentile values do not necessarily add to 100 per cent because these statistics represent ranges of separate measurement parameters.
- 2) Baseline blow counts are based on SPT “N”-values measured for 0.3 m of penetration.
- 3) Total weight of particles as compared to the total sample weight.
- 4) “Fines” content refers to the total weight of particles passing the 0.075 mm sieve opening size as compared to the total sample weight.

The split-spoon samples obtained from within the inferred completely to moderately weathered bedrock do not contain larger fragments of rock due to the sampler size and sampling method. Larger fragments of unweathered

shale bedrock may be present in-situ. In addition, the percentage of gravel size particles may include shale fragments that either remained intact after or were broken during sampling and sample preparation. Therefore, the results of the grain size distribution testing may not be representative of the bulk grain size distribution or behavior of the in-situ or excavated completely to moderately weathered shale bedrock. It should be noted that the moderately to highly weathered rock represents a transition with layers or fragments of both intact shale and soil-like material with varying amounts of both materials. For baseline purposes the engineering properties of the soil-like component of the moderately to highly weathered shale are shown in Table 3 and the intact shale component shall be assumed to be the same as the fresh to slightly weathered shale defined below.

4.2.2.4 Slightly Weathered to Fresh Shale

The bedrock consists of shale of the Georgian Bay Formation. In general, the bedrock core samples are described as very thinly to thinly laminated to medium bedded, very fine to fine grained, faintly porous, weak, grey, shale, with slightly weathered to fresh, grey, fine grained, laminated to medium bedded, strong to very strong limestone interbeds at varying intervals of depth.

4.2.2.5 Discontinuities

Based on the structural data from the borehole logging, the rock mass structure is dominated by the near horizontal to shallow dipping bedding joints. For baseline purposes, the bedding joints shall be assumed to dip between 0 degrees and 30 degrees in any dip direction and will be continuous across the excavation. For baseline purposes the bedding within the shale bedrock will be extremely close spaced to moderately close spaced (CFEM, 2006; Table 3.9). Joint spacing for the bedding joints will vary from 0.05 m to greater than 2 m. For baseline purposes the bedding joints are planar to undulating and smooth and all of the bedding joints will be partially coated, completely coated or infilled with clay.

In addition to the bedding joints, there are two major joint sets, one of which dips at relatively steep angles (i.e. greater than 50 degrees) and another which dips shallower between 20 degrees and 30 degrees. In this report "bedding joints" is used to refer to all joints along existing bedding planes. All joints, other than bedding joints, are referred to as "joints" in this report. Average joint spacing will vary from 0.3 m to greater than 4 m.

For baseline purposes, joints will have a dip angle of 20 degrees to 90 degrees and the joint continuity will range from less than 1 m to greater than the span of the tunnel and shaft excavations. For baseline purposes, the joints are planar to undulating or irregular and smooth to rough and may be partially coated, completely coated or infilled with clay.

For comparison to the bedding joint and joint set orientation baselines, a statistically significant number of joint orientation measurements representative of the bedrock structure within the tunnel alignment are required.

4.2.2.6 Hydraulic Conductivity

Although hydraulic conductivity data was not available within the completely to moderately weathered shale bedrock, for baselining purposes the upper 2.5 m of weathered and/fractured shale bedrock will have a higher hydraulic conductivity ranging from 1×10^{-3} m/s to 1×10^{-5} m/s. Based on permeability tests carried out by MH in the monitoring wells installed by Golder, for baselining purposes the hydraulic conductivity for the slightly weathered to fresh shale bedrock ranges from 1×10^{-5} m/s to 1×10^{-7} m/s.

4.2.2.7 Intact Rock Strength Properties

Results of the uniaxial compressive strength on the shale and hard layers are contained in the Foundation Investigation Report. Considering the friable nature of the Georgian Bay shale, it was often difficult to obtain samples of the weaker shale and as such the core samples tested for Uniaxial Compressive Strength are typically biased toward the higher strength range, thus the lower limit of the baseline values have been adjusted accordingly.

The baseline values for the Uniaxial Compressive Strength (UCS) of the intact bedrock are shown in Table 4.

Table 4: Baseline Values for Unconfined Compressive Strength for Bedrock

Rock Type	Uniaxial Compressive Strength		
	10 th Percentile (MPa)	Mean (MPa)	90 th Percentile (MPa)
Shale	5	15	30
Hard Layers	80	140	200

4.2.2.8 Abrasivity

The results of Cerchar Abrasivity tests are contained in the FIR. The baseline values for the CERCHAR abrasivity are shown in Table 5.

Table 5: Baseline Values for CERCHAR Abrasivity for Bedrock

Rock Type	CERCHAR Abrasivity Baseline		
	10 th Percentile (MPa)	Mean (MPa)	90 th Percentile (MPa)
Shale	0.1	0.5	2
Hard Layers	0.5	1.5	4

4.2.2.9 Slake Durability

The results of two Slake Durability tests on samples of shale are contained in the FIR. Based on Deere and Gamble's Slake Durability Classification System (Deer & Gamble, 1971) for baseline purposes the shale can be considered as low to medium high slake durability.

4.2.2.10 Hard Layers

For baseline purposes the relative proportion of cored shale versus cored harder limestone or siltstone layers referred to as "hard layers" within the bedrock as logged in the drillholes is shown in Plate 1. The frequency is based on the percentage of hard layers in each drill run cored and does not include the completely to moderately weathered shale bedrock that was sampled in split-spoons. The data used for this assessment was from drillholes advanced at both the proposed watermain at Station 15+825 and the sanitary sewer at Station 15+850.

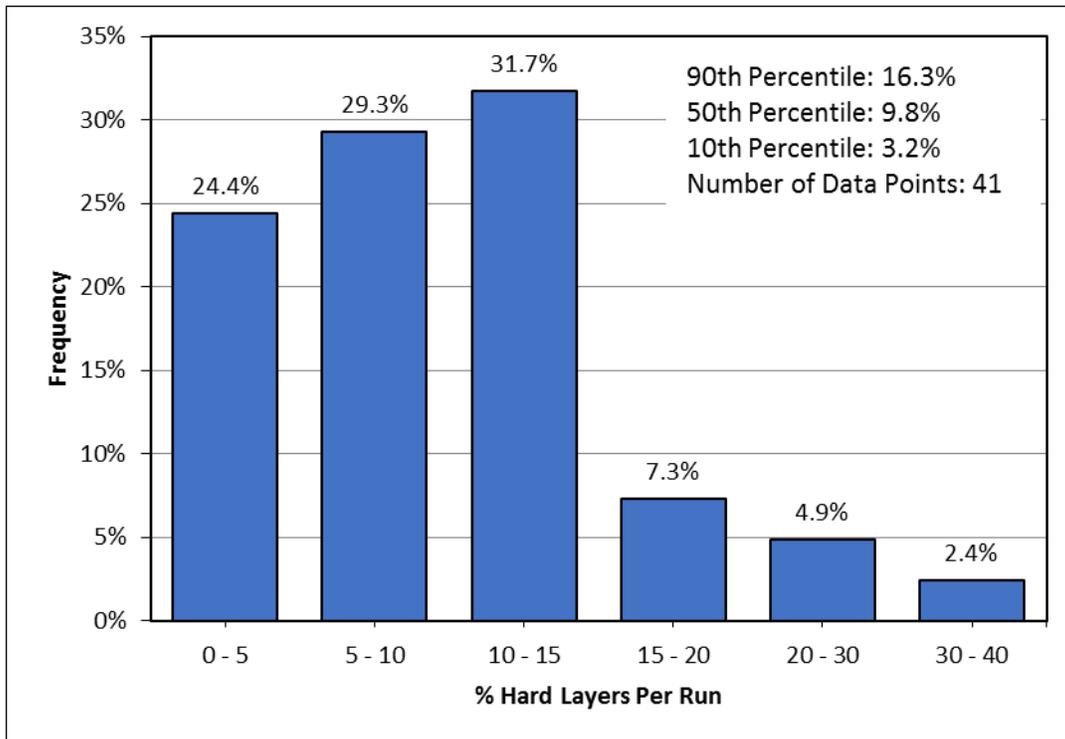


Plate 1: Baseline Distribution of the Percentage of Hard Layers

For the purposes of this baseline, the maximum thickness for these hard layers refers to a discrete interbed of pure limestone or siltstone. For baseline purposes, Plate 2 shows the frequency for the thickness of the “hard layers” that can be expected to be encountered within the bedrock.

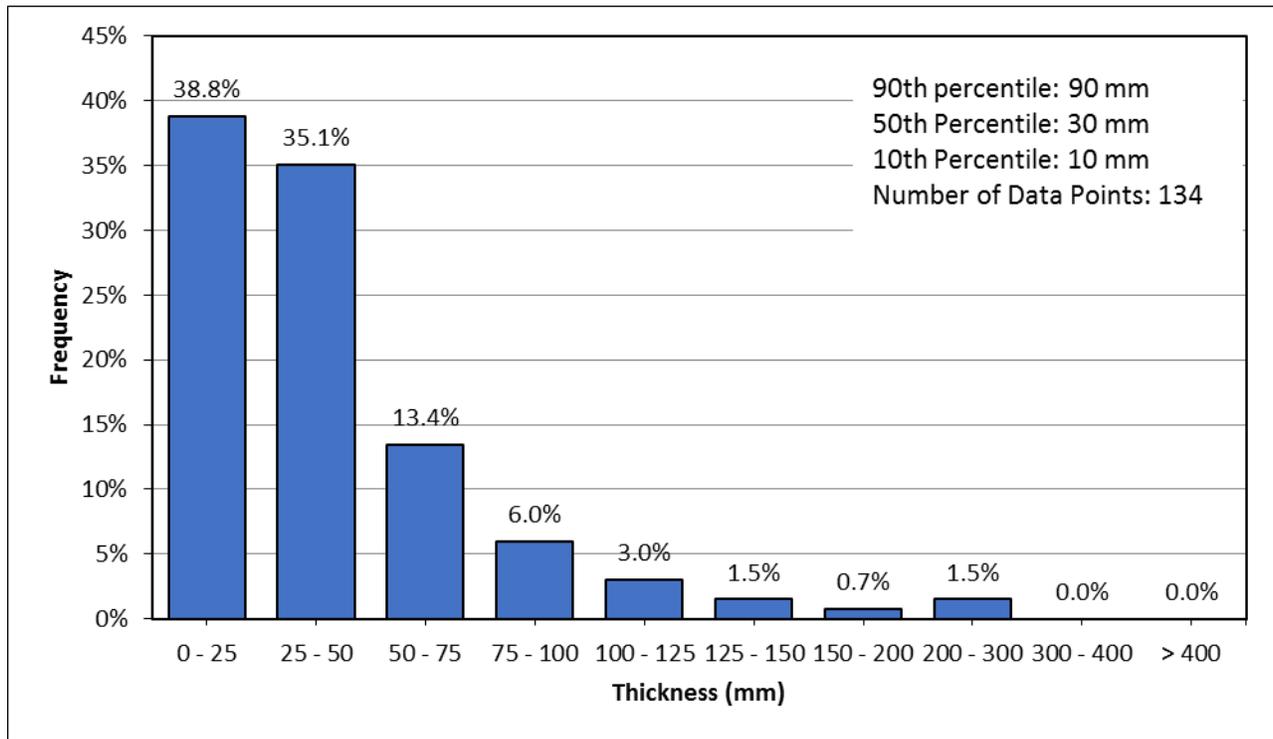


Plate 2: Baseline Distribution of the Hard Layer Thicknesses

4.2.2.11 Bedrock Quality

For baseline purposes the cored bedrock quality expressed in terms of the Rock Quality Designation (Deere, 1964) is shown below in Table 6. The bedrock quality has been baselined separately for each of the shafts, the tunnel, and the cut and cover segment of the alignment. Inferred completely to moderately weathered bedrock that was sampled in split-spoons are not included in this assessment, and for baseline purposes will range from very poor to fair quality (Table 3.10 of CFEM, 2006¹). The baselines below indicate the relative quantities expressed as a percentage of the total quantity excavated for each segment shown in the table. The baselines are therefore applicable for the entire segment of the work listed in the table (i.e. each shaft, the tunnel and the open cut segments). For comparison to the baselines given in Table 6, the entire segment must be excavated and the rock quality for the entire segment compared to the baseline percentages given in the table. For baseline purposes the percentage for each RQD class shown in Table 6 can vary by +/- 5% from the values shown in the table.

Table 6: Rock Quality Designation in Bedrock

Baselined Segment	Very Poor (%) RQD 0 - 25	Poor (%) RQD 25 - 50	Fair (%) RQD 50 - 75	Good (%) RQD 75 - 90	Very Good (%) RQD 90 - 100
Entry Shaft	0	0	0	73	27
Tunnel	0	0	20	22	52

¹ Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual (CFEM), 4th Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.

Baselined Segment	Very Poor (%) RQD 0 - 25	Poor (%) RQD 25 - 50	Fair (%) RQD 50 - 75	Good (%) RQD 75 - 90	Very Good (%) RQD 90 - 100
Exit Shaft	0	0	47	0	53
Cut and Cover	0	0	20	51	29

4.2.2.12 Time Dependent Deformation

The time dependant deformation (swelling) characteristics of the Georgian Bay shale will need to be taken into consideration in the design of the underground structures located within this rock formation including tunnel and shaft linings. For any structures such as the tunnel lining which are directly in contact with the rock, the time-dependent deformation (swelling) of the rock will cause pressure to build up with time at the rock-structure interface. The magnitude of the pressure will depend on the rigidity of the structure, the timing of the construction, the swelling characteristics of the bedrock and the initial in situ stresses in the rock formation. Analyses should be carried out to assess the impact of the swelling of the shale on the design of the precast segmental lining and shaft linings. Depending on the results it may be necessary to incorporate a compressible grout between the precast segments and the bedrock.

For baseline purposes, the swelling potentials for horizontal free swell rates will range from 0.05% to 0.2% per log cycle of time and vertical free swell rates will range from 0.5% to 2.0% per log cycle of time, with a baseline suppression pressure of between 0.5 and 0.8 MPa to 1 MPa to 2 MPa in the horizontal and vertical direction, respectively. The lining should be checked for all loading cases including different combinations of the horizontal and vertical swelling which may result in higher forces and moments.

4.2.2.13 In Situ Stresses in the Bedrock

For baseline purposes the in situ stresses are shown in Table 7.

Table 7: Baseline Values for In Situ Secondary Principal Stresses

	Maximum Vertical Secondary Principal Stress	Locked-In Stress near Bedrock Surface	Horizontal to Vertical Ratio of Secondary Principal Stresses
Baseline Value	Equal to the lithostatic stress	1 MPa to 3 MPa	1 MPa to 1.5 MPa

4.2.3 Natural Gas

Methane and hydrogen sulphide are known to be present in the shale bedrock of the Georgian Bay formation and have been encountered in the soils of Southern Ontario, typically in granular layers capped by cohesive tills.

Methane forms an explosive mixture with air and is a potential hazard for excavation and construction work and it should be assumed that it will be encountered in the bedrock and soil at this site. Changes in groundwater pressure which may be caused by dewatering or seepage into excavations/underground spaces can lead to migration/release of gaseous or dissolved methane. Therefore, the absence of methane in a particular area should not be construed to indicate that there is no risk of the presence of methane in the future or in other site areas.

The tunnel should be considered, according to the OSHA Underground Construction (Tunneling) Regulations (29 CFR Part 1926.800, "Tunnels and Shafts.") as "potentially gassy".

For baseline purposes it should be assumed that methane gas could be encountered anywhere along the tunnel alignment or in the shafts. For baseline purposes the Contractor should expect to encounter gas in numerous small pockets that can be vented within 2 hours and up to a total of 4 hours in a 24 period.

As a consequence of this designation, the Specifications include specific provisions for gas monitoring and specific requirements for selected underground equipment.

4.2.4 Groundwater Conditions

Water levels were observed in the open boreholes upon completion of drilling operations and were between 4.6 m and 5.7 m below ground surface (between Elevations 96.4 m and 90.9 m) in the boreholes. However, the water level observed in the open boreholes during and/or upon completion of drilling does not necessarily represent the longer-term, stabilized groundwater level at the site. Standpipe piezometers were installed in bedrock in Boreholes C1-4, C2-1 and C3-3 and measured groundwater levels in the standpipe piezometers are summarized in Table 8:

Table 8: Recorded Groundwater Levels in Standpipe Piezometer

Borehole No.	Depth to Water Level (m)	Groundwater Elevation (m)	Date of Measurement
C1-4	4.6	95.8	March 19, 2019
C2-1	4.7	96.4	March 13, 2019
	5.7	95.4	March 21, 2019
C3-3	4.8	90.9	February 20, 2019
	4.7	91.0	March 13, 2019
	4.7	91.0	March 21, 2019

A baseline groundwater level is provided on Drawing 2. For baseline purposes, groundwater levels are to be expected to fluctuate seasonally in response to changes in precipitation and snow melt to as much as 1 m above or below the baseline groundwater level. Any water infiltrating through the roadway and embankment fill is also expected to be inhibited by and therefore perched within the Fill overlying the Cohesive Deposits.

4.3 Cobbles, Boulders and Other Obstructions

The date of construction of the original QEW and South Sheridan Way fill placement is unknown; as well, typical construction practices with regards to clearing and grubbing of the site prior to fill placement at the time of construction are unknown. No records of construction of the highway or local roadways (site records/journals, photographs, as-constructed drawings, etc.) were available at the time of writing this SCBR. For baseline purposes it is to be expected that where construction penetrates fill materials there will be debris consisting of broken concrete, reinforcing bars, logs, stumps and brush from previous clearing and grubbing operations and cobbles and boulders buried in the fill. The contractor must select a construction method that is capable of removing these types of obstructions in fill materials.

5.0 EXISTING UTILITIES

Utilities present along the project alignment, including but not limited to lighting, communications cables, storm sewers, and natural gas pipelines, must be accurately located and either protected or relocated. Depending upon the location, utilities relocated to avoid the trenchless construction could be affected by the settlement trough created by excavations required for trenchless construction. The Contractor is responsible for protecting existing and newly relocated utilities from settlement and horizontal displacement. Protection of utilities, support of excavations, instrumentation and monitoring have been specified elsewhere in the Contract Documents to control, measure and document the amount of displacement at these sites.

For baseline purposes, it is to be expected that utilities will be bedded in and backfilled with Class A Granular Fill materials. These materials will conduct water and where these are within surrounding lower permeability soils or rock and/or below the baseline piezometric levels they will be saturated.

6.0 CLASSIFICATION OF ANTICIPATED GROUND BEHAVIOUR

This section of the SCBR describes the Engineering Classes of the various soil types as identified along the proposed primary liner alignment, and their anticipated behaviour if exposed and unsupported. The interpreted baseline stratigraphy and the baseline piezometric level along the sanitary sewer alignment are shown on Drawing 2.

The anticipated ground behaviour presented in this report is described using the Ground Behaviour Classification System provided below in Table 9. The Tunnelman's Ground Classification System (Heuer, 1974), as derived from the original system by Terzaghi (1950), has been used as a basis to describe the anticipated behaviour of the ground. No account is taken in the given classifications of the supporting pressure provided to the face by tunneling equipment and fluids or to the response of the ground to support or modifications that are selected and implemented by the Contractor (e.g., dewatering, shoring, tunneling systems, etc.); the intent of using the Engineering Classes is to describe the behaviour of the material if exposed during excavation and tunneling without provision of support or ground modifications.

Table 9: Ground Behaviour Classification

Classification and Descriptive Terms	Sub-Classification	Behaviour
Firm		Excavation face(s) can be cut without initial support
Ravel, Raveling	Slow raveling	Chunks or flakes of material begin to drop out of the excavation face(s) sometime after the ground has been exposed, due to loosening, overstress, fissures, and "brittle" fracture (ground separates or breaks along distinct surfaces, as opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes; otherwise the ground is slow raveling.
	Fast raveling	
Squeeze, Squeezing		Ground squeezes or extrudes plastically from excavation face(s) without visible fracturing or loss of continuity, without perceptible increase in water content, and exhibits ductile plastic yield and flow.

Classification and Descriptive Terms	Sub-Classification	Behaviour
Run, Running	Cohesive-running	Apparent cohesion in moist sand, silt, or mixtures of these, or weak cementation in any non-cohesive soil, allows the material to stand for a brief period of raveling, before it breaks down and degrades to running or flowing behaviour.
	Running	Dry non-cohesive materials without cohesion are unstable at a slope greater than their angle of repose (approximately 30 to 35 degrees). When exposed at steeper slopes, the soils run like granulated sugar or dune sand until the slope flattens to the angle of repose. Soil exhibiting such behaviour is running.
Flow, Flowing		A mixture of soil and water flows from excavation face(s) like a viscous fluid. The material can flow for great distances, completely filling excavations or tunnels in some cases.

In granular soils, face stability is commonly assessed using groundwater conditions, soil gradation, variability in gradation and in situ density. The “fines content” (combined silt and clay-size fraction of soil) for the Class A and B soils is described in Section 4.2. While the fines content is conventionally useful for assisting with interpretation of soil behaviour, in the Greater Toronto Area granular soils can commonly include a significant “fines” content and yet also run, ravel or flow (depending on water content) contrary to conventional interpretations of likely behaviour because the “fines” can consist primarily of relatively uniformly graded silt. Therefore, the baseline behaviour descriptions and classifications provided in this report have been developed specifically for this project.

Excavation through the Cohesive Deposits (Class B) will also encounter water-bearing granular layers that will flow upon initial exposure where these are below the baseline groundwater level. Excavation difficulties such as lumping, balling and sticking to equipment are to be expected where zones/lenses with higher silt and clay contents are encountered within the Cohesive Deposits (Class B).

In general, the shale bedrock will have a relatively short stand-up time (several hours) if not supported immediately behind the cutter head. Overstressing of the rock will occur in the tunnel crown and invert due to the high horizontal in situ stresses. The overstressing will manifest itself as shear slip along bedding planes, as and crushing and tensile failure through the intact shale.

If not retained in place by the tunnelling shields and timely installation of proper support, the overstressed rock material will delaminate along bedding planes and will loosen and fallout resulting in overbreak beyond the original excavation lines.

7.0 CLOSURE

This Subsurface Conditions Baseline Report was prepared by Golder Associates Ltd., with input and consultation by the project designer, Morrison Hershfield Limited, on behalf of the Ministry of Transportation, Ontario for the proposed sanitary sewer installation located under the QEW at about Station 15+850, in Mississauga, Ontario. It is intended for use by bidders of MTO Contract 2019-2016.

Golder Associates Ltd.

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AB/MK/MJT/SMM/SJB/rb

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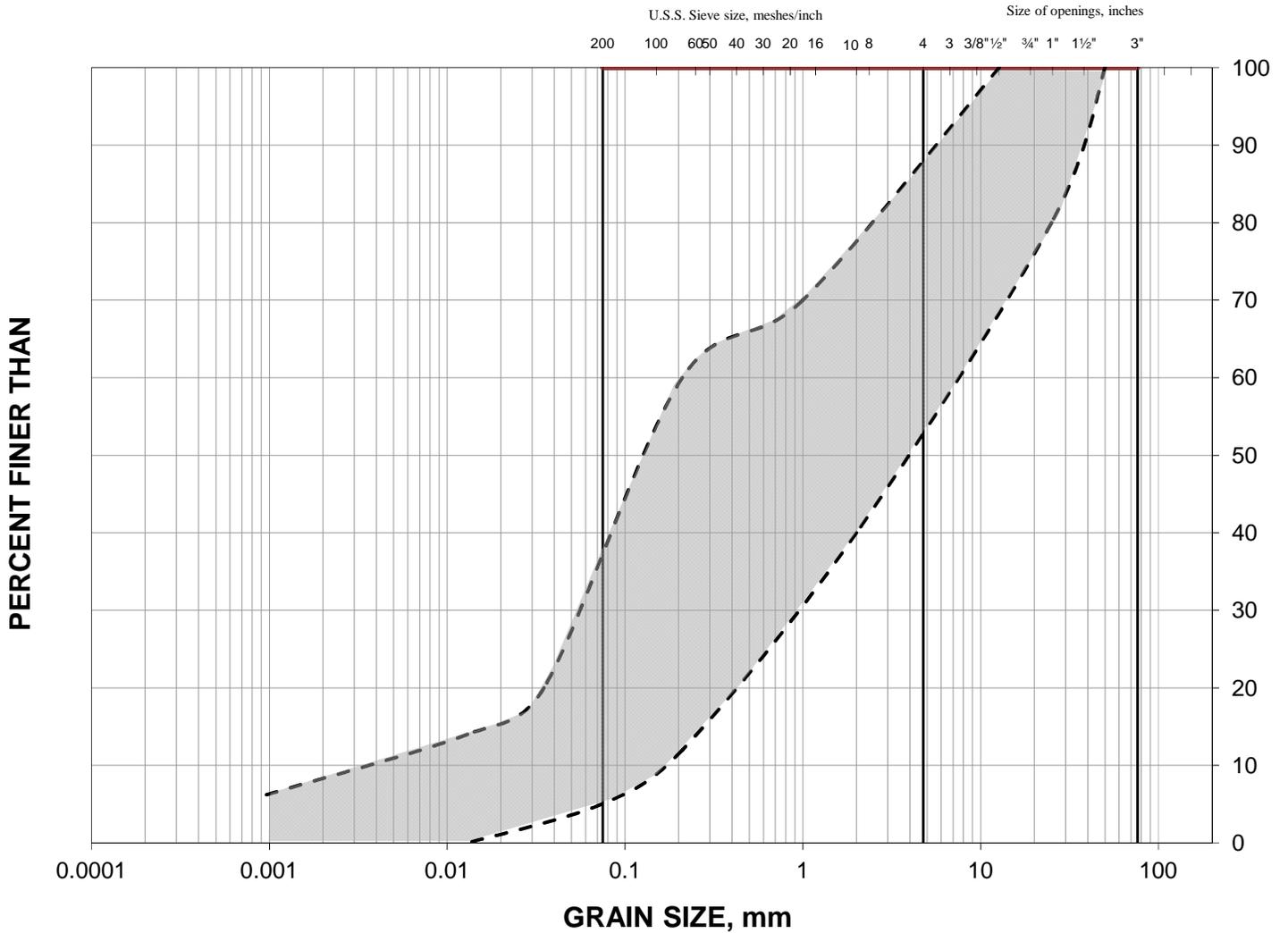
Table A: Description of Engineering Soil Classes

Engineering Soil Class	Colour Code	Soil Type Description	Description of Engineering Class	Major Deposits	Behaviour of Engineering Classes
A		<ul style="list-style-type: none"> ■ Clayey Silt ■ Construction and demolition debris 	<ul style="list-style-type: none"> ■ Near-surface materials placed by man-made processes, with random and broad compositions ■ Fill can include natural and man-made materials related to highway/roadway embankment construction, containing varying fractions of gravel silt, sand and clay, along with organic material and other debris. 	<ul style="list-style-type: none"> ■ Fill 	<ul style="list-style-type: none"> ■ Above groundwater levels, firm. ■ Below groundwater levels, slow raveling. ■ Groundwater flows from and within coarser layers will decrease following exposure. ■ Material behaviour in exposed areas will be sensitive to variation in water content and construction traffic. ■ Sand and gravel components of material are abrasive.
B		<ul style="list-style-type: none"> ■ Clayey Silt ■ Silty Clay 	<ul style="list-style-type: none"> ■ Well graded, low to medium plasticity silts and clays, trace to some sand, some gravel. ■ The plasticity index of this soil class will range between about 5 per cent and 24 per cent. 	<ul style="list-style-type: none"> ■ Cohesive Deposit 	<ul style="list-style-type: none"> ■ Above groundwater levels, firm. ■ Below groundwater levels, firm to slow raveling. ■ Material behaviour in exposed areas will be sensitive to variation in water content and construction traffic. ■ Sand and gravel components of material are abrasive.

Figures

**Baseline Grain Size Distribution
Gravelly Sand to Sand and Gravel (Fill)
(Class A)**

FIGURE 1



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		

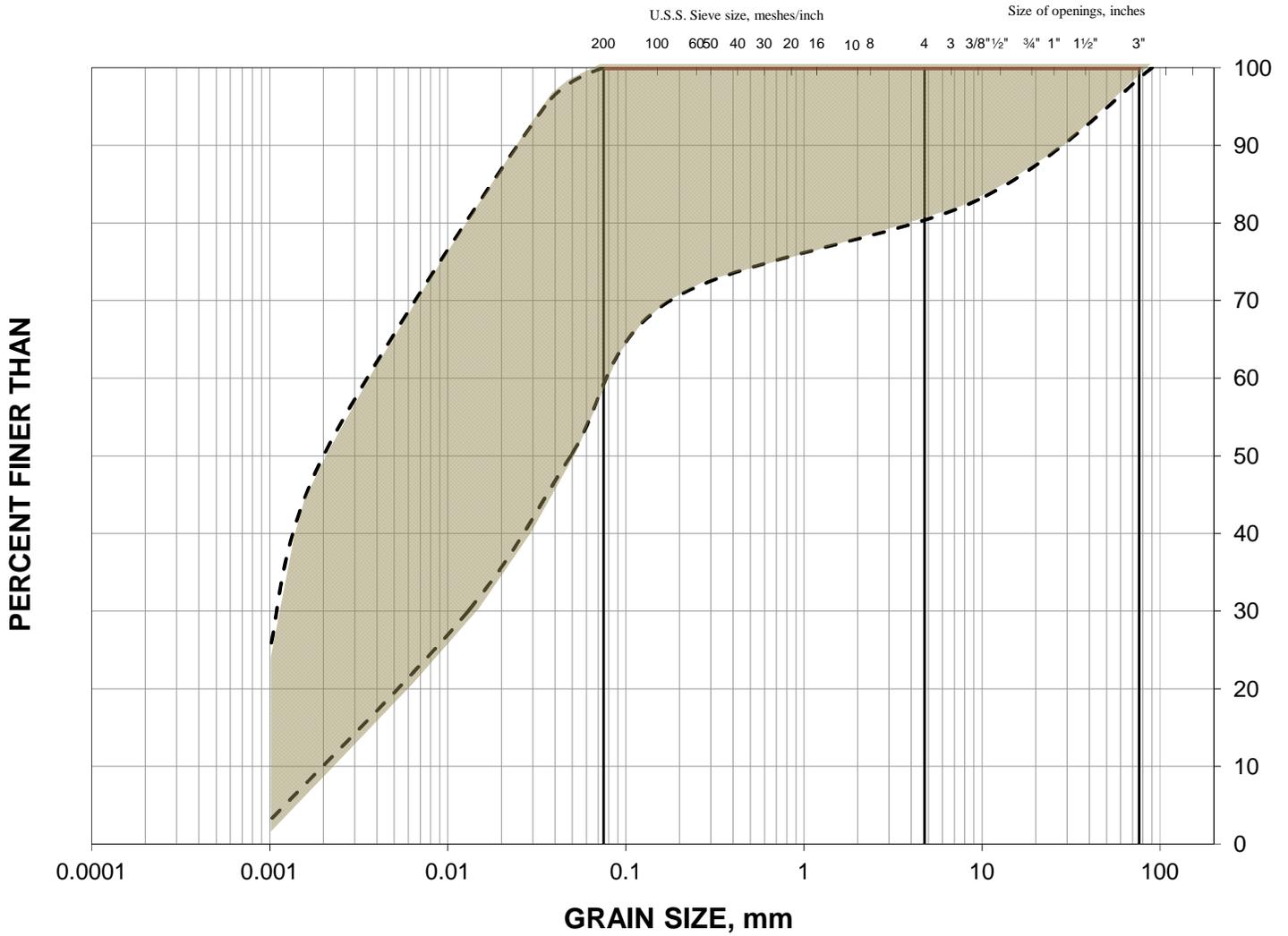
Date: April, 2019
Project: 1662333

Golder Associates

Drawn: MWK
Checked: SM

**Baseline Grain Size Distribution
Cohesive Deposits
(Class B)**

FIGURE 2

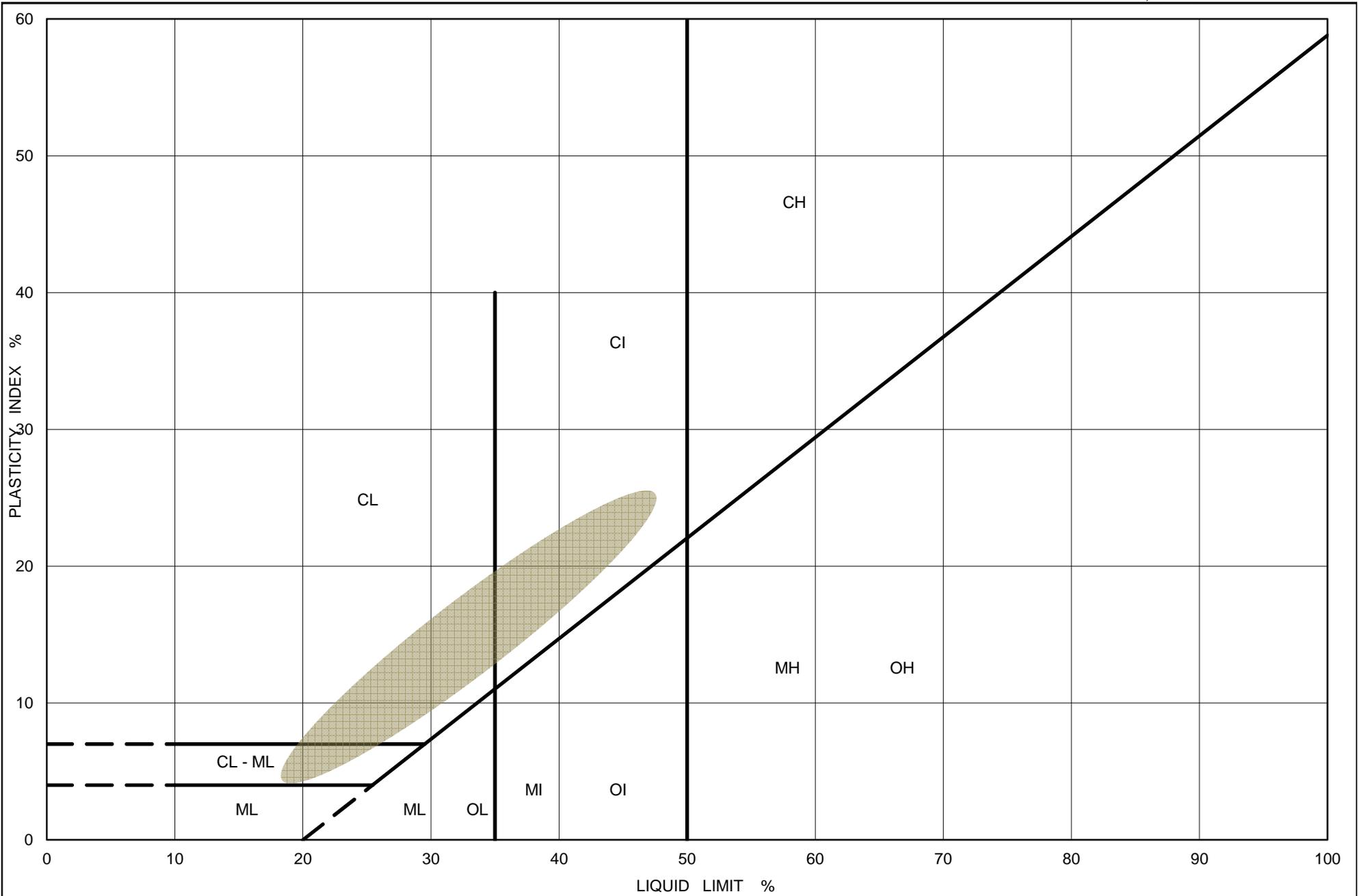


SILT AND CLAY SIZES		FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED		SAND SIZE			GRAVEL SIZE		

Date: April, 2019
Project: 1662333

Golder Associates

Drawn: MWK
Checked: SM



Ministry of Transportation

PLASTICITY CHART Cohesive Deposits (Class B)

Ontario

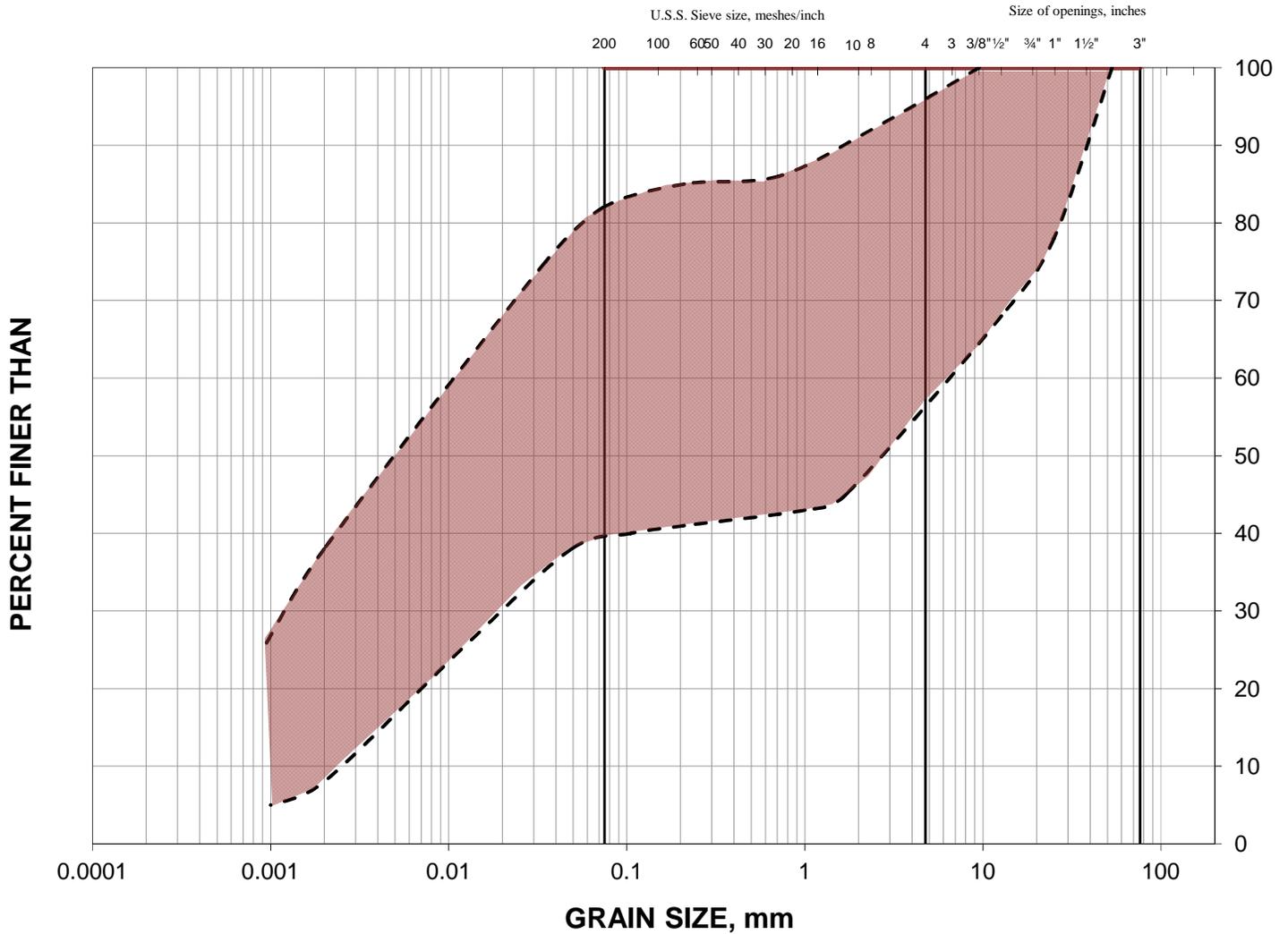
Figure No. 3

Project No. 1662333

Checked By: SM

**Baseline Grain Size Distribution
Completely to Moderately Weathered
Bedrock (Class C)**

FIGURE 4

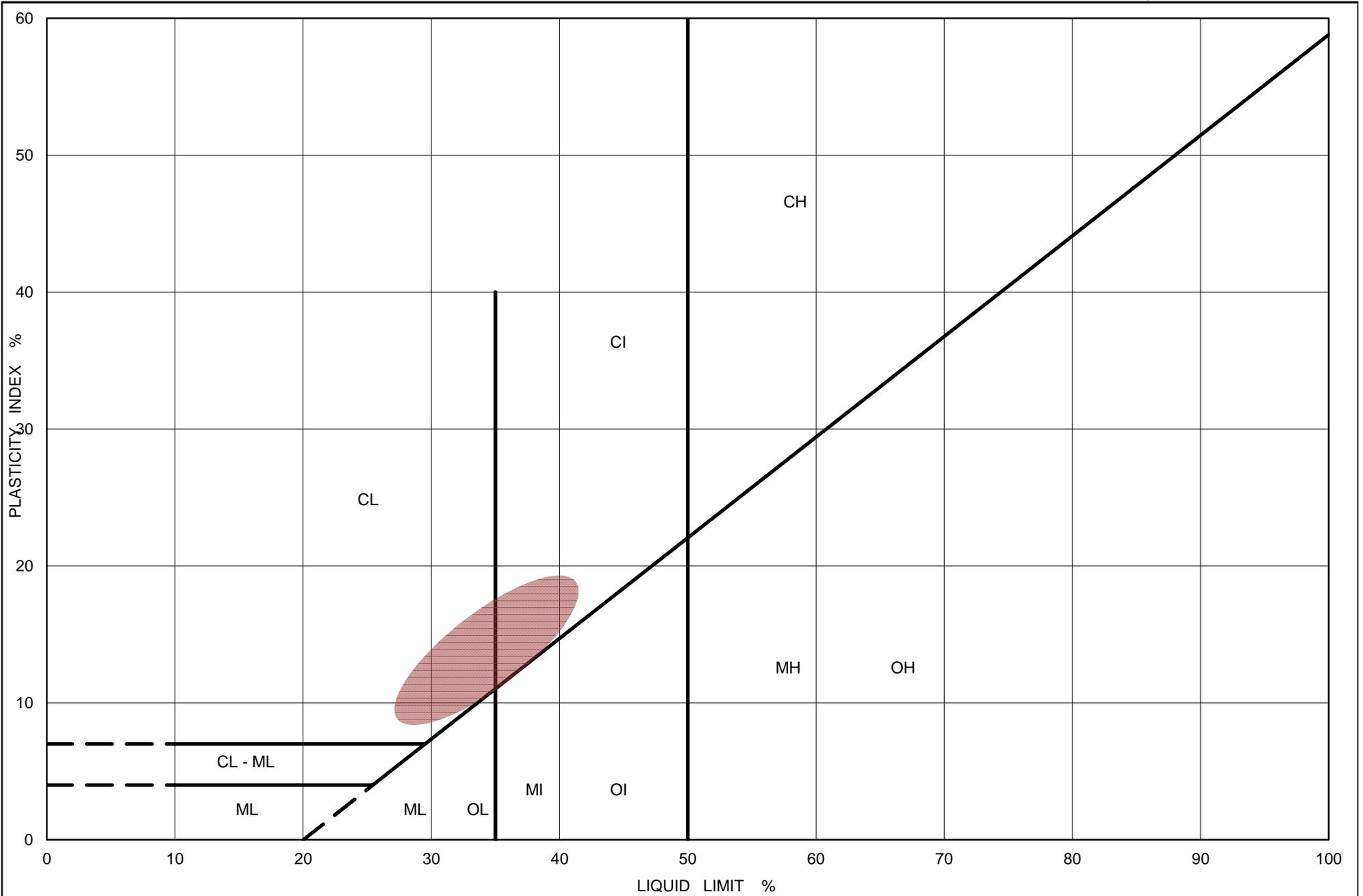


SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		

Date: April, 2019
Project: 1662333

Golder Associates

Drawn: MWK
Checked: SM



Ministry of Transportation

Ontario

PLASTICITY CHART

Completely to Moderately Weathered Bedrock

Figure No. 5

Project No. 1662333

Checked By: SM

Drawings

METRIC
 DIMENSIONS ARE IN METRES AND/OR
 MILLIMETRES UNLESS OTHERWISE SHOWN.
 STATIONS IN KILOMETRES + METRES.

CONT No. WP No. 2002-13-00



QEW WIDENING - MISSISSAUGA RD TO HURONTARIO ST
 SANITARY SEWER INSTALLATION STATION 15+850

SHEET

BOREHOLE LOCATIONS



KEY PLAN
 SCALE 1:50,000
 0 2 4 km

LEGEND

- Borehole
- ⊥ Seal
- ⊏ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL in piezometer March 21, 2019
- ≡ WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
C1-4	100.4	4823329.3	295310.9
C2-1	101.1	4823392.0	295230.5
C2-2	101.1	4823375.4	295283.7
C2-3	100.8	4823357.4	295318.2
NW4-5	100.3	4823340.0	295333.6



NOTES

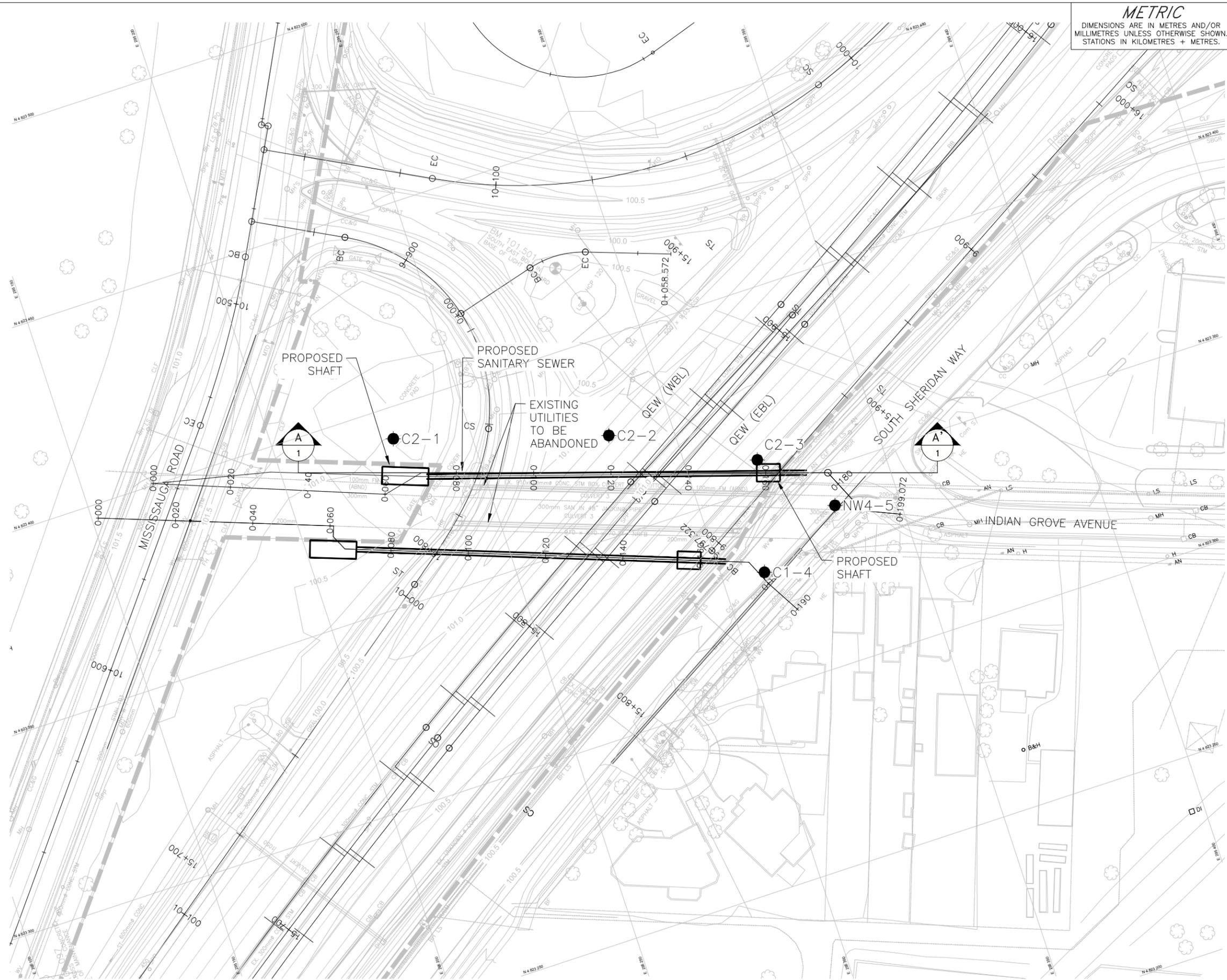
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

REFERENCE

Base plans provided in digital format by MH, drawing file nos. X11609340Base.dwg, X-Final Merged Util.dwg, X-PROP-UTIL.dwg, Existing Property.dwg, 11609340 - QEW Prop Util-Dickson & Lynchmere - C3D 2017.dwg, 11609340 - QEW Prop Util-Stevebank Rd - C3D 2017.dwg, 11609340 - QEW Prop Util-Knareswood Dr - C3D 2017.dwg, and x1160934_Align.dwg, received March 25, 2019 and 11609340 - QEW Prop Util-IndianGroveAve - C3D 2017.dwg, received June 21, 2019.

NO.	DATE	BY	REVISION

Geocres No. 30M12-456		PROJECT NO. 1662333		DIST. CENTRAL	
HWY. QEW	CHKD. MWK	DATE: 07/30/2019	SITE:		
SUBM'D. AB/EJ	CHKD. SMM	APPD. SJB	DWG. 1		



PLAN SCALE



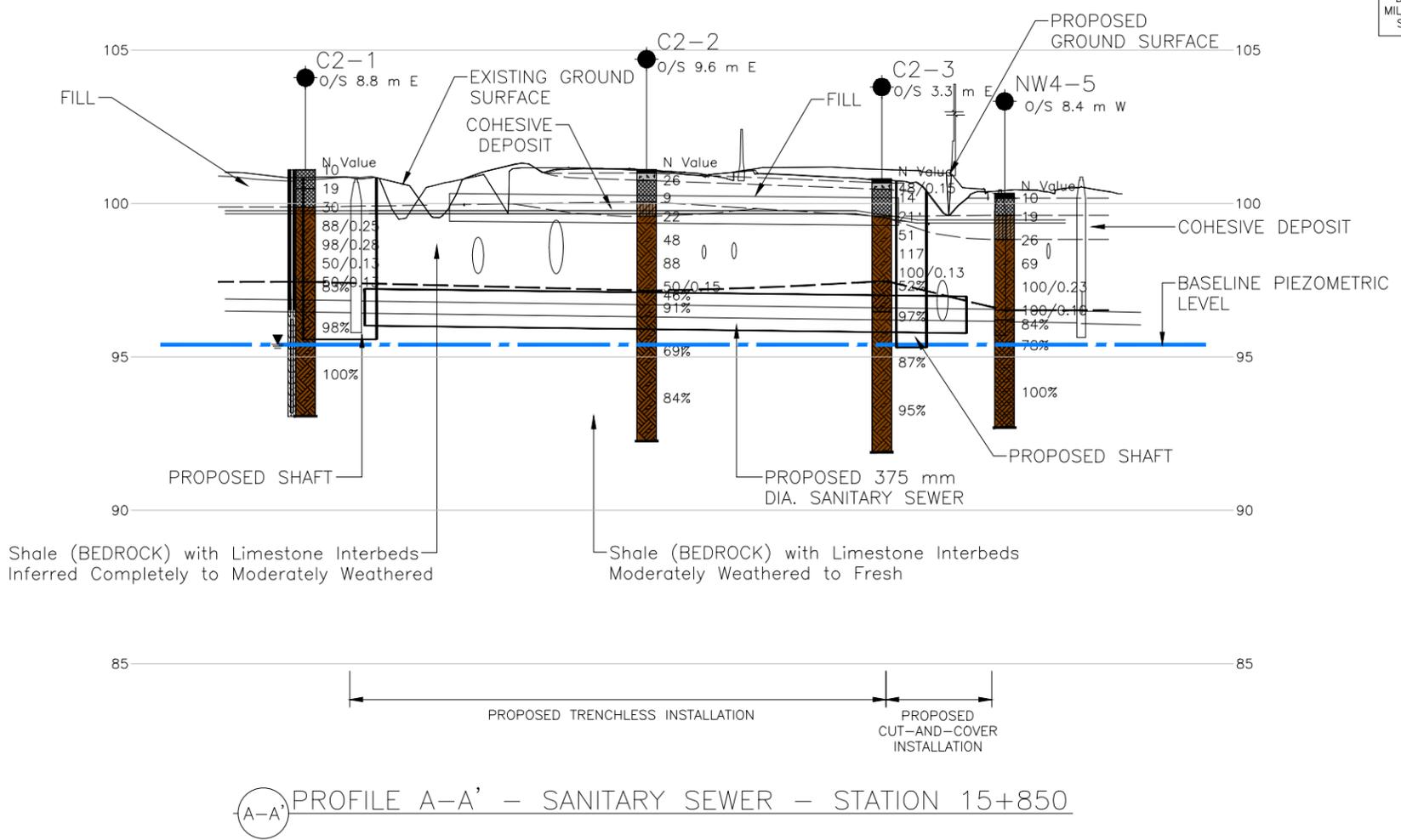
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METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. WP No. 2002-13-00
QEW WIDENING - MISSISSAUGA RD TO HURONTARIO ST
SANITARY SEWER INSTALLATION STATION 15+850
BASELINE STRATIGRAPHIC PROFILE



KEY PLAN
SCALE 1:2000
2 0 2 4 km



PROFILE A-A' - SANITARY SEWER - STATION 15+850

LEGEND

- Borehole
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer March 21, 2019
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
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C2-1	101.1	4823392.0	295230.5
C2-2	101.1	4823375.4	295283.7
C2-3	100.8	4823357.4	295318.2
NW4-5	100.3	4823340.0	295333.6

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

This drawing is to be read in conjunction with the report titled "Subsurface Conditions Baseline Report, Sanitary Sewer Installation Station 15+850, QEW Widening From West of Mississauga Road to West of Hurontario Street, Mississauga, Ontario, Ministry of Transportation Ontario, GWP 2002-13-00, Dated May X, 2019.

This interpreted stratigraphy is a simplification of the subsurface conditions. Detailed descriptions of the conditions encountered at the borehole locations are found on the records of boreholes in the Foundation Investigation Report referenced in this report.

Major soil deposit and rock formations are delineated by the boundary line identified above. The boundary established represents the baseline ground conditions; however, variation in the boundaries from those illustrated must be anticipated both parallel and perpendicular to the section line.

The characteristics and variability anticipated with the major soil deposits are described in the text of this report. Significant layers, interlayers, and lenses within the major deposits are illustrated where identified on the borehole logs. The boundaries so illustrated are intended to highlight the variability within the deposits that will exhibit gradual transitions from one soil type to another. In addition, lenses and interlayers not detected by the subsurface investigation will be present between boreholes.

Construction equipment and procedures must be selected to accommodate variation in the deposit boundaries as well as variations within the deposits as described in the report text. Where precise determination of deposit boundaries and deposit variability are critical for safety and stability they should be verified by investigation during design and construction.

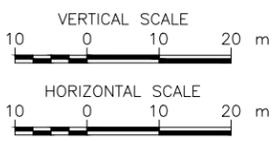
The ground surface profile and plan and profile of proposed construction are approximate and shown for illustrative purposes only. Refer to Contract Drawings for dimensions and limits of the work.

Borehole width in profile is not to scale.

For baseline purposes, the stratigraphic boundary along the tunnel alignment between overburden and the bedrock can vary vertically by +/- 2 m.

For baseline purposes, the stratigraphic boundary along the tunnel alignment between moderately to completely weathered bedrock and the fresh to slightly weathered bedrock can vary vertically by +/- 3 m.

Class A	FILL	Fill
Class B	COHESIVE DEPOSIT	Organics/Topsoil
Class C	SHALE BEDROCK	Clayey Silt to Silty Clay
		Shale (BEDROCK)
		Baseline Major Soil Deposit or Bedrock Boundary
		Interpreted Piezometric Water Level



NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

REFERENCE

Base plans provided in digital format by MH, drawing file nos. X11609340Base.dwg, X-Final Merged Util.dwg, X-PROP-UTIL.dwg, Existing Property.dwg, 11609340 - QEW Prop Util-Dickson & Lynchmere - C3D 2017.dwg, 11609340 - QEW Prop Util-Stavebank Rd - C3D 2017.dwg, 11609340 - QEW Prop Util-Knareswood Dr - C3D 2017.dwg, and x1160934_Align.dwg, received March 25, 2019 and 11609340 - QEW Prop Util-IndianGroveAve - C3D 2017.dwg, received June 21, 2019.

NO.	DATE	BY	REVISION

Geocres No. 30M12-456

HWY. QEW	PROJECT NO. 1662333	DIST. CENTRAL
SUBM'D. AB/EJ	CHKD. MWK	DATE: 07/30/2019
DRAWN: SW	CHKD. SMM	APPD. SJB
		DWG. 2



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