



# Foundation Investigation and Design Report

*Glanworth Drive Underpass, Site 19-406*

*Highways 401, 4 and 21 Structural Replacements*

*GWP 3030-11-00, Assignment No. 1 (3011-E-0046)*

*Ministry of Transportation, Ontario - West Region*

Submitted to:

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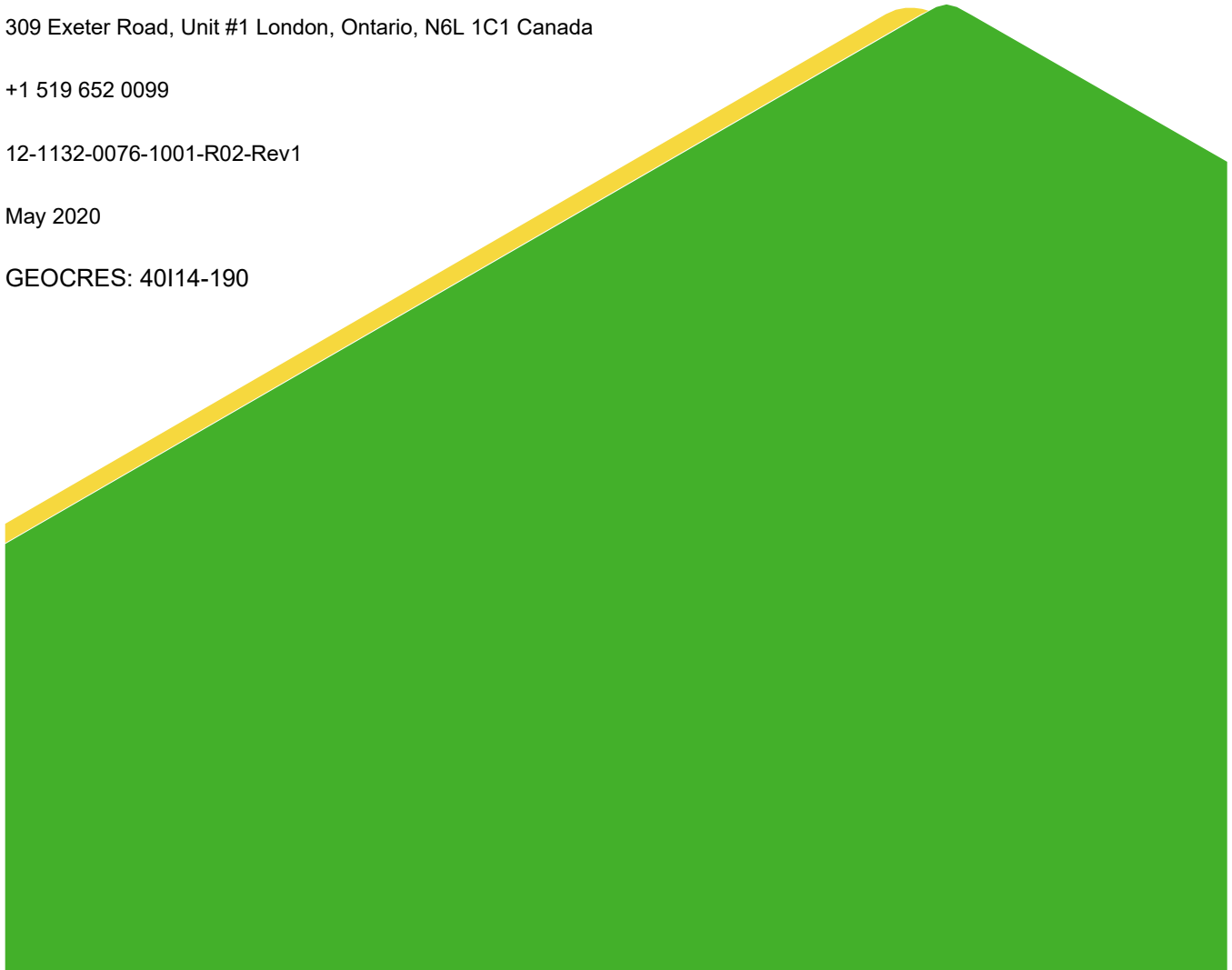
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# Table of Contents

## PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

<b>1.0 INTRODUCTION .....</b>	<b>1</b>
<b>2.0 SITE DESCRIPTION .....</b>	<b>1</b>
2.1 General.....	1
2.2 Site Geology.....	1
<b>3.0 INVESTIGATION PROCEDURES .....</b>	<b>2</b>
<b>4.0 SUBSURFACE CONDITIONS.....</b>	<b>3</b>
4.1 General.....	3
4.2 Soil Conditions .....	3
4.2.1 Topsoil.....	3
4.2.2 Pavement Structure and Fill.....	3
4.2.3 Silty Clay Including Interlayers.....	3
4.2.4 Lower Clayey Silt .....	4
4.3 Groundwater Conditions .....	4
<b>5.0 MISCELLANEOUS .....</b>	<b>5</b>

## PART B – PRELIMINARY FOUNDATION DESIGN REPORT

<b>6.0 PRELIMINARY FOUNDATION ENGINEERING RECOMMENDATIONS .....</b>	<b>6</b>
6.1 Bridge Foundations .....	6
6.1.1 Shallow Foundations.....	6
6.1.2 Deep Foundations.....	7
6.2 Liquefaction Potential and Seismic Analysis.....	12
6.2.1 Seismic Parameters .....	12
6.2.2 Seismic Hazard Assessment .....	12
6.3 Lateral Earth Pressures for Design.....	13
6.4 Construction Considerations .....	14
6.4.1 Shallow Foundations.....	14

6.4.2	Deep Foundations.....	14
6.5	Embankments .....	14
6.6	Excavations and Temporary Cut Slopes.....	16
6.6.1	Temporary Protection Systems.....	16
<b>7.0</b>	<b>RECOMMENDATIONS FOR DETAIL DESIGN .....</b>	<b>17</b>
<b>8.0</b>	<b>MISCELLANEOUS .....</b>	<b>18</b>

TABLE I – Comparison of Foundation Alternatives

LIST OF ABBREVIATIONS

LIST OF SYMBOLS

RECORD OF BOREHOLE SHEETS

DRAWING 1 - Borehole Locations

DRAWING 2 - Soil Strata

## **APPENDICES**

### **APPENDIX A**

Laboratory Testing Results

**PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT**

**Glanworth Drive Underpass, Site 19-406**

**Highways 401, 4 and 21 Structural Replacements**

**GWP 3030-11-00, Assignment No. 1 (3011-E-0046)**

**Ministry of Transportation, Ontario – West Region**

## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a preliminary foundation investigation for the proposed Highway 401/Glanworth Drive Underpass (Site 19-406) west of London, Ontario as part of preliminary design for GWP 3030-11-00, Assignment No. 3011-E-0046.

The purpose of the work described in this report was to explore the subsurface conditions at the location of the proposed new structure (which is replacing the existing Glanworth Drive structure, Site 19-406, located about 440 metres to the west) by drilling a limited number of boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are provided in the MTO's Request for Proposal, Golder's proposal P12-1132-0076-P01 dated September 10, 2012 and Golder's subsequent change order letters. The work was carried out in accordance with Golder's Quality Control Plan for Foundation Engineering dated November 2012.

Dillon provided Golder with preliminary drawings for this project in digital format.

## 2.0 SITE DESCRIPTION

### 2.1 General

The project is located, as shown on the Key Plan, Figure 1, in the southwest portion of the City of London. This section of Highway 401 is currently a four-lane divided highway oriented generally northeast-southwest. The area adjacent to the site consists of relatively flat-lying agricultural and commercial lands.

For the purposes of this report, Highway 401 and Glanworth Road are assumed to be oriented east-west and north-south, respectively.

### 2.2 Site Geology

This project lies within the physiographic region of southern Ontario known as the Westminster Moraine. The physiographic mapping indicates that the Glanworth Drive Underpass site is situated on a till moraine.<sup>1</sup> The available surficial geology mapping indicates that lacustrine silt and sand and clayey silt to silty clay till are present at the site.<sup>2</sup>

Bedrock in the area of the site is described as medium brown, microcrystalline limestone of the Dundee Formation which belongs to the Hamilton Group of Middle Devonian Age.<sup>3</sup> The bedrock surface is estimated, based on the available mapping, to be at about Elevation 160 metres (m) or some 90 m below ground surface.

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<sup>1</sup> Chapman, L.J. and Putnam, D.F., 1984: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2.

<sup>2</sup> Dreimanis, A., 1963: Pleistocene Geology of the St. Thomas Area (West Half), Southern Ontario. Ontario Department of Mines, Preliminary Geological Map 238, scale 1:50,000.

<sup>3</sup> Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.

### 3.0 INVESTIGATION PROCEDURES

Field work was carried out between January 9 and March 29, 2019, during which time five boreholes, identified as BH-501 to BH-505, were drilled for the proposed new structure. The Record of Borehole sheets are attached to this report and the results of laboratory testing are provided in Appendix A.

The approximate locations of the boreholes are shown on the Borehole Location Plan, Drawing 1. The table below summarizes the approximate borehole location coordinates, ground surface elevations at the borehole locations and borehole depths.

Borehole	Approximate Location		Ground Surface Elevation (m)	Depth (m)
	Northing (m)	Easting (m)		
BH-501	4,747,544.1	405,419.5	261.2	56.4
BH-502	4,747,549.9	405,384.4	263.6	30.5
BH-503	4,747,555.5	405,350.2	261.0	55.9
BH-504	4,747,539.4	405,428.3	261.0	10.4
BH-505	4,747,560.1	405,338.4	261.7	9.6

Field work was carried out using all-terrain drilling equipment supplied and operated by a specialist drilling contractor. In the boreholes, samples of the overburden were obtained at generally 0.76 to 1.5 m intervals of depth using 50 millimetre (mm) outside diameter split spoon sampling equipment in accordance with ASTM D1586 using an automatic hammer. The results of the Standard Penetration Testing (SPT), as presented on the Record of Borehole sheets and in Section 4, are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.).

The samplers limit the maximum particle size that can be sampled and tested to about 40 mm. Therefore, particles that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes, including cobbles and boulders, are known to be present in the native soils, as discussed in the text of this report.

Boreholes were terminated between 9.6 m and 56.4 m below the existing ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations. Upon completion of drilling and sampling, the boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was monitored on a full-time basis by Golder staff who also located the boreholes in the field, monitored the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in labelled containers and transported to Golder's London laboratory for further examination and testing. Index and classification tests, consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations, were carried out on selected soil samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

## 4.0 SUBSURFACE CONDITIONS

### 4.1 General

Detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory testing carried out on selected samples, are presented on the Record of Borehole sheets, Drawings 1 and 2 and on Figures A-1 and A-8 in Appendix A. The stratigraphic boundaries shown on the Records of Boreholes and on the interpreted stratigraphic profile and cross-sections on Drawings 1 and 2 are inferred from non-continuous samples and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions encountered in the boreholes consist of the existing Highway 401 pavement structure or topsoil underlain by an extensive stratum of stiff to hard silty clay containing clayey silt and sandy silt to silt layers. An interpreted stratigraphic profile is shown on Drawing 2. More detailed descriptions of the subsurface conditions encountered in the boreholes are provided on the Record of Borehole sheets and in the following subsections of this report.

### 4.2 Soil Conditions

#### 4.2.1 Topsoil

About 120 to 400 mm of topsoil was encountered at the ground surface in BH-501, BH-503, BH-504 and BH-505. Materials designated as topsoil in this report were classified solely based on visual and textural evidence. Testing of 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.

#### 4.2.2 Pavement Structure and Fill

Borehole BH-502 was advanced through the median shoulder and encountered about 100 mm of asphalt and about 110 mm of granular base. The granular base was underlain by a 75 mm thick layer of buried asphalt. Beneath the buried asphalt, approximately 1.1 metres of sand and crushed gravel fill was encountered. The granular fill exhibited a Standard Penetration Test (SPT) N value of 17 blows per 0.3 m of penetration and a water content of about 8 per cent.

Beneath the granular fill, a layer of firm to very stiff silty clay fill about 2.1 m thick was encountered at about Elevation 262.2 m. The silty clay fill had SPT N values ranging from 8 to 17 blows per 0.3 m with water contents ranging from about 13 to 21 per cent. A grain size distribution curve for a sample of the silty clay fill is provided on Figure A-2 in Appendix A.

#### 4.2.3 Silty Clay Including Interlayers

An extensive deposit of stiff to hard silty clay was generally encountered beneath the surficial topsoil or pavement structure and fill in all boreholes. The surface of the silty clay was encountered between about Elevation 259.6 and 261.5 m. Boreholes BH-502 to BH-505 were terminated in the silty clay. In BH-501, where the deposit was fully penetrated, the silty clay was about 52.7 m thick. The low plasticity silty clay had SPT N values ranging from



9 to 102 blows per 0.3 m of penetration, measured water contents ranging from about 9 to 23 per cent, and average plastic and liquid limits of 15 and 29 per cent, respectively, based on 23 Atterberg limits determinations. Atterberg limits data are shown on Figures A-6 to A-8 in Appendix A. Grain size distribution curves for samples of the silty clay are shown on Figures A-1 to A-3. Cobbles and boulders should be expected in the silty clay.

The silty clay was interlayered with clayey silt and silt ranging in thickness from 0.2 m to 2.2 m. A few granular interlayers of sand were also encountered with thicknesses of as much as 1.1 m. Such interlayering is considered a typical geologic characteristic of this predominantly fine-grained deposit and these interlayers will be variable in both thickness and vertical position throughout the site.

#### 4.2.4 Lower Clayey Silt

Hard clayey silt was encountered below the extensive silty clay deposit in BH-501 at about Elevation 208.3 m. Borehole BH-501 was terminated in the clayey silt after exploring it for about 3.6 m. The clayey silt had SPT N values of 76 to greater than 100 blows per 0.3 m of penetration, with water contents ranging from about 8 to 12 per cent. The clayey silt had average plastic and liquid limits of 13 and 21 per cent, respectively, based on two Atterberg limits determinations, indicating clayey silt of low plasticity. The Atterberg limits data are shown on Figure A-6 in Appendix A. Grain size distribution curves for samples of the clayey silt are shown on Figure A-4.

### 4.3 Groundwater Conditions

All boreholes, except BH-503, were free of water during drilling. Groundwater was encountered at about 9.7 m depth or Elevation 251.3 m during drilling in BH-503 on January 9, 2019, interpreted to represent a saturated gravelly sand interbed within the silty clay. Groundwater levels/conditions encountered in the boreholes during and shortly after drilling may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized.

Based on the sample water contents and the transition of soil colour from brown to grey, the long-term stabilized groundwater level (or elevation below which the soils should be expected to be saturated) is estimated to be about 3 to 6 m below the ground/pavement surface corresponding to approximately Elevation 258 m. Groundwater levels and seepage conditions in the area will fluctuate seasonally and in response to precipitation events.

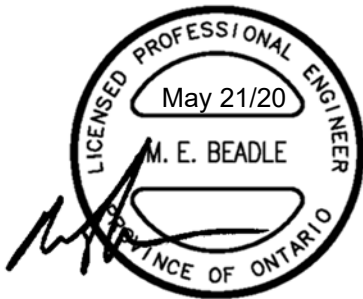
## 5.0 MISCELLANEOUS

The field work described in this report was carried out using equipment supplied and operated by Orbit Drilling Services Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Michael Arthur under the direction of Golder's Project Manager, Mr. Michael E. Beadle, P. Eng.

Routine laboratory tests were carried out at Golder's London laboratory under the direction of Ms. Katie Patton. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates.

This report was prepared by Mr. Randy Axford and reviewed by Mr. Michael E. Beadle, P.Eng., an Associate with Golder. An independent quality review of this report was carried out by Ms. Lisa C. Coyne, P.Eng., Golder's MTO Designated Foundations Contact and Quality Control Auditor.

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**PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT**  
**Glanworth Drive Underpass, Site 19-406**  
**Highways 401, 4 and 21 Structural Replacements**  
**GWP 3030-11-00, Assignment No. 1 (3011-E-0046)**  
**Ministry of Transportation, Ontario – West Region**

## 6.0 PRELIMINARY FOUNDATION ENGINEERING RECOMMENDATIONS

This section of the report provides our preliminary recommendations on the foundation aspects of the design of the new Glanworth Drive underpass. The recommendations are based on Golder's interpretation of the factual information obtained during the field explorations and laboratory testing. It should be noted that the interpretation and recommendations are intended for use only by MTO's design engineer. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

Based on the preliminary design information provided by Dillon, it is understood that the new Glanworth Drive alignment and structure will be located about 440 m east of the existing Glanworth Drive underpass. It is anticipated that the replacement structure will be a two-span structure with approximately 36/38 m spans. The recommendations provided in the following sections reflect the 2014 edition of the Canadian Highway Bridge Design Code (CHBDC 2014).

### 6.1 Bridge Foundations

The subsurface soil conditions at the site typically consist of topsoil or the existing pavement structure and fill underlain by an extensive layer of glacial silty clay that is characteristically interbedded with silt and sand seams/layers. The stabilized groundwater level is inferred to be at about Elevation 258 m.

Golder understands that integral, semi-integral and conventional abutments are considered viable alternatives for design of the structure. The suitability of integral or semi-integral abutments is influenced by the length, type and geometry of the structure, abutment and wingwall heights, number of spans and the subsurface soil conditions. Provided the abutment heights and wingwall lengths are limited to a maximum of 6 and 7 m, respectively, use of integral or semi-integral abutments at the site is considered geotechnically feasible. Integral abutments are typically supported by driven steel H-piles installed with single or double corrugated steel pipe (CSP) liners filled with sand over the top 3 m. Consideration may also be given to supporting integral abutments on concrete-filled steel tube piles provided the increased stiffness can be accommodated in the design. Conventional or semi-integral or hybrid-integral abutments may be founded on spread footings bearing on native soils, steel H-piles or drilled concrete shafts (caissons).

If a conventional or semi-integral design is selected, shallow foundations are the preferred alternative for support of the abutments for this bridge. Shallow foundations are also preferred for support of any piers and the retaining walls. A comparison of foundation alternatives is presented in Table I following the text of this report.

#### 6.1.1 Shallow Foundations

The abutments and retaining walls for the new bridge may be founded on conventional spread/strip footings. A factored ultimate geotechnical resistance of 250 kilopascals (kPa) and a factored serviceability geotechnical resistance of 175 kPa may be used for abutment footings founded on the stiff to very stiff silty clay at about Elevation 260 m, based on a 6 metre wide footing. The foundation for the median pier may be designed using a factored ultimate geotechnical resistance of 500 kPa. Based on an 11,000 kN loading at the pier, the pier footing is expected to settle about 50 mm. The factored serviceability geotechnical resistance for the abutments is not necessarily applicable as the existing ground has the potential to settle as much as 75 millimetres in conjunction

with the embankment loads and the embankment load and settlement govern in this case. Thus, the effects of about 25 to 50 millimetres of differential settlement between the abutments and the centre pier (with greater settlement at the abutments) should be considered in the structural design if spread foundations are to be used at the pier and abutments. Alternatively, if a settlement mitigation strategy is adopted to reduce the settlement under the embankment loading, lesser differential settlements may apply between the strip-footing-supported abutments and centre pier.

Alternatively, “perched” abutments may be used where these are founded on engineered fill approach embankments. The engineered fill should be constructed on the properly prepared stiff to very stiff silty clay at about Elevation 260 m and should consist of a minimum of 2 m of Ontario Provincial Standard Specifications (OPSS).PROV 1010 Granular A placed and compacted in accordance with OPSS.PROV 501. The engineered fill should extend a minimum of 1 m plus the fill thickness beyond the footing and should be sloped at an inclination of 1 horizontal to 1 vertical outward from the underside of the footing to meet the prepared subgrade. Abutment footings founded on engineered fill as described above may be designed using a factored ultimate geotechnical resistance at ULS of 600 kPa, assuming 6 m wide footings. As indicated above, the factored serviceability geotechnical resistance is not necessarily applicable given the embankment settlement impacts without mitigation, and the effects of differential settlement should be considered in the structural design depending on the option(s) adopted.

### **Resistance to Lateral Forces**

Resistance to lateral forces/sliding between the concrete spread/strip footings and the native, undisturbed subsoil should be calculated in accordance with Section 6.10.5 of CHBDC 2014. Assuming that the founding soils are not loosened or disturbed during excavation and footing construction, an effective angle of friction between the cast-in-place concrete footings and the native founding soils of 30 degrees and corresponding unfactored coefficient of friction,  $\tan \delta$ , of 0.58 may be used. An effective angle of friction of 35 degrees and corresponding unfactored  $\tan \delta$  of 0.7 may be used for the resistance between a Granular A pad and cast-in-place concrete.

### **Frost Protection**

All footings should be provided with a minimum of 1.2 m of earth cover or thermal equivalent for frost protection purposes (OPSD 3090.101). Alternatively, the abutment footings could be constructed on free-draining granular engineered fill with reduced cover provided that an appropriate subdrain and outlet is provided to drain the granular materials.

## **6.1.2 Deep Foundations**

### **Geotechnical Axial Resistance - Driven Piles - Abutments**

Based on the boreholes, practical refusal to piling will be achieved at depths of about 53 to 55 m below ground surface. Piling to these depths may not be cost effective nor practicable given the generally very stiff to hard character of the silty clay. Thus, both friction piles and end-bearing piles have been considered. Driven tube piles are not discussed further within this preliminary report since they are commonly considered to be too laterally rigid for effective integral abutment design; however, if the increased stiffness can be accommodated for integral abutment design, this foundation type may be considered further in detail design.

For the design of piles driven to practical refusal at about Elevation 206 m, the factored ultimate geotechnical axial resistances and factored serviceability geotechnical resistances provided in the following table may be used for preliminary design. Based on the conditions encountered in Boreholes BH-501 and BH-503, it is anticipated that refusal will occur on the hard silty clay or clayey silt at about Elevation 206 m. This will result in pile lengths in excess of about 53 to 55 m that may be impractical and/or not cost-effective for this site.

Pile Type	Founding Strata	Maximum (Highest) Tip Elevation (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
HP 310 x 110	Hard clayey silt/silty clay, very dense silty sand	206.0	1,600	1,200
HP 360 x 108		206.0	1,750	1,300

Alternatively, shorter steel H-piles driven into the very stiff to hard silty clay at approximately Elevation 228 m may be designed using the factored ultimate geotechnical axial resistances and factored serviceability geotechnical resistances provided in the table below.

Pile Type	Founding Strata	Maximum (Highest) Founding Elevation (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
HP 310 x 110	Very stiff to hard silty clay	228.0	1,000	750
HP 360 x 108		228.0	1,100	850

The SLS values provided in the tables above correspond to an estimated foundation settlement of about 25 mm, though in comparison with spread foundations, driven piles may be subject to down-drag loads since the piles will resist movement while the ground above compresses under the stresses of the newly constructed approach

embankments. Downdrag loads, as described below, have been accounted for within the SLS resistance values provided above.

While pile foundation settlement will likely not be significantly influenced by approach embankment settlement, because they resist settlement better than shallow spread footings, the magnitude of differential settlement between the abutments and approach embankments could be exacerbated. Further discussion of this issue is provided in Section 6.5 of this report.

Piles supporting integral abutments require placement of a corrugated steel pipe (CSP) liner filled with loose uniform sand around the upper 3 metres of the pile to allow appropriate pile flexibility to accommodate thermal effects and other forces at the bridge deck; depending on the site grades, pre-augering may be required to install these CSPs. A Non-Standard Special Provision (NSSP) for the CSPs detailing the sand gradation should be included in the Contract Documents. Piles should be installed and monitored in accordance with OPSS.PROV 903. The maximum ultimate resistance of two times the factored ultimate geotechnical resistance value indicated above should be noted on the foundation drawing(s).

The silty clay, clayey silt and various interlayers are known to contain cobbles and boulders which may interfere with driving of the piles or cause damage to pile tips. Driving shoes should be provided but should be flush with the pile cross-section (i.e., not oversized) to assist with developing frictional resistance.

The actual pile penetration and pile set characteristics will be dependent, to some extent, upon the driving equipment selected by the contractor. The pile driving hammer should be selected such that the energy delivered to the piles is sufficient to achieve the termination criteria (ultimate geotechnical resistances and tip elevations) stated in the Contract Documents.

### **Geotechnical Axial Resistance – Drilled Shafts (Caissons) - Pier**

It is understood that the preliminary design is based on supporting the median pier on two drilled shafts with each shaft required to support 8,000 kN at Ultimate Limit States (ULS) and 5,500 kN at Serviceability Limit States (SLS). Based on the loadings, the following would be required:

Shaft Diameter (m)	Length (m)	Geotechnical Resistance (kN)	
		Factored Ultimate	Factored Serviceability
1.5	37	8,000	5,500
1.8	34	>8,000	5,500
2.1	31	>8,000	5,500

Given the drilled shaft lengths required to achieve the geotechnical resistance, founding the median pier on a spread/raft footing is preferable from a geotechnical perspective. Consideration could also be given to increasing the number of drilled shafts (provided that the spacing can accommodate any reduction due to group effects) to permit the use of shorter drilled shafts to achieve the required capacity.

### Downdrag Loads - Abutments

The new approach embankments will cause long-term consolidation settlement of the underlying silty clay deposit as a result of the embankment surcharge. The consolidation settlement is time-dependent and will not completely occur during the construction period unless the embankments are placed well in advance (i.e., more than one year in advance) of bridge construction or lightweight fill is used to construct some portions of the embankments. Post-construction settlement of the silty clay deposit relative to the piles will result in development of negative skin friction acting on the piles.

Based on the results of the investigation, for preliminary design, the unfactored downdrag load acting on an HP 310x110 pile driven to Elevation 228 metres may be taken as approximately 300 kilonewtons per pile. The downdrag load on piles driven to Elevation 206 metres may be taken as approximately 1,000 to 1,500 kilonewtons per pile. The downdrag load has no effect on the geotechnical axial capacity of the pile and should not be included in the design check that considers the factored ultimate geotechnical resistance. At ULS, the pile structural capacity is to be checked using the following:

$$P_f > 1.25 Q_d + \gamma_p DF$$

where:  $P_f$  = factored axial compression resistance of the pile

$Q_d$  = permanent dead load/sustained load on the pile

$\gamma_p$  = load factor for drag force = 1.25

$DF$  = drag force on the pile

Downdrag loads are not expected for the drilled shafts for the median pier. Should drilled shafts be used to support the abutments, downdrag loads will need to be considered and will be significantly higher than those provided above for the H-piles.

During detailed design, the downdrag loads should be re-evaluated based on the actual pile loads, bridge configuration, embankment heights and expected settlements.

### Frost Protection

Pile caps should be provided with a minimum frost cover of 1.2 metres of soil cover or thermal equivalent.

### Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. In the case of integral abutments, the vertical piles must provide the resistance to the lateral loading. The horizontal reaction to the pile can be estimated using the following equation and ranges of soil properties.

$$K_h = \text{coefficient of horizontal subgrade reaction (MPa/m)} = \frac{67 S_u}{d} \quad \text{for cohesive soils}$$

where

$D$  = pile width or diameter (m)

$n_h$  = constant of horizontal subgrade reaction (MPa/m)



- $S_u$  = undrained shear strength of the soil (MPa)  
 $Z$  = depth below ground surface (m)

The following parameters may be incorporated into the calculations of the coefficient of horizontal subgrade reaction ( $k_h$ ) for structural analysis of a single vertical pile. The range in values reflects the variability in subsurface conditions as well as the two extremes of design: the requirement for flexibility if integral abutments are selected and the requirement for lateral support in the cases of non-integral abutments or pier foundations, as well as the variability in subsurface conditions, the approximate nature of analysis and the non-linear nature of the soil behaviour (such that  $k_h$  is a function of deflection).

Soil Type	$n_h$ (MPa/m)	$S_u$ (MPa)
Granular backfill within CSPs	2 - 4	n/a
Very stiff to hard silty clay	n/a	0.15 to 0.25

If the design is sensitive to the range in  $n_h$  and  $S_u$ , analyses using p-y curves should be completed as part of detailed design.

The lateral resistances for the various foundation options are summarized in the following table.

Pile Type	Lateral Resistance	
	Factored Ultimate (kN)	Factored Serviceability (kN)
Integral abutments		
- HP 310 x 110, weak axis bending	125	30
- HP 310 x 108, weak axis bending	175	50
Semi-integral or conventional abutments		
- HP 310 x 110, strong axis bending	300	35
- HP 360 x 108, strong axis bending	400	60
1.8 metre diameter drilled shaft (caisson)	2,250	1,500

The lateral resistances are based on Brom's Method from "Design and Construction of Driven Pile Foundations Workshop Manual – Volume 1, FHWA, Pub. No. FHWA H1 97-013, Revised November 1998". Fixed-head conditions were assumed for the H-pile, while free head conditions were assumed for concrete caissons. These represent "order of magnitude" estimates of lateral resistance. A compressive strength of 32 MPa was assumed

for the concrete caissons. The factored serviceability values are based on 10 millimetres of horizontal deflection at the top of the pile.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction and factored resistance values in the direction of loading by a reduction factor,  $R$ , as follows:

<i>Pile Spacing in Direction of Loading, <math>d</math> = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor, <math>R</math></i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

## 6.2 Liquefaction Potential and Seismic Analysis

### 6.2.1 Seismic Parameters

The new bridge is in Seismic Performance Category (SPC) 1. Unless this bridge is considered a lifeline bridge, based on  $S(0.2) = 0.1$  and  $S(1.0) = 0.06$  which satisfies the criteria in Table 4.10 for structure periods less than 0.5 seconds and greater than or equal to 0.5 seconds, seismic analysis of bridges in SPC 1 is not a requirement of the CHBDC (Clause 4.4.5.1). However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clauses 4.4.3.5 and 4.4.10.5.

As outlined in Clause 4.4.3.2, Table 4.1 a Site Class C (soil undrained shear strength,  $s_u > 100 \text{ kPa}$ ) should be used for design purposes.

### 6.2.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.<sup>4</sup> The characteristics of the cohesive soils indicate that they are not susceptible to liquefaction. Although layers of saturated granular materials are present, they are relatively thin. The liquefaction potential is considered low based on the soil profile type, age of the deposits, relative density/consistency and the historically low regional seismicity. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations and the effect of seismic forces on embankment stability is not considered warranted unless the structure is deemed to be a lifeline bridge.

<sup>4</sup> FHWA, 1997: "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.

### 6.3 Lateral Earth Pressures for Design

The lateral pressures acting on the bridge abutments and associated wingwalls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the freedom of lateral movement of the structure and the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutments, in accordance with the CHBDC.

- Select, free-draining granular fill meeting the specifications of OPSS.PROV 1010 Granular A or Granular B Type III should be used as backfill behind the abutments and walls. This fill should be placed and compacted in accordance with OPSS.PROV 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 and 3190.100.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment walls in accordance with CHBDC Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501.
- The granular fill may be placed either in a zone with a width equal to at least 1.2 metres behind the back of the stem (Case a from Commentary on CHBDC Figure C6.20) or within the wedge-shaped zone defined by a line drawn at a 1 horizontal to 1 vertical slope extending up and back from the rear face of the footing (Case b from Commentary on CHBDC Figure C6.20).
- For Case a, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM).

Soil unit weight: 20 kN/m<sup>3</sup>

Coefficient of lateral earth pressure:  
At rest,  $K_o$  0.50

- For Case b, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	<u>GRANULAR A</u>	<u>GRANULAR B</u> <u>Type III</u>
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of lateral earth pressure:		
Active, $K_a$	0.27	0.31
Passive, $K_p$	3.7	3.3

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- For integral abutments, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance.

A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.2 of the CHDBC.

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. The lateral earth pressure coefficients should be adjusted if there is sloping ground at the back of the wall.

## 6.4 Construction Considerations

### 6.4.1 Shallow Foundations

The founding soils are sensitive to disturbance and loosening due to water seepage and/or ponding and construction equipment or foot traffic when moist to wet. Placement of a concrete working slab (100 millimetres thick, 20 megapascal concrete) is recommended at the base of the excavations for the footing areas. Exposure without protection from the working slab may result in loosening or softening of the founding soils. The cleaned excavation base should be inspected by qualified geotechnical personnel prior to placing the working slab. It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation is completed with the geotechnical personnel on site and the working slab placed immediately after footing inspection.

### 6.4.2 Deep Foundations

Cobbles and boulders should be expected in the soils at the site and which may impact pile driving or caisson drilling operations. An NSSP should be added to the Contract Documents to alert the contractor to the need for special procedures to deal with cobbles, boulders and other obstructions during pile or caisson installation.

Deep foundations should be installed and monitored in accordance with OPSS.PROV 903 and Ontario Provincial Standard Drawing (OPSD) 3000.150 and 3001.150, and SS103-11 (Pile Driving Control). It is also recommended that Pile Dynamic Analyzer (PDA) testing be completed on a representative portion of piles driven at each foundation element and in each stage, to augment or replace pile driving control using the Hiley formula. We note that the OPSD and construction monitoring requirements may require some additional provisions in the event that the very long steel H piles are selected for foundations since the lengths will require additional considerations for splicing and reducing the potential for misalignment and damage during driving.

## 6.5 Embankments

The bridge and its approach embankments will be constructed on new alignment with the new Glanworth Drive road surface as much as about 8 metres above the local ground surface. The site is underlain by an extensive deposit of generally stiff to very stiff silty clay glacial till. Bedrock was encountered about 50 to 60 metres below ground surface.

For embankments up to 8 m in height, with side slopes oriented no steeper than 2 horizontal to 1 vertical, the factor of safety for global stability will be greater than 1.3 in temporary/short-term conditions during construction, and greater than 1.5 in long-term/permanent conditions. Embankments greater than 8 m in height should be provided with a mid-height bench with a minimum bench width of 2 m, to promote surficial stability and erosion protection on the embankment side slopes.

Settlement calculations indicate that about 75 millimetres of settlement may occur as a result of embankment construction; the founding soils are relatively stiff and overconsolidated, but the silty clay layer is quite thick and stresses induced by the new embankments penetrate deeply because of the embankment geometry. It is estimated that about one-third to one-half of this settlement will occur during construction and the remainder of the settlement should be complete within about two to three years following completion of embankment construction. Even though these settlement estimates are based on site-specific information, published research and recognised analysis methods, settlement estimates in geotechnical engineering tend to be both conservative and subject to a degree of uncertainty on the order of  $\pm 20$  per cent even with a significant body of site-specific detailed laboratory and field testing beyond what is typically available for MTO projects.

In addition, the magnitude of post-construction settlement could have some impact(s) on the design of an approach slab/sleeper slab system. As a result, potential methods for mitigating post-construction settlements were evaluated and are discussed below:

- **Preload, Surcharge with Vertical Drains** – Prefabricated vertical drains (wick drains) are typically used in softer cohesive soils to rapidly dissipate excess pore water pressures and, hence, accelerate settlements. This alternative is not considered to be appropriate or cost-effective at this site due to the relatively stiff and overconsolidated nature of the site soils, and the relatively small magnitude of settlement. In softer soils, wick drains can be rapidly installed by pushing them into the ground with a mandrel system. However, at this site, the stiff to very stiff silty clay till is not suitable for installation by this method; boreholes drilled to about 40 metres depth and backfilled with concrete sand would be required. We also understand that this project could be subject to an accelerated schedule. It is estimated that at least three months would be required to properly install sand drains at the required frequency and to the required depth, and a further two to four months would be required following embankment construction for sufficient settlement to occur. It is estimated that the cost of the embankments would approximately double to accommodate the drains, compared to conventional construction with no special settlement mitigation measures.
- **Construct Approaches with Lightweight Fill** – Portions of the new approach embankments could be constructed using lightweight fill materials such as expanded polystyrene (EPS) or lightweight cellular concrete. For this site, for preliminary decision making, the lightweight fill would be placed to some percentage of the embankment height and extend back from the abutments at least 10 metres. Beyond this 10-metre zone, the lightweight fill height would be tapered down to intersect the original ground at an inclination of about 2 horizontal to 1 vertical and provide a settlement and fill type transition zone. Conventional fill would be placed over the lightweight fill to construct the remainder of the embankments to grade, construct the pavements and flatten the slopes. While this alternative will address the short- and long-term settlement concerns, we estimate that embankment construction costs will be about three to five times greater compared to conventional earth-fill embankment construction without settlement mitigation measures.
- **“Do Nothing” with Provision to Correct Road Profile** – Since the magnitude of the settlements are not excessive, conventional embankment construction using earth fill and without any settlement mitigation effort could be considered, particularly since the settlement estimates are subject to some degree of uncertainty, both in terms of magnitude and rate. The embankments would be constructed in a conventional manner. It would be beneficial to monitor ground settlement during initial embankment construction through to paving. Construction-phase settlement monitoring would permit a more refined estimate of post-construction ground and embankment settlement performance. Following paving, long-term settlement points would be

strategically located to permit monitoring of any finished roadway settlements with time. Provided that a provision for milling and paving of the surface course asphalt and any adjustment to guiderails and the like are included in the contract to be completed about two to three years after construction, correction of the roadway profile to meet MTO performance criteria could be carried out. There remains some probability that this treatment may not even be necessary depending on the actual settlement behaviour of the silty clay till beyond completion of the embankment filling and roadway paving. We do note that, to be proactive and to reduce the potential for needing to carry out approach slab repairs, an approach slab/sleeper slab design that can accommodate about 50 millimetres of post-construction settlement and a subsequent mill and overlay treatment should be considered if this design approach is implemented.

Based on our evaluation, from a geotechnical/foundations engineering perspective, the “do nothing” alternative with provision for future grade correction is likely to be the most cost-effective alternative and, in our opinion, represents the preferred alternative.

## 6.6 Excavations and Temporary Cut Slopes

Excavations for pile caps and/or abutments will penetrate the existing fill and organic materials into the silty clay and should be completed in accordance with OPSS 902 and FOUN0003. The groundwater level is expected to be at about Elevation 258 m and will fluctuate seasonally. The excavations may extend below the groundwater level; however, seepage volumes from the cohesive founding soils are expected to be low, although excavations may encounter some groundwater “perched” within non-cohesive fill or native materials atop the clayey soils. If necessary, groundwater control for such seepage may be achieved by pumping from properly constructed and filtered sumps in the base of the excavation in accordance with OPSS 517 and SP517F01. Sumps should be maintained outside of the foundation limits. Surface water runoff should be directed away from the excavations at all times. The specifications should also require that the contractor be prepared to immediately support and drain any of the saturated silt, silty sand and silty gravel interlayers that are encountered during excavation.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The topsoil and fill materials at this site would be classified as Type 3 soils as would any cohesionless materials below the groundwater level. The native clayey materials would be classified as Type 2 soils.

### 6.6.1 Temporary Protection Systems

To support the excavation sides and permit the use of vertical cuts, temporary protection systems will be required where space is restricted and will not permit the use of open cuts. These systems are to be designed by and the limits determined by the contractor.

Temporary protection systems could consist of soldier piles and lagging, where the H-piles would be driven or installed within a pre-bored hole to a suitable depth and horizontal lagging installed as the excavation proceeds, or driven steel sheet piling. Support of the system(s) could be in the form of struts and walers in the case of pile cap/abutment excavations or rakers and anchors. The protection system must be designed to accommodate the loads applied from pressures and surcharge pressures from area line or point loads as well as address the impact(s) of sloping ground behind the system. The lateral movement of the temporary support system should meet Performance Level 2 as specified in OPSS.PROV 539.

## 7.0 RECOMMENDATIONS FOR DETAIL DESIGN

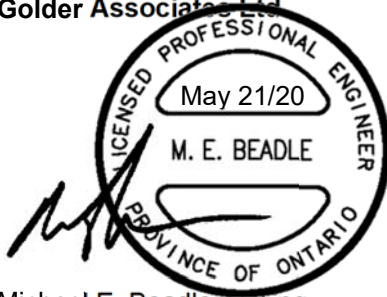
Depending on the selection of integral, semi-integral or conventional abutments, both driven piles and shallow foundations are considered to be feasible alternatives for support of the replacement structure. Depending on the structure/abutment type and foundation option selected, additional foundation investigation may be required during detail design. A standard MTO foundation investigation for a bridge structure is considered appropriate for the site, consistent with the requirements of MTO's Guideline for Foundation Engineering Services, Version 1 (May 2019). The number and location of required boreholes will be dependent on the location and alignment of the proposed structure selected for Detail Design and may be reduced if the location of any part of the of the proposed structure coincides with the location of existing boreholes. In addition, if refinement of the magnitude and time-rate of embankment settlement is required to support the settlement mitigation strategy, oedometer testing is recommended, although it may be challenging to obtain relatively undisturbed samples within the stiff, overconsolidated deposits at this site.

The recommendations given in this Preliminary Foundation Design Report should be expanded upon and updated in the Foundation Design Report for Detail Design in accordance with MTO's Guideline for Foundation Engineering Services, Version 1 (May 2019).

## 8.0 MISCELLANEOUS

This report was prepared by Mr. Randy Axford and reviewed by Mr. Michael E. Beadle, P.Eng., an Associate with Golder. An independent quality review of this report was carried out by Ms. Lisa C. Coyne, P.Eng., Golder's Designated MTO Contact and Quality Control Auditor.

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TABLE I

**COMPARISON OF FOUNDATION ALTERNATIVES**

Glanworth Drive Underpass  
 Highway 401 Structural Replacements  
GWP 3030-11-00

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>
Steel piles driven to refusal	<ul style="list-style-type: none"> <li>• Feasible</li> </ul>	<ul style="list-style-type: none"> <li>• High axial resistance</li> <li>• Little to negligible settlement at abutments</li> <li>• Suitable for integral abutments</li> </ul>	<ul style="list-style-type: none"> <li>• Significant pile lengths (55 m +/-)</li> <li>• Negative skin friction reduces available ULS and SLS geotechnical resistances</li> <li>• Potentially longer installation time due to pile splicing</li> <li>• Potential for misalignment and pile damage in the clay due to pile lengths</li> </ul>
Steel friction piles	<ul style="list-style-type: none"> <li>• Feasible and preferred foundation type</li> </ul>	<ul style="list-style-type: none"> <li>• Conventional pile lengths (30 m +/-)</li> <li>• Splicing time reduced or eliminated</li> <li>• Negative skin friction greatly reduced compared to end-bearing piles</li> <li>• Suitable for integral abutments</li> </ul>	<ul style="list-style-type: none"> <li>• Some settlement of piles will occur</li> <li>• Negative skin friction reduces available SLS geotechnical resistance</li> <li>• Additional design and construction considerations for adaptation of sleeper/approach slabs and monitoring for roadway profile control</li> </ul>

**COMPARISON OF FOUNDATION ALTERNATIVES**

<b>FOUNDATION OPTION</b>	<b>FEASIBILITY</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>
Shallow foundations	<ul style="list-style-type: none"> <li>• Feasible</li> </ul>	<ul style="list-style-type: none"> <li>• Low cost</li> <li>• Rapid construction</li> <li>• Moderate geotechnical resistances available</li> <li>• Suitable for semi-integral abutments and could be suitable for integral abutments (hybrid system)</li> <li>• Largely avoids creation of relatively “fixed” abutments as compared to the potential long-term settlement of approach embankments and, therefore, controls potential differential settlement between abutment and approach embankments better than other foundation solutions</li> </ul>	<ul style="list-style-type: none"> <li>• Settlement of abutment foundations, beyond typical SLS expectations, will occur as a result of long-term compression of the ground from approach embankment loads</li> <li>• Additional design and construction considerations to adapt for integral abutment design</li> <li>• Additional design and construction considerations for adaptation of sleeper/approach slabs and monitoring for roadway profile control</li> </ul>
Drilled Shafts	<ul style="list-style-type: none"> <li>• Feasible</li> </ul>	<ul style="list-style-type: none"> <li>• Reduces temporary roadway protection extents</li> <li>• Eliminates requirements for pier footing</li> </ul>	<ul style="list-style-type: none"> <li>• 30m+ lengths required to support design loads</li> <li>• Deep shafts or larger diameter shafts may not be feasible for local contracts</li> </ul>

NOTES: 1. Table to be read in conjunction with accompanying report.

Prepared by: MEB  
Checked by: LCC

# ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

## MINISTRY OF TRANSPORTATION, ONTARIO

### PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

### MODIFIERS FOR SECONDARY COMPONENTS<sup>1,2</sup>

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component ( <i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some ( <i>i.e.</i> , some sand)
≤ 10	trace ( <i>i.e.</i> , trace fines)

- Only applicable to components not described by Primary Group Name.
- Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

### PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

#### Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q<sub>t</sub>*), porewater pressure (*u*) and sleeve friction (*f<sub>s</sub>*) are recorded electronically at 25 mm penetration intervals.

#### Dynamic Cone Penetration Resistance (DCPT); N<sub>d</sub>:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

- PH:** Sampler advanced by hydraulic pressure  
**PM:** Sampler advanced by manual pressure  
**WH:** Sampler advanced by static weight of hammer  
**WR:** Sampler advanced by weight of sampler and rod

### SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

### SOIL TESTS

w	water content
PL, w <sub>p</sub>	plastic limit
LL, w <sub>L</sub>	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
D <sub>R</sub>	relative density (specific gravity, G <sub>s</sub> )
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
Y	unit weight

- Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

### COARSE-GRAINED SOILS

#### Compactness<sup>1</sup>

Term	SPT 'N' (blows/0.3m) <sup>2</sup>
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	➤ 50

- Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
- SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

### FINE-GRAINED SOILS

#### Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' <sup>1,2</sup> (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
- SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

### Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

# LIST OF SYMBOLS

## MINISTRY OF TRANSPORTATION, ONTARIO

Unless otherwise stated, the symbols employed in the report are as follows:

### I. GENERAL

$\pi$	3.1416
$\ln x$	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta\sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
U	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
E	void ratio
N	porosity
S	degree of saturation

\* Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity)

#### (a) Index Properties (continued)

w	water content
$w_l$ or LL	liquid limit
$w_p$ or PL	plastic limit
$I_p$ or PI	plasticity index = $(w_l - w_p)$
NP	non-plastic
$w_s$	shrinkage limit
$I_L$	liquidity index = $(w - w_p) / I_p$
$I_C$	consistency index = $(w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

#### (b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

#### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_\alpha$	secondary compression index
$m_v$	coefficient of volume change
$C_v$	coefficient of consolidation (vertical direction)
$C_h$	coefficient of consolidation (horizontal direction)
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$


#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction = $\tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 - \sigma_3)$
$S_t$	sensitivity

Notes: 1  
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$


<b>PROJECT</b> 12-1132-0076		<b>RECORD OF BOREHOLE No BH-501</b>		1 OF 4	<b>METRIC</b>
<b>W.P.</b> 3030-11-00		<b>LOCATION</b> N 4747544.1 , E 405419.5 (Lat 42.861046 , Long -81.268680)		<b>ORIGINATED BY</b> MA	
<b>DIST</b> _____ <b>HWY</b> 401		<b>BOREHOLE TYPE</b> POWER AUGER (HOLLOW STEM), MUD ROTARY (HQ ROD), TRICONE		<b>COMPILED BY</b> AMS	
<b>DATUM</b> GEODETIC		<b>DATE</b> January 18 - March 29, 2019		<b>CHECKED BY</b> 	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		W <sub>P</sub>	W	W <sub>L</sub>	GR	SA	SI	CL
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE		WATER CONTENT (%)										
261.15	GROUND SURFACE																			
0.00	TOPSOIL, silty Black																			
0.12	SILTY CLAY (CL), some sand, trace gravel Brown turning grey at about elev. 258.3m Stiff to hard		1	SS	30															
			2	SS	26															
			3	SS	12															
			4	SS	9															
			5	SS	10															
			6	SS	10															
			7	SS	11															
		8	SS	15																
		9	SS	50																
		10	SS	45																
		11	SS	24																
246.35																				
14.80			12	SS	53															

LDN\_MTO\_06 12-1132-0076-1001.GPJ LDN\_MTO.GDT 23/10/19

Continued Next Page

 +<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

<b>PROJECT</b> 12-1132-0076		<b>RECORD OF BOREHOLE No BH-501</b>		2 OF 4	<b>METRIC</b>
<b>W.P.</b> 3030-11-00		<b>LOCATION</b> N 4747544.1 , E 405419.5 (Lat 42.861046 , Long -81.268680)		<b>ORIGINATED BY</b> MA	
<b>DIST</b> _____ <b>HWY</b> 401		<b>BOREHOLE TYPE</b> POWER AUGER (HOLLOW STEM), MUD ROTARY (HQ ROD), TRICONE		<b>COMPILED BY</b> AMS	
<b>DATUM</b> GEODETIC		<b>DATE</b> January 18 - March 29, 2019		<b>CHECKED BY</b> 	


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>P</sub>	W	W <sub>L</sub>		GR SA SI CL				
								20 40 60 80 100	WATER CONTENT (%)									
							○ UNCONFINED      + FIELD VANE											
							● QUICK TRIAXIAL      × LAB VANE											
							20 40 60 80 100	10 20 30										
245.95 15.20	CLAYEY SILT (ML), some gravel, some sand Grey Hard  SILTY CLAY (CL), trace to some gravel, trace to some sand Grey Very stiff					246												
			13	SS	16	245						○						
						244												
			14	SS	21	243						○						
						242												
			15	SS	26	241						○						
						240												
			16	SS	24	239												
						238												
			17	SS	21	237						○						
237.52 23.63 237.10 24.05	CLAYEY SILT (ML), some gravel, some sand Grey Hard  SILTY CLAY (CL), some sand, trace gravel Grey Hard		18	SS	33	236												
236.05 25.10 25.40	SILT (ML), some clay Grey Compact  SILTY CLAY (CL), some sand, trace gravel Grey Very stiff to hard		19	SS	25	235												
						234												
		20	SS	55	233													
						232												
232.80 28.35 28.50	SILT (ML), some clay Grey Dense  SILTY CLAY (CL), some sand, trace gravel Grey Stiff to hard		21	SS	33													



LDN\_MTO\_06 12-1132-0076-1001.GPJ LDN\_MTO.GDT 23/10/19

Continued Next Page

 +<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE



<b>PROJECT</b> 12-1132-0076		<b>RECORD OF BOREHOLE No BH-501</b>		4 OF 4	<b>METRIC</b>
<b>W.P.</b> 3030-11-00		<b>LOCATION</b> N 4747544.1, E 405419.5 (Lat 42.861046, Long -81.268680)		<b>ORIGINATED BY</b> MA	
<b>DIST</b> _____ <b>HWY</b> 401		<b>BOREHOLE TYPE</b> POWER AUGER (HOLLOW STEM), MUD ROTARY (HQ ROD), TRICONE		<b>COMPILED BY</b> AMS	
<b>DATUM</b> GEODETIC		<b>DATE</b> January 18 - March 29, 2019		<b>CHECKED BY</b> 	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>					
	SILTY CLAY (CL), some sand, trace gravel Grey Stiff to hard						20	40	60	80	100									
			32	SS	52															
			33	SS	31															
			34	SS	52															
			35	SS	52															
		36	SS	58																
208.27																				
52.88	CLAYEY SILT (ML-CL), trace to some gravel, trace to some sand Grey Hard		37	SS	76															
			38	SS	48/ 101mm															
204.76			39	SS	100															
56.39	END OF BOREHOLE  Borehole dry during drilling on June 18, 2019.																			


LDN\_MTO\_06 12-1132-0076-1001.GPJ LDN\_MTO.GDT 23/10/19









<b>PROJECT</b> 12-1132-0076		<b>RECORD OF BOREHOLE No BH-503</b>		1 OF 4	<b>METRIC</b>
<b>W.P.</b> 3030-11-00		<b>LOCATION</b> N 4747555.5 , E 405350.2 (Lat 42.861158 , Long -81.269525)		<b>ORIGINATED BY</b> MA	
<b>DIST</b> _____ <b>HWY</b> 401		<b>BOREHOLE TYPE</b> POWER AUGER (HOLLOW STEM), TRICONE (HQ ROD),		<b>COMPILED BY</b> AMS	
<b>DATUM</b> GEODETIC		<b>DATE</b> January 9 - 17, 2019		<b>CHECKED BY</b> 	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
261.04	GROUND SURFACE						20	40	60	80	100						
0.00	TOPSOIL, silty Black																
0.22	CLAYEY SILT (ML), some sand Brown																
0.52	SILTY CLAY (CL), some sand, trace gravel Brown to grey at about elev. 258.1m Stiff to very stiff		1	SS	15								○				
			2	SS	21								○				
			3	SS	27								○				
			4	SS	20								○				
			5	SS	13								○				
			6	SS	12								○				
			7	SS	14								○				
			8	SS	13								○				
			9	SS	17								○				
			10	SS	20								○				
			11	SS	10								○				
251.34			12	SS	45								○				
9.70	GRAVELLY SAND (SP-GW), some silt Grey Dense												○				
250.94	sandy CLAYEY SILT (ML), trace gravel Grey Dense												○				
10.10			13	SS	44								○				
249.53	sandy SILTY CLAY (CL), trace to some gravel Grey Very stiff to hard												○				
11.51			14	SS	24								○				
			15	SS	22								○				

LDN\_MTO\_06 12-1132-0076-1001.GPJ LDN\_MTO.GDT 23/10/19

Continued Next Page

 +<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT <u>12-1132-0076</u>		<b>RECORD OF BOREHOLE No BH-503</b>		2 OF 4	<b>METRIC</b>
W.P. <u>3030-11-00</u>	LOCATION <u>N 4747555.5 , E 405350.2 (Lat 42.861158 , Long -81.269525)</u>	ORIGINATED BY <u>MA</u>			
DIST <u>          </u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER (HOLLOW STEM), TRICONE (HQ ROD),</u>	COMPILED BY <u>AMS</u>			
DATUM <u>GEODETIC</u>	DATE <u>January 9 - 17, 2019</u>	CHECKED BY <u>[Signature]</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	w <sub>p</sub>			w	w <sub>L</sub>	
245.79																	
15.25	SILTY CLAY (CL), some sand, trace gravel Grey Very stiff to hard																
			16	SS	25									9 49 42			
			17	SS	21												
			18	SS	28												
			19	SS	25												
			20	SS	22												
			21	SS	24												
			22	SS	29									10 48 42			
			23	SS	30												
			24	SS	25												
			25	SS	34												

LDN\_MTO\_06 12-1132-0076-1001.GPJ LDN\_MTO.GDT 23/10/19

Continued Next Page

 +<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT <u>12-1132-0076</u>		<b>RECORD OF BOREHOLE No BH-503</b>		3 OF 4	<b>METRIC</b>
W.P. <u>3030-11-00</u>	LOCATION <u>N 4747555.5 , E 405350.2 (Lat 42.861158 , Long -81.269525)</u>	ORIGINATED BY <u>MA</u>			
DIST <u>          </u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER (HOLLOW STEM), TRICONE (HQ ROD),</u>	COMPILED BY <u>AMS</u>			
DATUM <u>GEODETIC</u>	DATE <u>January 9 - 17, 2019</u>	CHECKED BY <u>W</u>			




SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
	SILTY CLAY (CL), some sand, trace gravel Grey Very stiff to hard						231										
			26	SS	41		230										14 41 45
							229										
			27	SS	38		228										
							227										
			28	SS	48		226										
							225										
			29	SS	64		224										
							223										
			31	SS	42		222										2 13 42 43
							221										
			32	SS	50		220										
							219										
							218										
			34	SS	40		217										
216.54 44.50																	

LDN\_MTO\_06 12-1132-0076-1001.GPJ LDN\_MTO.GDT 23/10/19

Continued Next Page

 +<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

PROJECT <u>12-1132-0076</u>		<b>RECORD OF BOREHOLE No BH-503</b>		4 OF 4	<b>METRIC</b>
W.P. <u>3030-11-00</u>	LOCATION <u>N 4747555.5 , E 405350.2 (Lat 42.861158 , Long -81.269525)</u>	ORIGINATED BY <u>MA</u>			
DIST <u>          </u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER (HOLLOW STEM), TRICONE (HQ ROD),</u>	COMPILED BY <u>AMS</u>			
DATUM <u>GEODETIC</u>	DATE <u>January 9 - 17, 2019</u>	CHECKED BY <u>W</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>					
	sandy SILTY CLAY (CL), trace gravel Grey Hard		35	SS	65		216													
			36	SS	39		215													
							214													
							213													
			37	SS	45		212													
							211													
							210													
			38	SS	42		209													
							208													
							207													
			39	SS	35		206													
208.94																				
52.10	SILTY SAND (SM), fine to coarse, some gravel Grey Very dense		40	SS	45/ 76mm															
207.84																				
53.20	sandy SILTY CLAY (CL), trace gravel Grey Hard																			
				41	SS	102														
205.09			42	SS	71															
55.95	END OF BOREHOLE																			
	Groundwater encountered at about elev. 251.3m during drilling on January 9, 2019.																			

LDN\_MTO\_06 12-1132-0076-1001.GPJ LDN\_MTO.GDT 23/10/19

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



PROJECT <u>12-1132-0076</u>		<b>RECORD OF BOREHOLE No BH-505</b>		1 OF 1	<b>METRIC</b>
W.P. <u>3030-11-00</u>	LOCATION <u>N 4747560.1 , E 405338.4 (Lat 42.861201 , Long -81.269669)</u>	ORIGINATED BY <u>MA</u>			
DIST <u>          </u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER (HOLLOW STEM)</u>	COMPILED BY <u>AMS</u>			
DATUM <u>GEODETIC</u>	DATE <u>January 9, 2019</u>	CHECKED BY <u>W</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						PLASTIC LIMIT   NATURAL LIMIT   MOISTURE   CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED   + FIELD VANE ● QUICK TRIAXIAL   × LAB VANE	20	40	60	80	100	w <sub>p</sub>	w	w <sub>L</sub>					
261.73	GROUND SURFACE																				
0.00	TOPSOIL Black																				
0.30	SILTY CLAY (CL), some sand, trace gravel Brown to grey at about elev. 258.8m Stiff to hard																				
			1	SS	22									○							
			2	SS	29									○							
			3	SS	32									○	10	30			12   45   43		
			4	SS	20									○							
			5	SS	16									○							
			6	SS	16									○							
			7	SS	14									○							
			8	SS	12									○	10	30			8   46   46		
			9	SS	10									○							
			10	SS	13									○							
252.88																					
8.85	SILTY SAND (SM), some gravel Grey Compact													○							
252.34			11	SS	26									○							
9.39	SILTY CLAY (CL), some sand, trace gravel Grey Very stiff													○							
9.60	END OF BOREHOLE																				

LDN\_MTO\_06 12-1132-0076-1001.GPJ LDN\_MTO.GDT 23/10/19

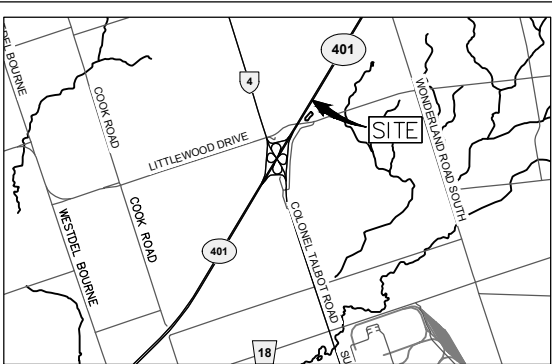
**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No. 3030-11-00



SHEET

GLANWORTH DRIVE UNDERPASS  
HIGHWAY 401 IMPROVEMENTS  
BOREHOLE LOCATIONS



KEY PLAN



LEGEND

Borehole - Current Investigation

No.	ELEVATION	CO-ORDINATES (NAD 83, MTM ZONE 11)	
		NORTHING	EASTING
501	261.15	4 747 544.1	405 419.5
502	263.60	4 747 549.9	405 384.4
503	261.04	4 747 555.5	405 350.2
504	260.96	4 747 539.4	405 428.3
505	261.73	4 747 560.1	405 338.4

NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans based on drawings provided by Dillon Consulting.



PLAN



GLANWORTH ROAD

BH-505

BH-503

BH-502

BH-501

BH-504

HIGHWAY 401

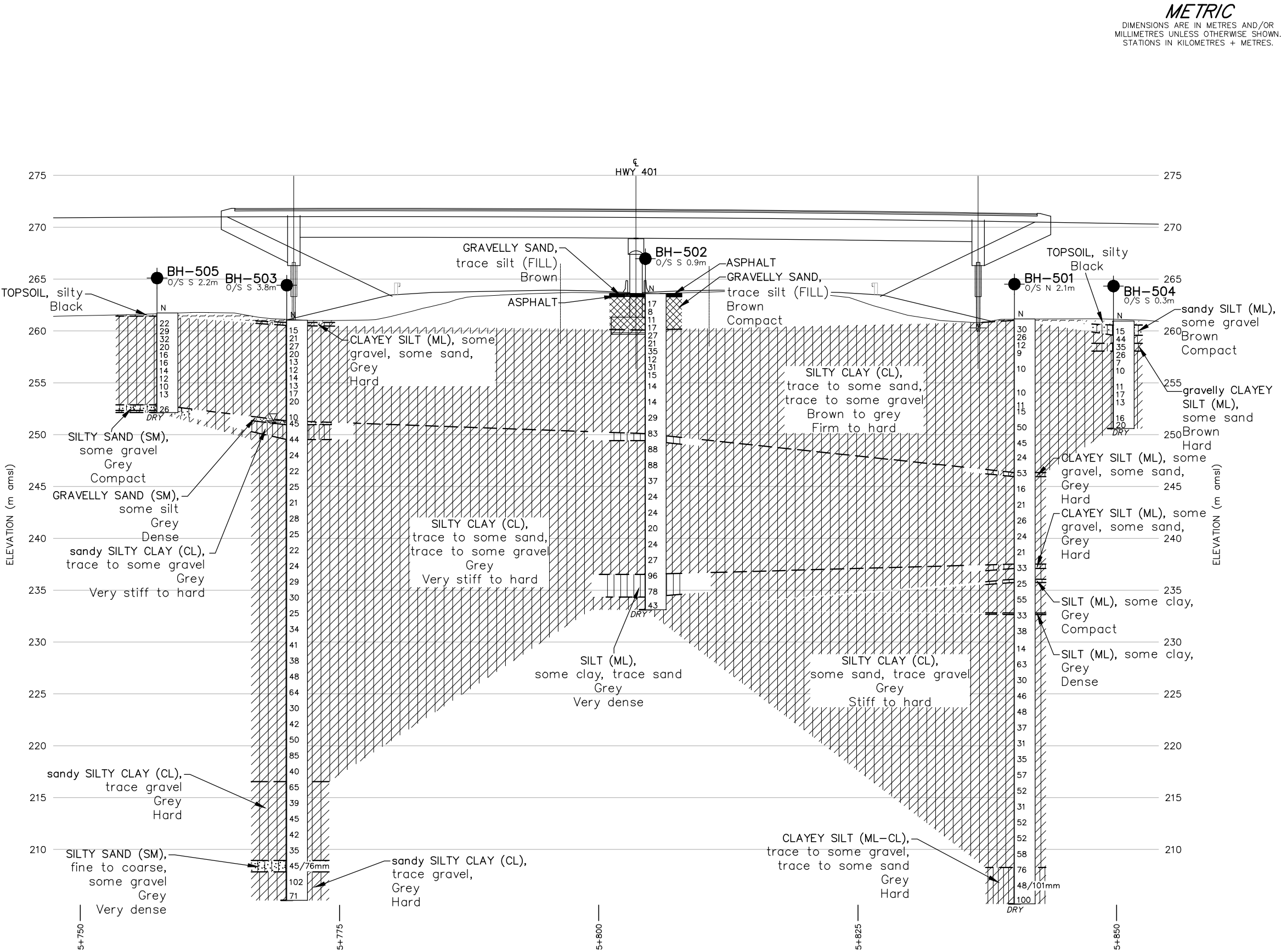
N 4 747 600  
E 405 350

N 4 747 550

E 405 400  
N 4 747 500

E 405 450

E 405 300



PROFILE ALONG E GLANWORTH ROAD



CONT No.  
WP No. 3030-11-00

GLANWORTH DRIVE UNDERPASS  
HIGHWAY 401 IMPROVEMENTS  
SOIL STRATA

SHEET

C:\0\_Golder\_Logs\Horizontal\LogoGreyScale.jpg

LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL encountered during drilling
- DRY Water level not established

No.	ELEVATION	CO-ORDINATES (NAD 83, MTM ZONE 11)	
		NORTHING	EASTING
501	261.15	4 747 544.1	405 419.5
502	263.60	4 747 549.9	405 384.4
503	261.04	4 747 555.5	405 350.2
504	260.96	4 747 539.4	405 428.3
505	261.73	4 747 560.1	405 338.4

NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

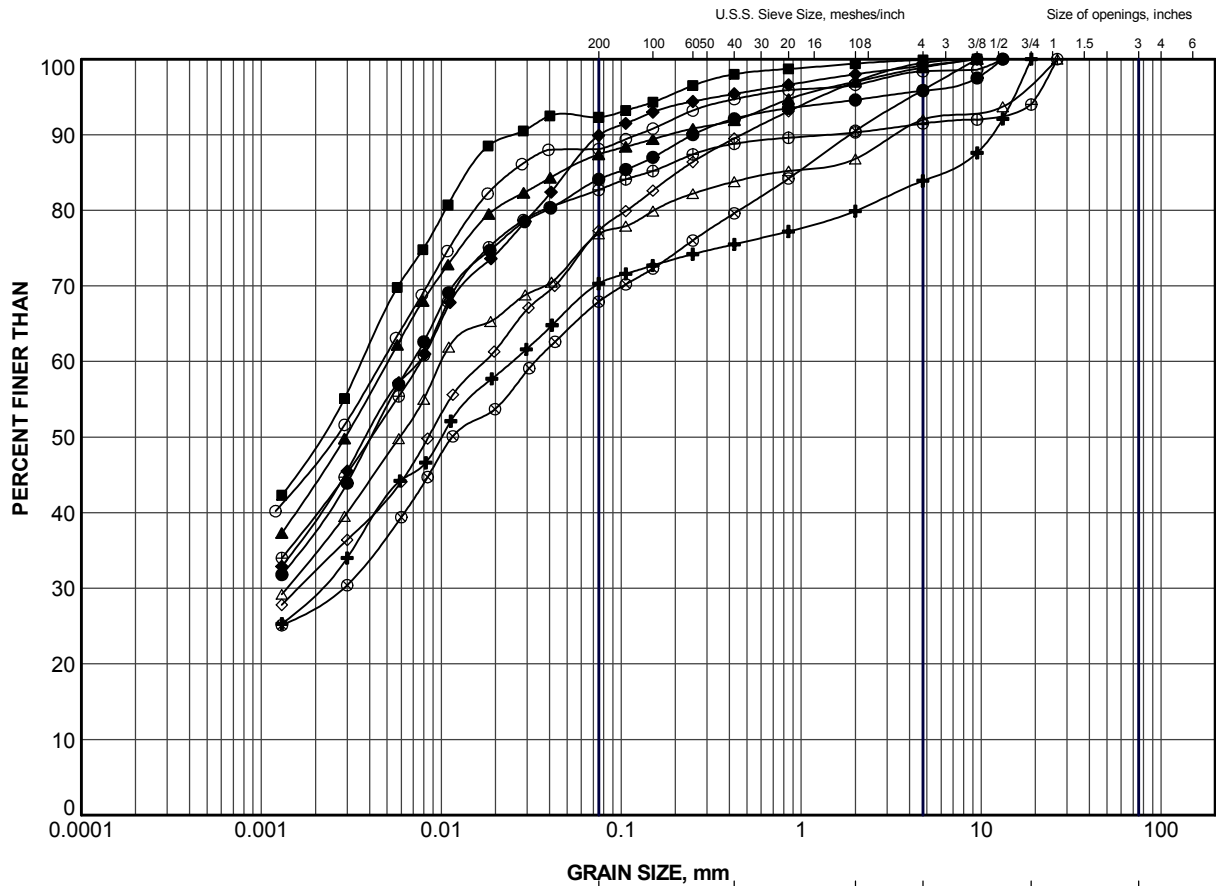
REFERENCE

Base plans based on drawings provided by Dillon Consulting.

NO.	DATE	BY	REVISION
Geocres No.	40114-190		
HWY.	401	PROJECT NO.	12-1132-0076
SUBM'D.	RA	CHKD.	MB
DRAWN:	ZJB	CHKD.	MB
DATE:	Mar 31/20	DIST.	
APPD.		SITE:	
DWG.	2		

**APPENDIX A**

# Laboratory Testing Results



CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-501	3	258.9
■	BH-501	6	254.3
▲	BH-501	9	250.9
+	BH-501	12A	246.5
◆	BH-501	17	238.9
◇	BH-501	20	234.3
○	BH-501	25	226.6
△	BH-501	31	217.4
⊗	BH-501	34	212.8
⊕	BH-502	2	262.1

PROJECT

GLANWORTH ROAD UNDERPASS  
HIGHWAY 401 IMPROVEMENTS  
GWP 3054-11-00

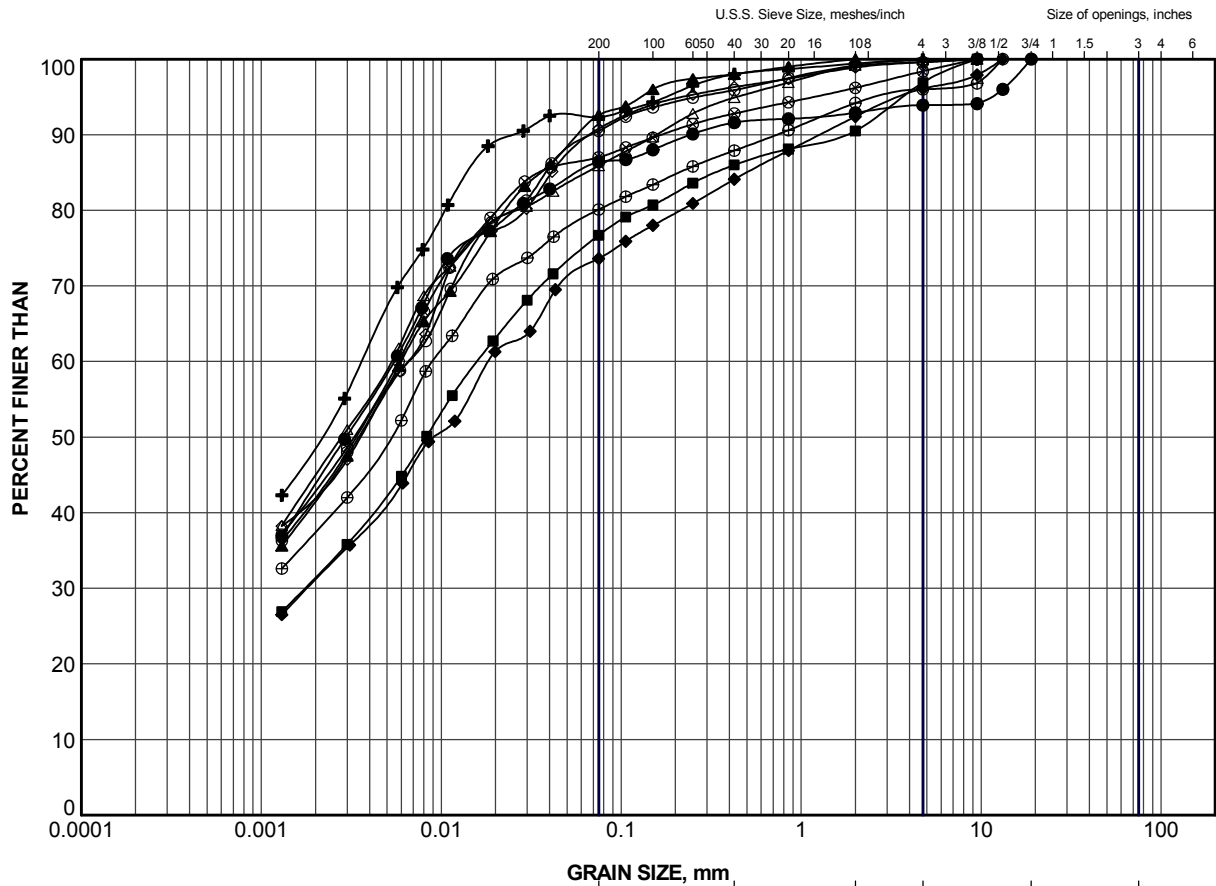
TITLE

**GRAIN SIZE DISTRIBUTION**  
**SILTY CLAY (1 OF 3)**



GOLDER

PROJECT No.	12-1132-0076	FILE No.	12-1132-0076-1001-R020A1
DRAWN	ZJB	Oct 23/19	SCALE N/A REV.
CHECK			<b>FIGURE A-1</b>



CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-502	8	257.5
■	BH-502	16	247.1
▲	BH-502	22	238.1
+	BH-503	6	256.2
◆	BH-503	12B	250.9
◇	BH-503	16	245.2
○	BH-503	22	236.1
△	BH-503	26	230.0
⊗	BH-503	31	222.3
⊕	BH-503	35	216.2

PROJECT

GLANWORTH ROAD UNDERPASS  
HIGHWAY 401 IMPROVEMENTS  
GWP 3054-11-00

TITLE

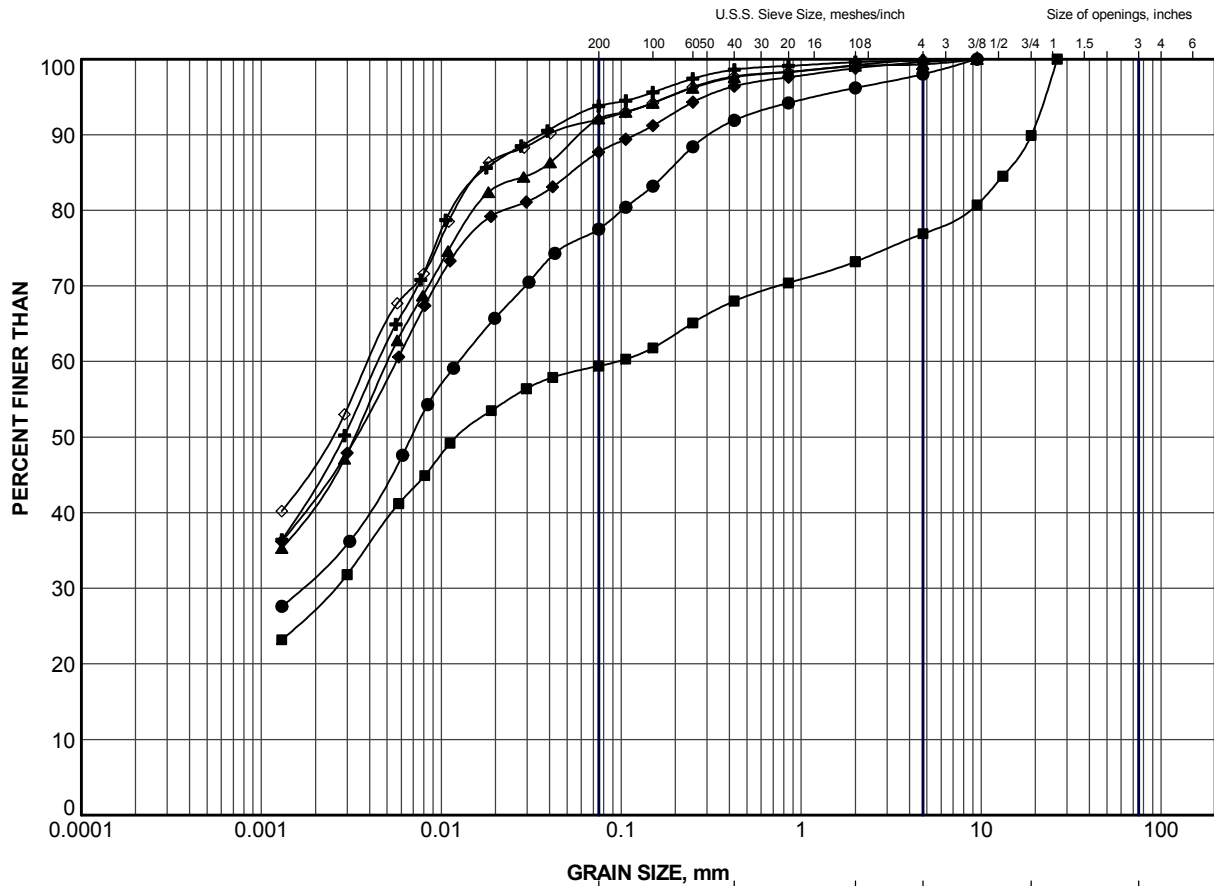
**GRAIN SIZE DISTRIBUTION**  
**SILTY CLAY (2 OF 3)**



GOLDER

PROJECT No.	12-1132-0076	FILE No.	12-1132-0076-1001-R020A2
DRAWN	ZJB	Oct 23/19	SCALE N/A REV.
CHECK			

**FIGURE A-2**

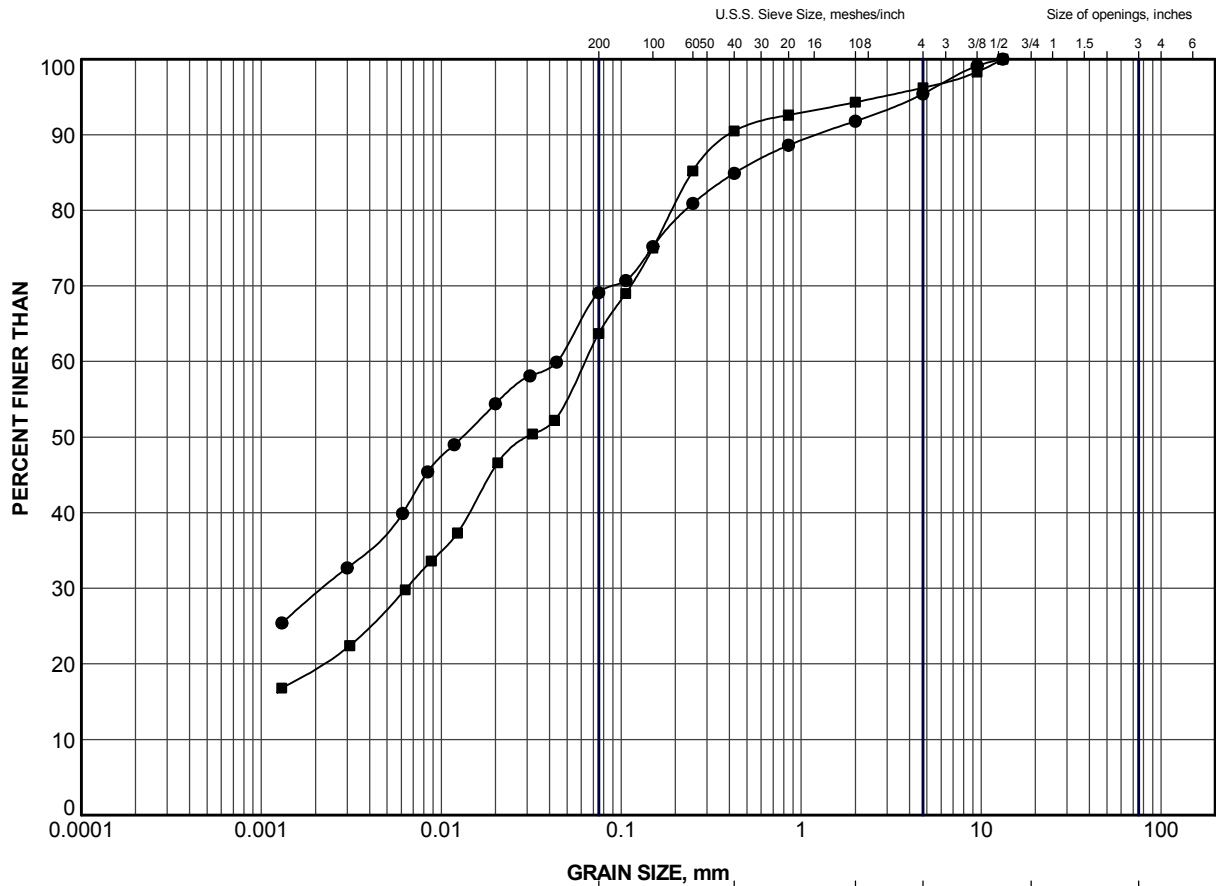


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-503	42	205.6
■	BH-504	2	259.4
▲	BH-504	4	257.9
+	BH-504	6	256.4
◆	BH-505	3	259.4
◇	BH-505	8	255.6

PROJECT				GLANWORTH ROAD UNDERPASS HIGHWAY 401 IMPROVEMENTS GWP 3054-11-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY CLAY (3 OF 3)			
PROJECT No.		12-1132-0076		FILE No.		12-1132-0076-1001-R020A3	
DRAWN		ZJB		SCALE		N/A	
CHECK		Oct 23/19		REV.			
				<b>FIGURE A-3</b>			



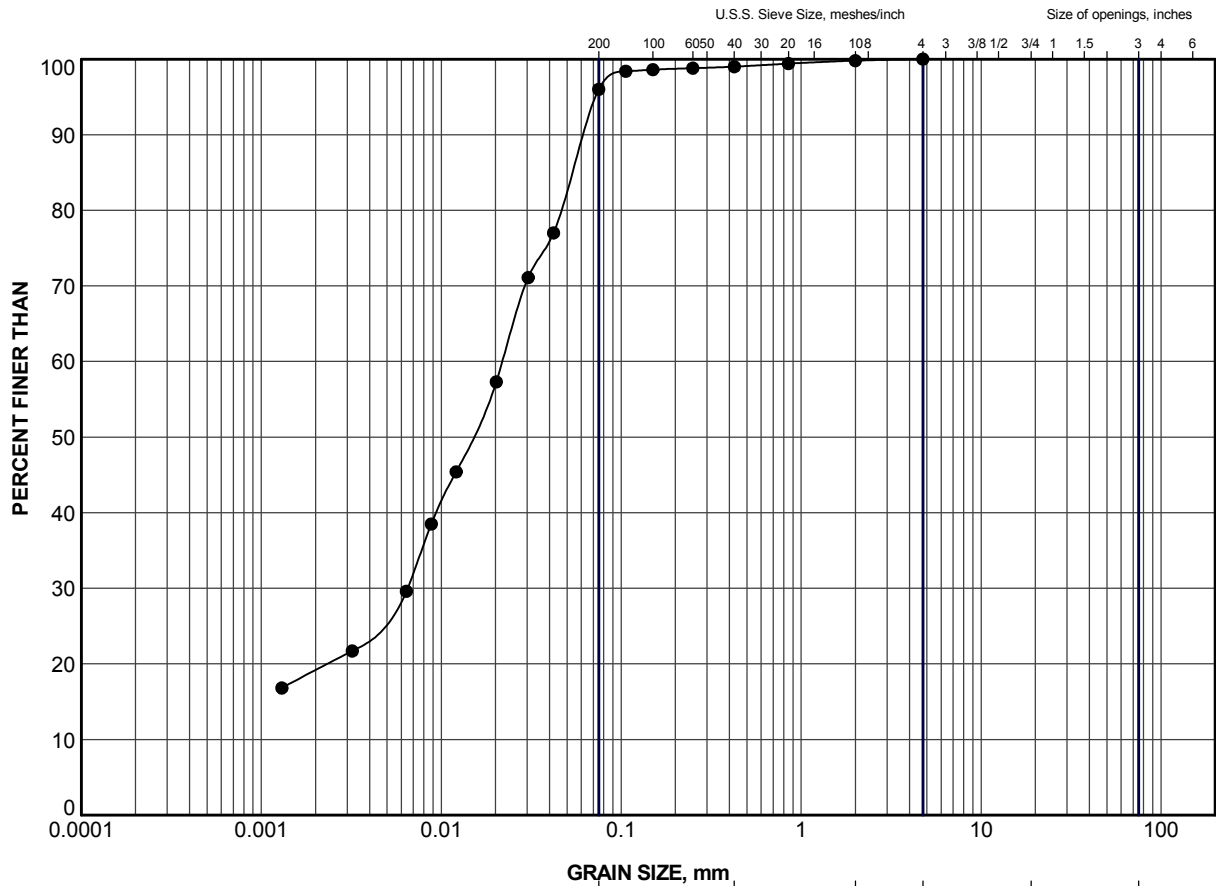
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

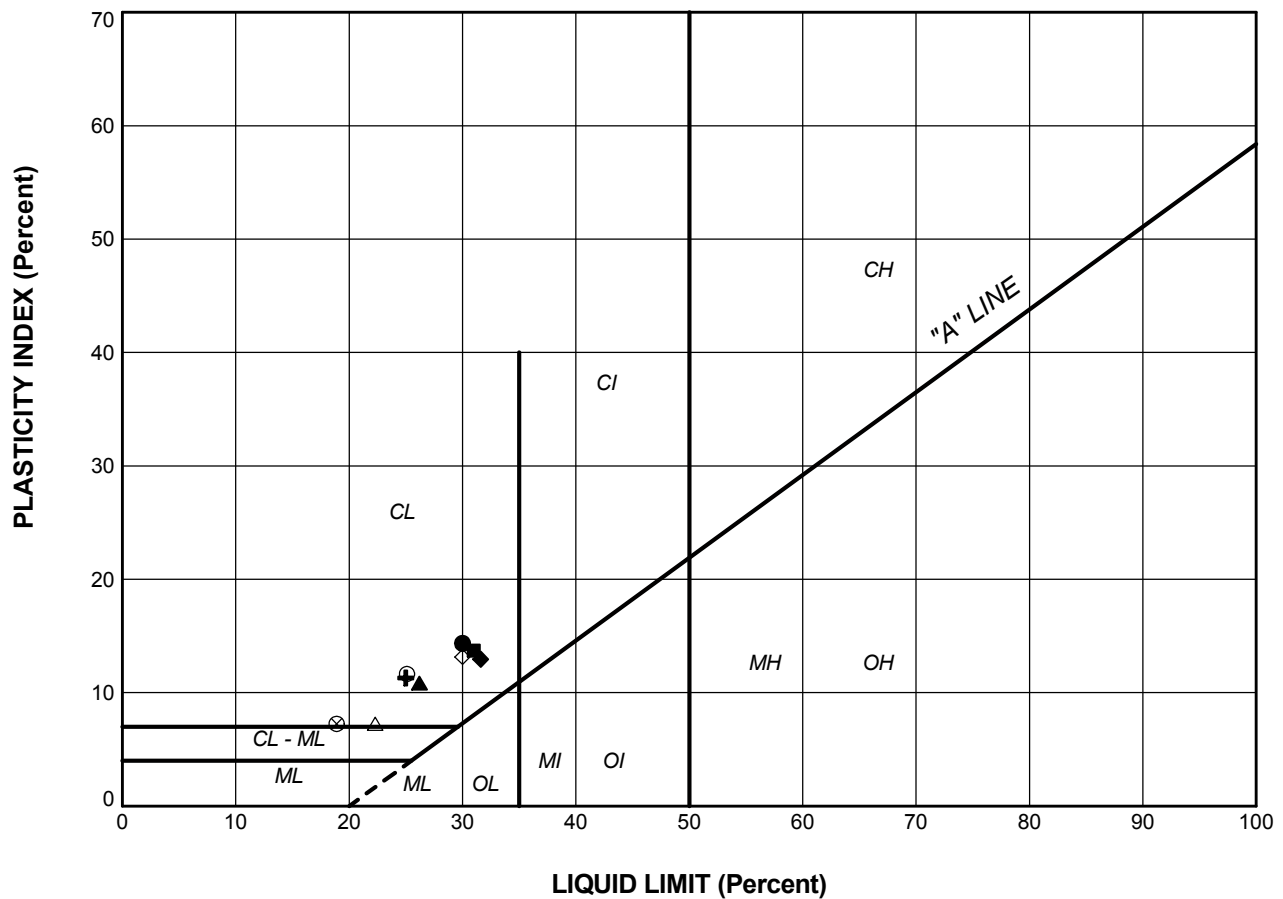
### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH-501	37	208.3
■	BH-501	38	206.7

PROJECT				GLANWORTH ROAD UNDERPASS HIGHWAY 401 IMPROVEMENTS GWP 3054-11-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT			
PROJECT No.		12-1132-0076		FILE No.		12-1132-0076-1001-R020A4	
DRAWN		ZJR		SCALE		N/A	
CHECK		Oct 23/19		REV.			
				FIGURE A-4			







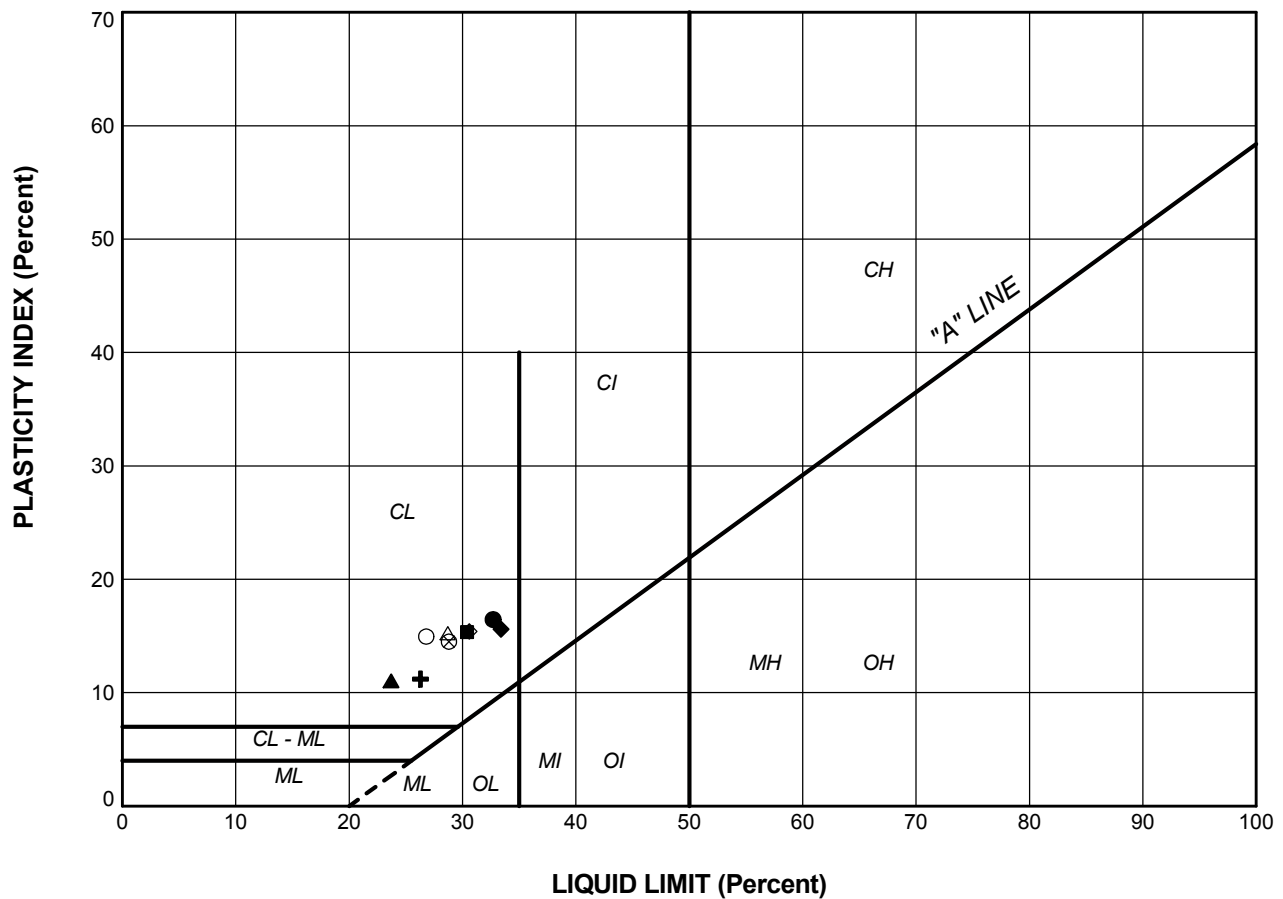
**SOIL TYPE**  
 C = Clay  
 M = Silt  
 O = Organic

**PLASTICITY**  
 L = Low  
 I = Intermediate  
 H = High

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-501	3	30.0	15.7	14.4
■	BH-501	9	31.0	17.3	13.7
▲	BH-501	17	26.2	15.4	10.9
+	BH-501	20	25.0	13.7	11.3
◆	BH-501	25	31.6	18.7	13.0
◇	BH-501	31	30.0	16.9	13.2
○	BH-501	34	25.1	13.5	11.7
△	BH-501	37	22.3	15.1	7.3
⊗	BH-501	38	18.9	11.7	7.3

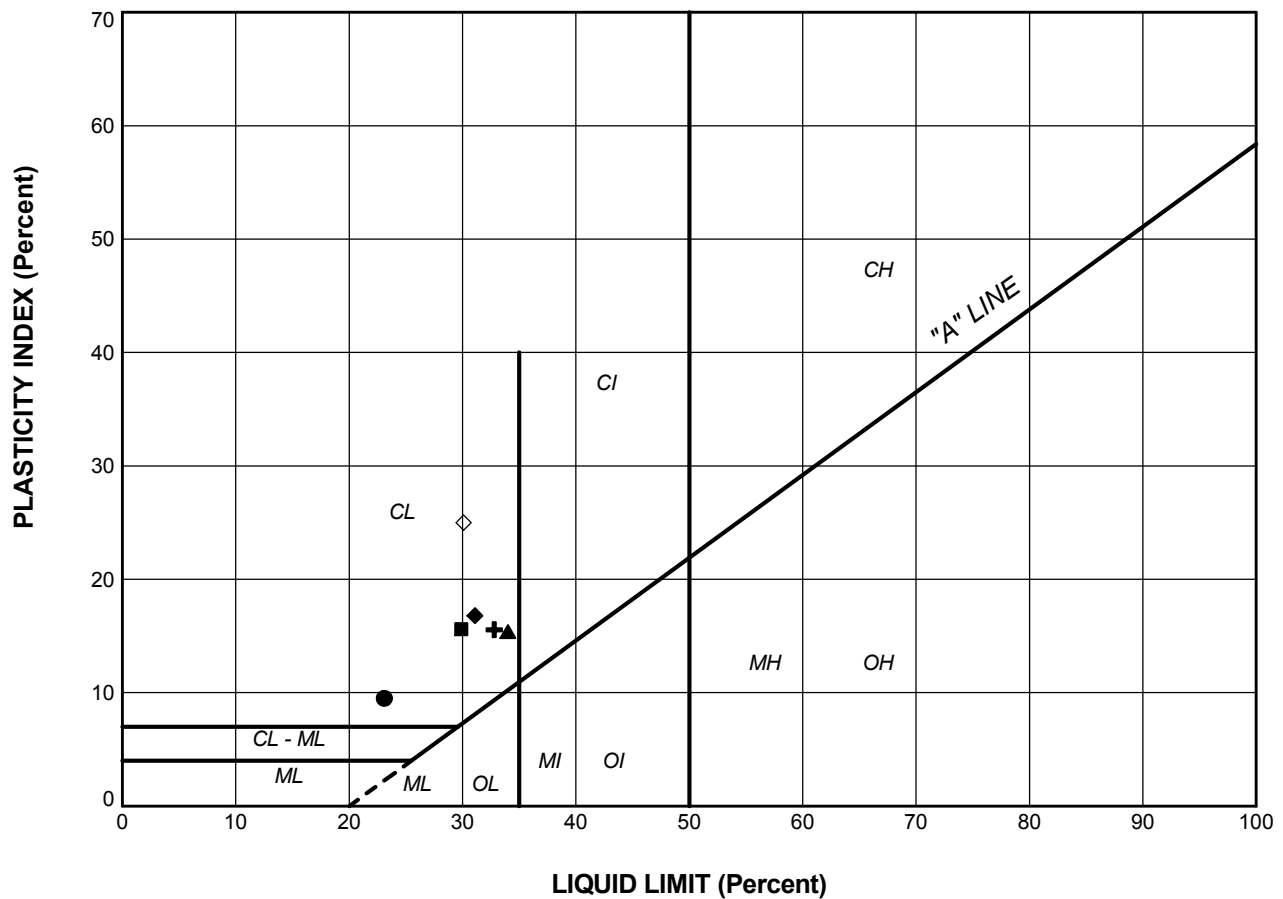
PROJECT				GLANWORTH ROAD UNDERPASS HIGHWAY 401 IMPROVEMENTS GWP 3054-11-00			
TITLE				PLASTICITY CHART			
PROJECT No.		12-1132-0076		FILE No.		12-1132-0076-1001-R020A6	
DRAWN		ZJB		SCALE		N/A	
CHECK		Oct 23/19		REV.			
				<b>FIGURE A-6</b>			



### LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-502	2	32.7	16.3	16.5
■	BH-502	8	30.4	15.1	15.4
▲	BH-502	16	23.7	12.7	11.1
+	BH-502	22	26.3	15.1	11.2
◆	BH-503	6	33.4	17.8	15.6
◇	BH-503	16	30.6	15.2	15.4
○	BH-503	22	26.8	11.9	15.0
△	BH-503	26	28.7	13.5	15.3
⊗	BH-503	31	28.8	14.3	14.5

PROJECT				GLANWORTH ROAD UNDERPASS HIGHWAY 401 IMPROVEMENTS GWP 3054-11-00			
TITLE							
PLASTICITY CHART							
PROJECT No.		12-1132-0076		FILE No.		12-1132-0076-1001-R020A7	
DRAWN		ZJB		SCALE		N/A	
CHECK		[Signature]		REV.			
GOLDER				FIGURE A-7			



### LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH-503	42	23.1	13.6	9.5
■	BH-504	2	29.9	14.3	15.6
▲	BH-504	4	34.0	18.6	15.4
+	BH-504	6	32.8	17.3	15.6
◆	BH-505	3	31.1	14.3	16.8
◇	BH-505	8	30.1	5.1	25.0

PROJECT				GLANWORTH ROAD UNDERPASS HIGHWAY 401 IMPROVEMENTS GWP 3054-11-00			
TITLE				PLASTICITY CHART			
PROJECT No.		12-1132-0076		FILE No.		12-1132-0076-1001-R020A8	
DRAWN		ZJR Oct 23/19		SCALE		N/A	
CHECK		[Signature]		REV.			
GOLDER				FIGURE A-8			



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