



May 19, 2017

## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

### Harmony Road Underpass Highway 401 Improvements from Brock Road to Courtice Road Regional Municipality of Durham W.O. 10-20011

**Submitted to:**

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REPORT



**GEOCRES No. 30M15-290**

**Report Number: 11-1184-0143-14**

**Distribution:**

1 E-Copy, 1 Hard Copy - MTO Central Region  
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**PRELIMINARY FOUNDATION REPORT  
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# **PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
HARMONY ROAD UNDERPASS  
HIGHWAY IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD  
REGIONAL MUNICIPALITY OF DURHAM  
W.O. 10-20011**



## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the future improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, Ontario.

This report addresses the results of the foundation investigation carried out for the proposed new Highway 401/Harmony Road underpass structure as part of the proposed Harmony Road and Bloor Street interchange improvements.

The terms of reference for the preliminary foundation engineering services are outlined in MTO's Request for Proposals (RFP) for Assignment No. 2010-E-0062, dated June 2011. The scope of work for the preliminary foundation engineering services is presented in Section 5.8 of URS Canada Inc.'s (now AECOM) *Technical Proposal* for this assignment, as well as Golder's Scope Change for Foundations Engineering Services letter dated December 8, 2014.

## **2.0 SITE DESCRIPTION**

The proposed Harmony Road underpass will be part of an extension of Harmony Road to the south of the existing Harmony Road and Bloor Street intersection. This new structure will be located approximately 250 m east of the Bloor Street overpass and approximately 5.5 km west of the Courtice Road interchange, in the City of Oshawa.

The existing ground surface at the site varies from about Elevation 83 m to 84 m on the north side of Highway 401, to Elevation 81 m to 82.5 m on the south side. The existing Highway 401 grade is at about Elevation 82.5 m to 83 m at the proposed structure site. The proposed grade of the new Harmony Road underpass varies from about Elevation 92 m at the north abutment, to about Elevation 91 m at the south abutment, with approach embankments up to approximately 8 m in height.

The site is currently undeveloped with vegetation cover consisting of trees and low lying brush and grasses.

## **3.0 INVESTIGATION PROCEDURES**

The field work for this preliminary subsurface investigation was carried out in March 2015, during which time two boreholes (Boreholes H1 and H2) were drilled at the locations shown on Drawing 1. The boreholes were advanced with a CME-55 track-mounted drill rig, supplied and operated by DBW Drilling Ltd. of Ajax, Ontario, using 150 mm diameter continuous flight solid stem augers. The boreholes extended to depths of 8.1 m and 9.6 m below ground surface. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586-11 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soil).

The groundwater conditions were observed in the open boreholes throughout the drilling operations and a standpipe piezometer was installed in one borehole (Borehole H1) to permit monitoring of the groundwater level at the site. The piezometer consists of 50 mm diameter PVC pipe, with a slotted screen sealed at a select depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the



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piezometer pipe was sealed with bentonite pellets and backfilled to the ground surface. The piezometer installation details and water level readings are indicated on the borehole record contained in Appendix A. Borehole H2 was backfilled to the ground surface with bentonite pellets in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member of Golder's engineering staff who located the boreholes in the field, supervised the drilling, sampling and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and testing. Index and classification tests consisting of water contents, Atterberg limits, and grain size distributions were carried out on selected soil samples. All of the geotechnical laboratory tests were carried out to MTO and/or ASTM Standards as applicable.

The borehole locations were established in the field by Golder personnel using a hand-held GPS unit. The ground surface elevation at each borehole was estimated from the digital terrain model for the site as provided by AECOM. The borehole locations (referenced to the MTM NAD83 co-ordinate system) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are shown on Drawing 1.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
H1	4,861,065.9	358,912.0	83.5	8.1
H2	4,860,978.6	358,909.8	82.5	9.6

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

This section of Highway 401 is located within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)<sup>1</sup> and *Urban Geology of Canadian Cities* (Brennand, 1998)<sup>2</sup>. The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in this area of the Iroquois Plain are typically comprised of glaciolacustrine clays, silts and sands to gravelly sands, which are underlain by an extensive till deposit that is mapped in this area as the Bowmanville Till. More recent alluvial deposits of gravel, sand, silt and/or clay are present in the creek valleys.

### 4.2 Subsurface Conditions

Boreholes H1 and H2 were advanced in the vicinity of the proposed north and south abutments, respectively, at the proposed Harmony Road underpass site. The detailed subsurface soil and groundwater conditions encountered in the boreholes, and the results of in situ and laboratory testing are presented on the borehole

<sup>1</sup> Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3rd Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.

<sup>2</sup> Brennand, T.A. 1998. *Urban Geology Note: Oshawa Ontario*. In P.F. Karrow, and O. L. White (Eds.), Geological Association of Canada, Special Paper 42: *Urban Geology of Canadian Cities*, p. 353-364.



records contained in Appendix A. The results of geotechnical laboratory testing are also presented in Appendix B. The interpreted stratigraphic profile is shown on Drawing 1.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile drawing are inferred from observations of drilling progress and non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the site consist of relatively thin layers of topsoil, fill and surficial silty clay underlain by a very stiff to hard clayey silt till deposit, which is in turn underlain by a silty sand to sand deposit. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

#### **4.2.1 Topsoil**

Approximately 500 mm and 200 mm of topsoil was encountered immediately below the existing ground surface in Boreholes H1 and H2, respectively.

#### **4.2.2 Fill**

Approximately 0.4 m and 0.6 m of fill material was encountered beneath the topsoil in Boreholes H1 and H2, respectively. The base of the fill was encountered at about Elevation 82.6 m in Borehole H1 on the north side of the highway, and at about Elevation 81.7 m in Borehole H2 on the south side of the highway.

The fill material consists of silty clay containing trace to some sand, trace gravel, and trace organics.

#### **4.2.3 Surficial Silty Clay**

A deposit of silty clay was encountered beneath the topsoil and fill material in both of the boreholes advanced at this site. The thickness of this silty clay deposit ranges from approximately 0.6 m to 1.6 m, with its base encountered at about Elevation 82.0 m in Borehole H1 on the north side of the highway, and Elevation 80.1 m in Borehole H2 on the south side of the highway.

The deposit consists of silty clay containing trace to some sand and trace to some gravel. An Atterberg limits test carried out on one selected sample of this deposit measured a liquid limit of about 44 per cent, a plastic limit of about 17 per cent, and a plasticity index of about 27 per cent. This result, which is plotted on a plasticity chart on Figure B1 in Appendix B, indicates that the material is classified as a silty clay of intermediate plasticity. The measured water content of one sample of the clayey silt to silty clay deposit is 28 per cent.

The SPT 'N'-values measured within the silty clay deposit range from 8 blows to 19 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

#### **4.2.4 Clayey Silt Till**

A deposit of clayey silt till was encountered underlying the surficial silty clay deposit in both of the boreholes advanced at this site. The thickness of this clayey silt till deposit varied from approximately 3.1 m to 5.5 m at the borehole locations. The surface of the clayey silt till was encountered between Elevation 82.0 m and 80.1 m, with the surface higher on the north side of the highway, and the deposit base was encountered between Elevation 76.5 m and 77.0 m.

The till deposit consists of clayey silt with sand, some gravel. The presence of cobbles and / or boulders was inferred based on grinding of the augers during drilling, as noted on the borehole records in Appendix A. The



results of grain size distribution tests performed on three selected samples of the clayey silt till deposit are shown on Figure B2 in Appendix B.

Atterberg limits tests carried out on selected samples of this deposit measured liquid limits ranging from about 15 to 20 per cent, plastic limits ranging from about 10 to 11 per cent, and plasticity indices ranging from about 5 to 9 per cent. These results, which are plotted on a plasticity chart on Figure B3 in Appendix B, indicate that the material is classified as a clayey silt of low plasticity. Measured water contents of samples of the clayey silt till deposit range from about 6 per cent to 10 per cent, near or below the plastic limit of the material.

The SPT 'N'-values measured within the clayey silt till range from 29 blows to 156 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

#### **4.2.5 Silty Sand to Sand**

A deposit of silty sand to sand was encountered below the clayey silt till in both of the boreholes advanced at this site. The surface of this deposit was encountered between about Elevation 76.5 m and 77.0 m, and both of the boreholes were terminated within this deposit after penetrating it for a thickness of 1.1 m to 4.1 m.

This deposit consists of silty sand to sand containing some gravel and trace clay. The result of a grain size distribution test performed on a selected sample of the deposit is shown on Figure B4 in Appendix B.

The measured SPT 'N'-values recorded within the silty sand to sand deposit range from 72 blows to 120 blows per 0.3 m of penetration, indicating a very dense relative density.

### **4.3 Groundwater Conditions**

The water levels were monitored in the open boreholes at the time of the field work. In addition, a piezometer was installed in Borehole H1 to monitor the groundwater level at the site. The water level measurements are shown on the borehole records contained in Appendix A and summarized as follows:

<b>Borehole No.</b>	<b>Ground Surface Elevation (m)</b>	<b>Depth to Water Level (m)</b>	<b>Groundwater Elevation (m)</b>	<b>Date</b>
H1	83.5	6.7	76.8	March 12, 2015 (Completion of drilling)
H2	82.5	4.0	78.5	March 12, 2015 (Completion of drilling)

The water levels observed in the open boreholes on completion of drilling may not represent the long-term stabilized groundwater levels. An additional groundwater level measurement will be obtained in the piezometer in Borehole H1 prior to finalizing this report. The groundwater level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and following periods of precipitation and snow melt.





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### 5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Ms. Maridalia Guerrero Pena, M.Sc., and was reviewed by Ms. Nikol Kochmanová, P.Eng. Ms. Lisa Coyne, P.Eng., a Principal with Golder and a Designated MTO Foundations Contact, conducted an independent review of this report.

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# **PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
HARMONY ROAD UNDERPASS  
HIGHWAY IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD  
REGIONAL MUNICIPALITY OF DURHAM  
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## **6.0 DISCUSSION AND PRELIMINARY GEOTECHNICAL ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides preliminary foundation recommendations in support of the proposed new Harmony Road underpass at Highway 401 and its associated wingwalls and retaining walls. These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the preliminary subsurface investigation at this site. This Preliminary Foundation Design Report, including the interpretations and recommendations contained herein, are intended for the use of MTO to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. This Preliminary Foundation Design Report shall not be used or relied upon for any other purpose or by any other parties, including contractors. Further investigation and design will be required during the detailed design stage.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the contract documents. Contractors must make their own interpretation of the factual information provided in the Preliminary Foundation Investigation Report, as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

### **6.2 Foundation Options**

It is understood that as part of the future improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, a new underpass will be constructed to carry Harmony Road over Highway 401. Based on the preliminary General Arrangement drawing provided by AECOM, the proposed underpass is to consist of a two-span structure with a total length of approximately 97 m. Highway 401 will be widened from the existing six lanes to ten lanes, plus two additional lanes, one eastbound and one westbound, to accommodate the off-ramps. The existing Highway 401 grade will be maintained, while Harmony Road will be constructed on approach embankments that are up to approximately 8 m in height at the abutments.

Both shallow and deep foundation options have been considered for support of the abutments and pier for the proposed Harmony Road underpass. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip footings founded on the till deposit:** Shallow footings are feasible for support of the abutments and pier, provided that they are extended through the stiff portion of the silty clay deposit, to found on the very stiff silty clay or very stiff to hard till deposit. This foundation type would preclude the use of integral abutments, but would permit the use of semi-integral abutments. Based on the preliminary borehole investigation, this option would require excavation to a depth of about 1.2 m in the vicinity of the north and south abutments. Based on interpolation between the boreholes, it is anticipated that excavation could extend to a depth of 1.5 m to 2.5 m, depending on the thickness of stiff silty clay and surface elevation of the very stiff silty clay or very stiff to hard clayey silt till at the pier location. Temporary protection systems would likely be required along Highway 401 to facilitate the construction of the new abutment and pier footings. Groundwater control requirements should be limited to controlling seepage from water perched within granular fill materials, on top of the relatively lower permeability silty clay and clayey silt till deposits.



- **Footings “perched” on a compacted granular pad within the approach embankments:** This option is feasible, but would require a longer structure span than for a closed structure. In addition, there is potential for some settlement in the “softer” portions of the surficial silty clay deposit, which would likely require subexcavation prior to construction of the approach embankments and perched footing. As this option is less desirable than either strip footings founded on native soils or driven steel piles, it is not discussed further in this report.
- **Driven steel H-piles or pipe piles founded within the very dense sand to silty sand deposit:** Driven steel H-piles or pipe piles are feasible for support of the abutments, and would permit design of conventional abutments, semi-integral abutments (for tube piles) or integral abutments (for H-piles). The abutments may be constructed with a pile cap perched above the Highway 401 grade in a false abutment configuration, with reinforced soil system (RSS) walls. Driven steel piles are also feasible at the centre pier, and may minimize the depth of excavation required as compared to a spread footing option, subject to further investigation during detailed design. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense/hard, glacially derived deposits.
- **Caissons founded on the very dense sand and silty sand deposits:** Caissons are considered feasible for the support of the abutments; however this option would preclude integral abutment design. A perched pile cap would minimize excavation and groundwater control requirements at the new abutments. Caissons are also feasible at the piers, and may be geometrically desirable as they may be constructed in a smaller footprint than a strip footing or pile cap. This option will be more expensive than either shallow foundations or pile foundations, although fewer caisson elements would be required in comparison to the number of steel piles that would be required. If caissons are adopted for support of the abutments or piers, they would extend into and through water-bearing cohesionless soil deposits; temporary liners would be required during construction to control potential ground losses and/or disturbance at the caisson base.

Based on the above considerations, both shallow and deep foundation options are considered feasible for the support of the new abutments. Typically at the abutments, pile foundations would be preferred with a perched pile cap in a false abutment configuration, to permit integral abutments and minimize excavation and groundwater control requirements. At the piers, shallow foundations are preferred from a geotechnical/foundations perspective due to the presence of a suitable bearing stratum at a relatively shallow depth (subject to borehole investigation at the centre pier in detailed design). However, caisson foundations may also be adopted at the piers; although more expensive, they may occupy a smaller footprint than other foundation types.

## **6.3 Shallow Foundations**

### **6.3.1 Founding Elevation and Frost Protection Requirements**

For support of the abutments, centre pier and associated wingwalls/retaining walls for the proposed new underpass, strip footings should be founded below the stiff portion of the surficial silty clay deposit, on the very stiff silty clay or very stiff to hard clayey silt till deposit. Strip or spread footings should be founded at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*). If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.



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The Highway 401 grade is proposed to be maintained at approximately Elevation of 82.5 m to 83.5 m below the proposed underpass structure, with some additional fill placed in the southern widening of the highway to raise the grade to approximately Elevation 84 m based on the crossfall requirements. The maximum (highest) founding elevations recommended for the preliminary design of the footings is summarized in the following table.

Foundation Element	Maximum (Highest) Founding Elevation (m)*	Approximate Depth of Excavation (m)*
North abutment	82.5	1+
Centre pier	81.0	2.5
South abutment	81.0	1+

\* Founding elevation selected to extend below stiff soils; depth must be checked to satisfy frost protection requirements.

Due to the variability encountered between Boreholes H1 and H2 at the north and south abutments, the subsurface conditions at the centre pier location will need to be investigated and confirmed during the detail design stage.

### 6.3.2 Geotechnical Axial Resistance and Reaction

The following factored geotechnical axial resistances at Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for preliminary design of strip footing founded on the properly prepared generally hard clayey silt till deposit. These values are based on a 3 m wide footing.

Foundation Alternative	Factored Geotechnical Axial Resistance at ULS (kPa)	Geotechnical Reaction at SLS for 25 mm of Settlement (kPa)
Footing on properly prepared very stiff silty clay or very stiff to hard clayey silt till	600	400

These preliminary geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above. These geotechnical resistances are given for loads that will be applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

## 6.4 Driven Steel H-Pile or Pipe Pile Foundations

### 6.4.1 Founding Elevation

The abutments and associated wingwalls for the proposed structure may be supported on steel piles driven to found within the very dense (“100-blow”) sand to silty sand deposit. For preliminary design purposes, if integral abutments were to be adopted for the design of the new proposed structure, it has been assumed that the pile caps would be “perched” within the embankments.



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The following pile tip elevations are recommended for preliminary design, assuming about 3 m of penetration into “100-blow” soil. Additional, deeper borehole investigation is recommended at the abutments and pier (if piles are adopted at that location) at the detail design stage to confirm these pile tip elevations.

Foundation Element	Estimated Pile Cap Elevation (m)	Approximate Surface Elevation of “100-Blow” Soil (m)	Estimated Design Tip Elevation (m)	Approximate Pile Length (m)
North Abutment	87.5	77.3	74.0	13
Centre Pier	82.0	73.5	70.5	11.5
South Abutment	88	73.4	70.5	17.5

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the glacially-derived soils at this site, as well as the potential for damage to the pile tips during seating on the very dense sand and silty sand deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of experiencing refusal on boulders or being deflected away from the vertical/battered orientation during installation due to their larger end area. Piles should be reinforced at the tip with driving shoes and/or flange plates in accordance with OPSD 3000.100 (*Steel H-Pile Driving Shoe*) or OPSD 3001.100 (*Steel Tube Pile Driving Shoe*) Type II, as appropriate, to reduce the potential for damage to the piles during driving. In very dense strata containing cobbles and/or boulders, as encountered at this site, driving shoes (such as Titus Standard ‘H’ Bearing Pile Points) are preferred over flange plates.

### 6.4.2 Geotechnical Axial Resistance/Reaction

For HP 310x110 piles driven to the design tip elevations given above, the factored axial geotechnical resistance at ULS and axial geotechnical resistance at SLS (for 25 mm of settlement) may be taken as follows:

Foundation Element	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Resistance at SLS (kN)
Abutments	1,400	1,200
Centre Pier	1,200	1,000

The same axial resistances may be used in the preliminary design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.).

Pile installation should be in accordance with OPSS 903 (*Deep Foundations*). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO’s Standard Drawing SS103-11, *Pile Driving Control*) during the final stages of driving to verify that the required ultimate capacity has been achieved.



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The preliminary geotechnical resistances/reactions provided above will have to be re-evaluated and modified, as necessary, during detail design in consideration of additional subsurface investigation at the foundation elements.

### 6.5 Caisson Foundations

#### 6.5.1 Founding Elevation

Caissons founded within the very dense sand to silty sand deposit may be considered for support of the abutments and centre pier. If adopted, the following caisson founding elevations may be used for preliminary design purposes, assuming about 3 m of penetration into "100-blow" soil:

Foundation Element	Estimated Pile Cap Elevation (m)	Approximate Surface Elevation of "100-Blow" Soil (m)	Estimated Design Base Elevation (m)
North Abutment	87.5	77.3	74.0
Centre Pier	82.0	73.5	70.5
South Abutment	88.0	73.4	70.5

The design base elevation for the centre pier should be taken as preliminary, and further investigation will be required during detail design if caisson foundations are adopted for support of the pier.

If caisson foundations are adopted, a temporary liner and a head of water/drilling slurry will be required to support the overburden soils during construction and balance groundwater pressures to minimize disturbance to the side walls and to control base disturbance/basal heave. In addition, placement of concrete by tremie methods would be required.

#### 6.5.2 Geotechnical Axial Resistance/Reaction

The following factored geotechnical axial resistance at ULS and geotechnical resistance at SLS (for 25 mm of settlement) may be used for preliminary design of caisson foundations:

Caisson Diameter (m)	Factored Geotechnical Axial Resistance at ULS (kN)	Geotechnical Reaction at SLS for 25 mm of Settlement (kN)
1.2	4,500	3,500
1.5	6,500	5,500

The preliminary geotechnical resistances/reactions provided above will need to be re-evaluated and modified, as necessary, during detail design in consideration of additional subsurface investigation at the foundation elements. Lower geotechnical resistance values may apply at the piers, depending on the depth to the 100-blow soil layer.





## **6.6 Retained Soil System (RSS) Walls**

### **6.6.1 Founding Elevations**

If perched pile caps are used in a false abutment configuration, and for retaining walls adjacent to the abutments and wingwalls at this site, retained soil system (RSS) walls are a suitable and feasible alternative to conventional concrete retaining walls supported on shallow foundations. It has been assumed that such walls would be constructed parallel to Harmony Road.

Typically, the front facing panels are supported on a footing and/or granular levelling pad at a minimum depth of 0.5 m below the lowest surrounding grade, in accordance with MTO's *RSS Design Guidelines*. Per this guideline, the levelling pad should consist of a minimum thickness of 0.3 m of compacted Granular A, which should extend at least 0.5 m beyond the outside edge of both sides of the facing footing, then outward/downward at 1H:1V.

However, for this site, to improve the performance of the settlement performance of the RSS wall, it is recommended that the front facing panels and the reinforced soil mass either be founded below the existing topsoil, fill and the stiff portion of the surficial silty clay deposit, or be constructed after subexcavation of these materials. For preliminary design, the subexcavation or founding levels for the RSS walls are as follows:

- North approach area: Elevation 82.5 m
- South approach area: Elevation 81.0 m

Further investigation will be required during detailed design to confirm the founding levels and/or subexcavation requirements for the RSS walls.

### **6.6.2 Geotechnical Resistance/Reaction**

For the RSS facing panels founded on compacted granular fill or a narrow footing as described above, preliminary design may be completed based on a factored geotechnical resistance at ULS of 150 kPa, and a geotechnical reaction at SLS (for 25 mm of settlement) of 100 kPa.

Assuming that the RSS wall (assumed to be up to approximately 6 m high) acts as a unit and uses the full width of the reinforced soil mass (which can be taken as approximately 0.8 times the wall height for preliminary design), a factored geotechnical resistance at ULS of 600 kPa and a geotechnical reaction at SLS of 400 kPa (for 25 mm of settlement) may be used for preliminary design. The preliminary geotechnical resistance/reaction values should be reviewed and revised during detail design, after the RSS wall configuration and any "step" elevations are confirmed, taking into account any additional subsurface information at that time.

### **6.6.3 Global Stability of RSS Walls**

Preliminary global stability analyses have been performed for an RSS wall adjacent to the south abutment, using the commercially available program *Slide 6.0*, produced by Rocscience Inc., to check that a minimum factor of safety of 1.5 is achieved for the proposed maximum retaining wall heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed walls on this site, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):





## PRELIMINARY FOUNDATION REPORT HARMONY ROAD UNDERPASS, W.O. 10-20011

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Effective Friction Angle	Undrained Shear Strength (kPa)
Existing and new embankment fill	21	32°	-
Granular backfill in RSS wall (note reinforcing elements assumed to prevent failure through reinforced soil mass)	21	35°	-
Stiff to very stiff clayey silt to silty clay	20	28°	50
Very stiff to hard clayey silt till	21	35°	-
Very dense sand to silty sand	21	35°	-

The results of the static global stability analysis indicate that a minimum factor of safety of 1.5 is achieved for an RSS wall up to approximately 6 m in retained soil height, assuming a slope of 2H:1V in front of the wall and level ground behind it. This preliminary assessment of the global stability of the retaining walls should be reviewed and confirmed as part of the detail design, if this type of wall is adopted, based on the final geometry of the wall.

It should be noted that the internal stability of a reinforced earth structure is to be assessed by the proprietary product designer.

### 6.7 Approach Embankments

Approach embankments up to approximately 8 m in height will be required along the new Harmony Road alignment.

#### 6.7.1 Embankment Slope Construction

In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where embankment slopes are equal to or greater than 8 m in height. To reduce erosion of the slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. Consideration may also be given to the use of a curb and gutter at the road edge, an interceptor drain along the crest of the embankment slope parallel to Harmony Road, and/or armoured drainage channels on the slope face to direct surface water flow from the Harmony Road grade down to original grade.

#### 6.7.2 Global Slope Stability

Preliminary static slope stability analysis has been completed for the Harmony Road approach embankment, using the commercially available program *Slide 6.0* from Rocscience, to check that the target minimum factor of safety is achieved. A target minimum factor of safety of 1.3 is normally used in the design of slopes under static conditions. This minimum factor of safety is considered appropriate for the proposed cut slope widening on this project, considering the design requirements and the available field and laboratory testing data.

Preliminary stability analysis were completed for an 8 m high slope in short-term (undrained) and long-term (effective stress) conditions, using the parameters outlined in Section 6.6.3. The results of the static global stability analysis indicate that a minimum factor of safety of 1.3 is achieved for 8 m high side slopes oriented no steeper than 2H:1V, as shown on Figure 2. This preliminary assessment of the slope stability of the approach embankments should be reviewed and confirmed during the detail design stage based on the refined geometry, including mid-height benching if utilized, and additional subsurface information as may be available.



### 6.7.3 Settlement Under Approach Embankment Loading

Preliminary settlement assessments have been completed for the new approach embankment construction using the commercially available computer program *Settle-3D* from Rocscience, using the estimated elastic deformation moduli given in the table below, based on correlations with the SPT 'N'-values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Elastic Modulus (MPa)
Embankment fill	21	-
Stiff to very stiff silty clay	20	15 MPa
Very stiff to hard clayey silt till	21	45 MPa
Very dense sand to silty sand	21	100 MPa

Based on this preliminary assessment, the settlement of the foundation soils under an 8 m high approach embankment is estimated to be approximately 25 mm to 35 mm. The majority of this settlement is associated with the surficial silty clay layer, and this settlement is anticipated to occur during or immediately following completion of the embankment construction.

The above preliminary estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

## 6.8 Construction Considerations

The following sections identify future construction considerations that may impact the future detail design, and for which provision may be required in the contract documents produced as part of detail design.

### 6.8.1 Open-Cut Excavation and Temporary Protection Systems

The construction of the strip footings for the proposed new structure will require excavations on the order of 1.2 m at the abutment locations, but that could extend to depths of about 2.5 m below the existing Highway 401 grade at the pier location.

It is anticipated that sufficient space may be available for open-cut excavations for strip footings if adopted at the abutment locations. The existing fill and the stiff portion of the silty clay deposit are classified as Type 3 soils, according to the Occupational Health and Safety Act (OHSA). Where space permits, temporary open-cut excavations above the groundwater level should be made with side slopes no steeper than 1H:1V. All excavations must be carried out in accordance with Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (as amended).

It is anticipated that due to space constraints, a temporary protection system will be required for shallow foundation excavations within Highway 401 at the pier location. Temporary protection systems should be designed and



constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided any adjacent utilities can tolerate this magnitude of deformation.

The selection and design of the protection system will be the responsibility of the Contractor.

### **6.8.2 Groundwater Control**

The soils encountered at this site are cohesive, with a relatively low permeability. Some groundwater seepage is anticipated from water perched within granular fills on top of the cohesive deposits, or from seams/interlayers of non-cohesive soil within the native deposits. Based on the available subsurface information and the potential depth of excavation, it is anticipated that this seepage will be relatively minor, such that it can be handled by pumping from properly filtered sumps within the excavation. Further assessment of this aspect will be required as part of detailed design, based on the results of further investigation.

### **6.8.3 Subgrade Protection**

The native soils that will be exposed within the excavations at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or precipitation and ponded water. To limit the effects of this disturbance, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a minimum 28-day compressive strength of 20 MPa.

### **6.8.4 Obstructions**

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. Further observation is recommended in future investigation as part of the detailed design assignment for this site, to further assess the presence of cobbles and boulders and permit the contractor to assess the impact on foundation construction and protection system installation.

### **6.8.5 Vibration Monitoring During Pile or Caisson Installation**

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition; lower thresholds would be applicable for nearby residential and commercial facilities (between 25 mm/s and 50 mm/s). If pile driving is adopted at the abutments, or if caissons are adopted at the piers, then vibration monitoring is recommended adjacent to the abutment areas to demonstrate/confirm that vibration levels do not exceed the threshold levels.

## **6.9 Recommendations for Future Work during Detail Design**

During the detail design phase, additional investigation is recommended to confirm or assess the following:

- the subsurface conditions and geotechnical/foundation recommendations at the centre pier location;
- the subsurface conditions at greater depth below deep foundation elements at the south and north abutments,;
- the presence, thickness and strength/compressibility properties of soils within the footprint of the retaining/RSS walls, to confirm founding elevations, geotechnical resistances, global stability and settlement; and



## PRELIMINARY FOUNDATION REPORT HARMONY ROAD UNDERPASS, W.O. 10-20011

- the groundwater level for more detailed assessment of groundwater control requirements during construction.

### 7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Maridalia Guerrero Pena, M.Sc., and reviewed by Ms. Nikol Kochmanová, P.Eng. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact, conducted an independent technical and quality review of this report.

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MGP/NK/LCC/sm

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## PRELIMINARY FOUNDATION REPORT HARMONY ROAD UNDERPASS, W.O. 10-20011

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#### ASTM International

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

#### Ministry of Transportation Ontario

Drawing SS103-11 Pile Driving Control

#### Ontario Occupational Health and Safety Act

Ontario Regulation 213 Construction Projects (as amended)

#### Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

#### Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile, Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Piles, Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

#### Ontario Water Resources Act

Ontario Regulation 903 Wells (as amended)



## PRELIMINARY FOUNDATION REPORT HARMONY ROAD UNDERPASS, W.O. 10-20011

**TABLE 1 – COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES**

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Strip footings founded on native soils	<ul style="list-style-type: none"> <li>Feasible for support of abutments and centre pier</li> </ul>	<ul style="list-style-type: none"> <li>Conventional excavation and construction techniques</li> <li>Very stiff to hard soils present at reasonable depth, with good geotechnical resistance and settlement performance</li> <li>Minimal groundwater seepage and limited dewatering required</li> </ul>	<ul style="list-style-type: none"> <li>May require up to about 2.5 m of excavation at centre pier with protection systems require along Highway 401, although further investigation will be required at detailed design</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$600/m<sup>3</sup> for construction of shallow foundations</li> </ul>	<ul style="list-style-type: none"> <li>Nominal risks associated with excavation up to 2.5 m deep at centre pier location</li> </ul>
Footings perched on granular pad in approach embankments	<ul style="list-style-type: none"> <li>Feasible for support of abutments, but would likely require some subexcavation within footprint of approach embankment</li> </ul>	<ul style="list-style-type: none"> <li>Conventional excavation and construction techniques</li> </ul>	<ul style="list-style-type: none"> <li>Would require topsoil stripping and some subexcavation of firm/stiff silty clay soils within the footprint of the approach embankments</li> <li>Longer structure span required</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$600/m<sup>3</sup> for construction of shallow foundations</li> </ul>	<ul style="list-style-type: none"> <li>Some risk for settlement of the abutment footings relative to the centre pier depending on the thickness/properties of the silty clay layer under the embankment footprint</li> </ul>
Steel H-piles or pipe piles	<ul style="list-style-type: none"> <li>Feasible for support of abutments and centre pier</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction methods for H-pile or steel pipe pile foundations</li> <li>Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation and protection system requirements</li> </ul>	<ul style="list-style-type: none"> <li>Design geotechnical resistance may not be achieved if piles refuse on cobbles and boulders layers</li> <li>Large working area required for pile driving within Highway 401 median, which may make this option less attractive</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$250/m length for pile installation and \$600/m<sup>3</sup> for pile cap construction</li> </ul>	<ul style="list-style-type: none"> <li>Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving</li> <li>Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving</li> </ul>

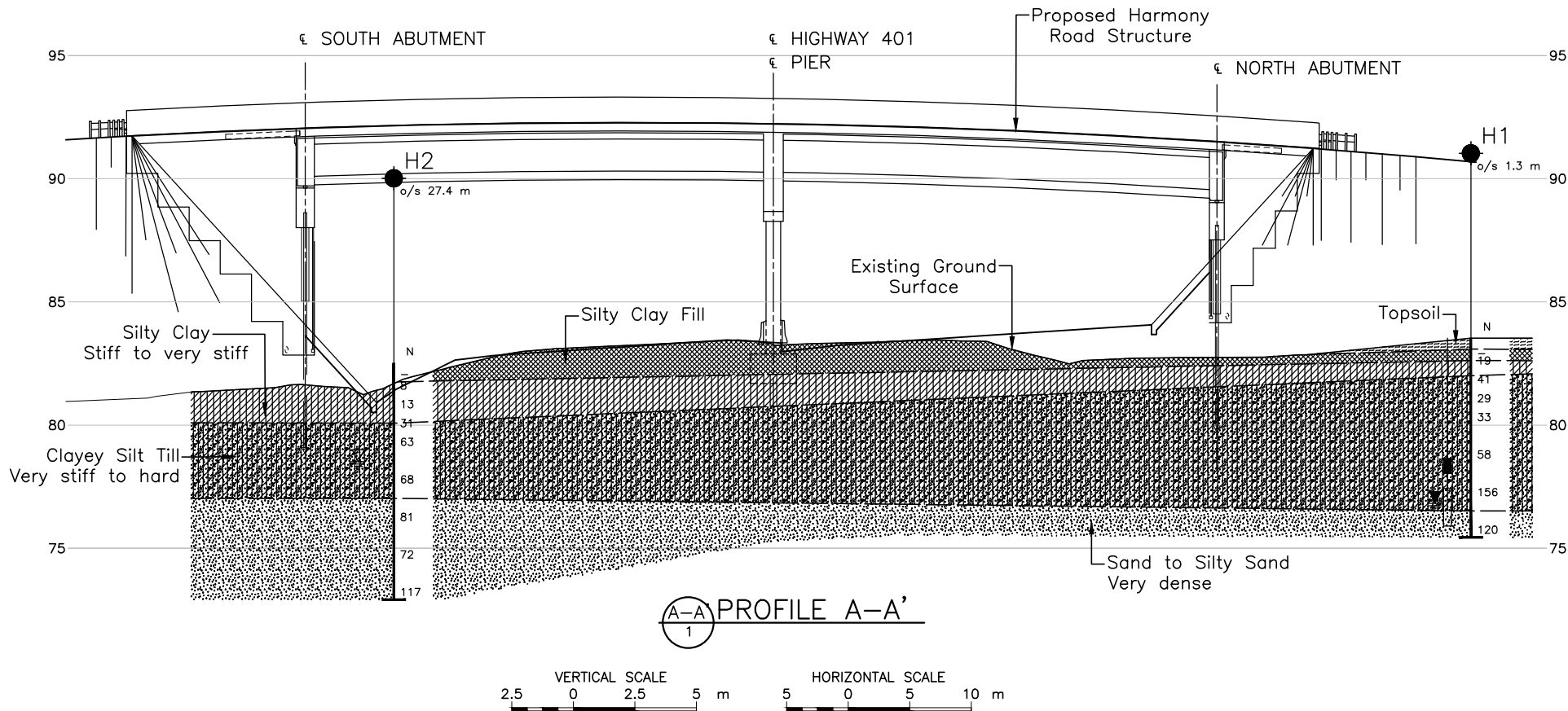
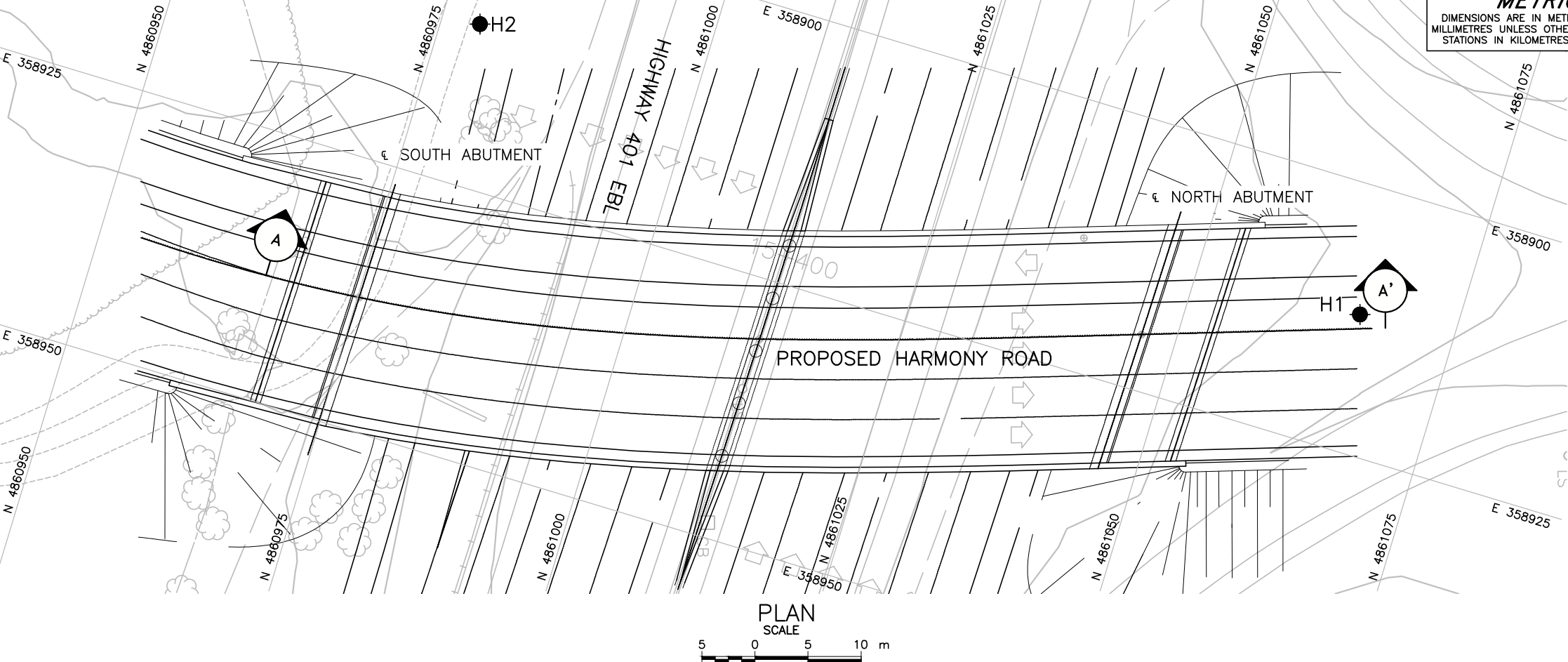


## PRELIMINARY FOUNDATION REPORT HARMONY ROAD UNDERPASS, W.O. 10-20011

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
		<ul style="list-style-type: none"> <li>Excavations would be maintained above the groundwater level at the site</li> <li>Steel H-piles allow for integral abutment configuration, and pipe piles for semi-integral abutment configuration</li> </ul>			
Caissons	<ul style="list-style-type: none"> <li>Feasible but not recommended for support of abutments</li> <li>Feasible for support of centre pier</li> </ul>	<ul style="list-style-type: none"> <li>Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation and protection system requirements</li> <li>Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles</li> </ul>	<ul style="list-style-type: none"> <li>Caissons would extend below the groundwater level at the site into water-bearing cohesionless soils, with potential for loss of ground or base disturbance</li> <li>Temporary liners would be required, plus special measures such as use of drilling mud and tremie placement of concrete; likely not possible to inspect caisson base</li> <li>Precludes use of integral abutments</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m<sup>3</sup> for pile cap construction; the cost may be higher to account for temporary liners</li> </ul>	<ul style="list-style-type: none"> <li>Risk of loosening or disturbing founding soils at base of caissons</li> </ul>

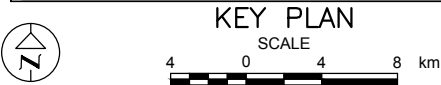
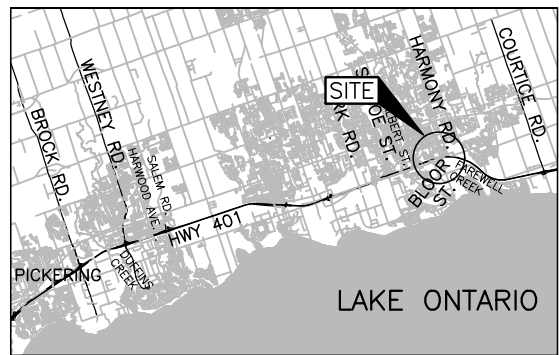


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.



CONT No. WO No. 10-20011

HARMONY ROAD UNDERPASS  
HIGHWAY 401 IMPROVEMENTS  
BOREHOLE LOCATIONS AND  
SOIL STRATA



LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- WL in piezometer, measured on March 13, 2015

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
H1	83.5	4861065.9	358912.0
H2	85.5	4860978.6	358909.8

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 Preliminary Design Report.

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

REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. ACAD-01\_GA\_Harmony Rd. Underpass.dwg



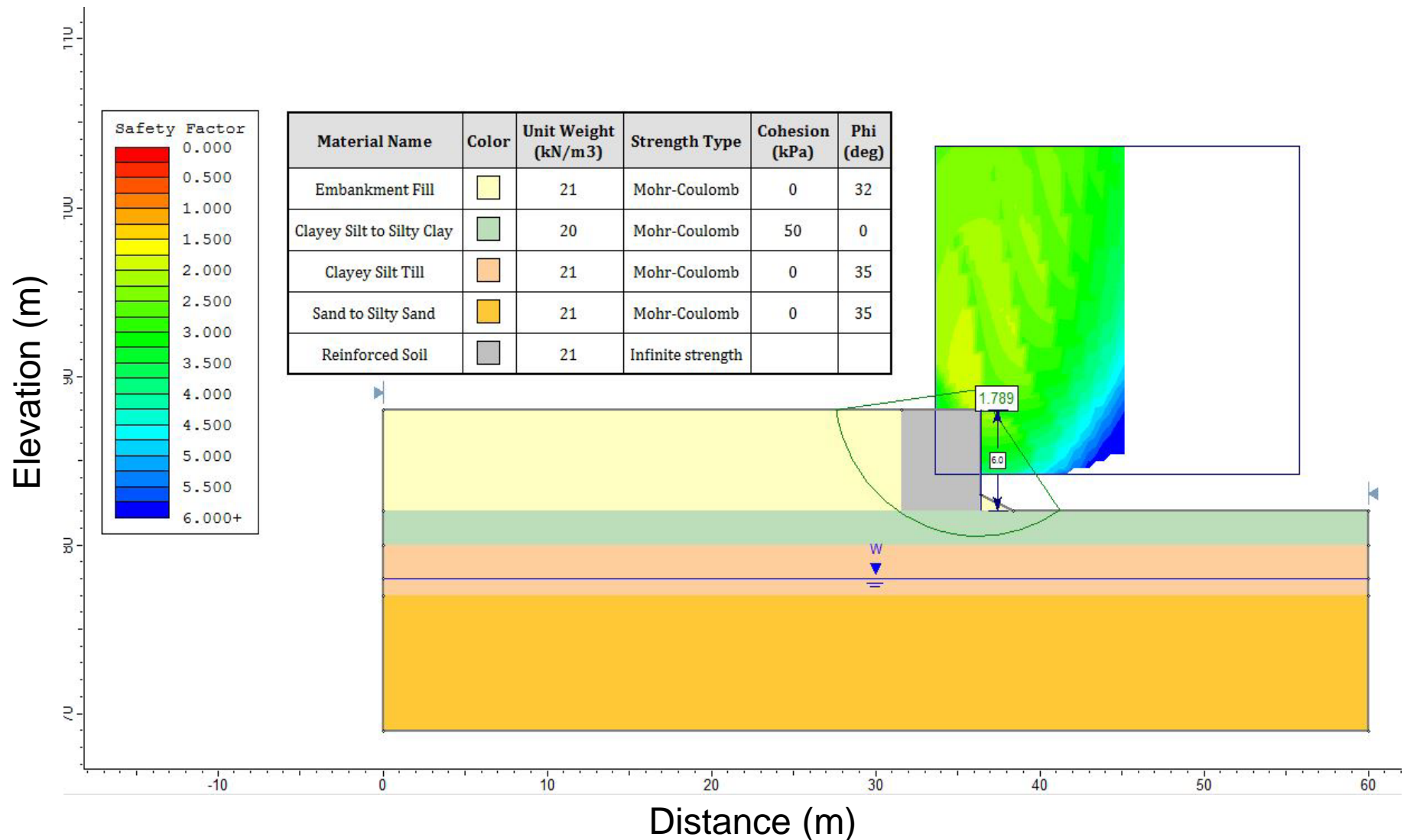
NO.	DATE	BY	REVISION
1	4/5/2017	MR/DD	1
Geocres No. 30M15-290			
HWY. 401		PROJECT NO. 11-1184-0143	
SUBM'D. PKS		DATE: 4/5/2017	
DRAWN: MR/DD		APPD. LCC	
CHKD. MGP		DIST. CENTRAL	
CHKD. NK		DWG. 1	





# STATIC GLOBAL ANALYSIS HARMONY ROAD UNDERPASS – RETAINED SOIL SYSTEM WALL

Figure 1





# **APPENDIX A**

## **Borehole Records**



## LIST OF SYMBOLS

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

<b>(a)</b>	<b>Index Properties</b>
$\rho(\gamma)$	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

### (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 = $(w_l - w_p)$
$w_s$	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

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

Notes: 1  
2

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

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



## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### II. 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.) drive open sampler for a distance of 300 mm (12 in.)

#### Dynamic Cone Penetration Resistance; $N_d$ :

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

#### Piezo-Cone Penetration Test (CPT)

A 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 ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III. SOIL DESCRIPTION

#### (a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils Consistency

	$C_u, S_u$	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	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>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
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
$SO_4$	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note:** 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

### V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

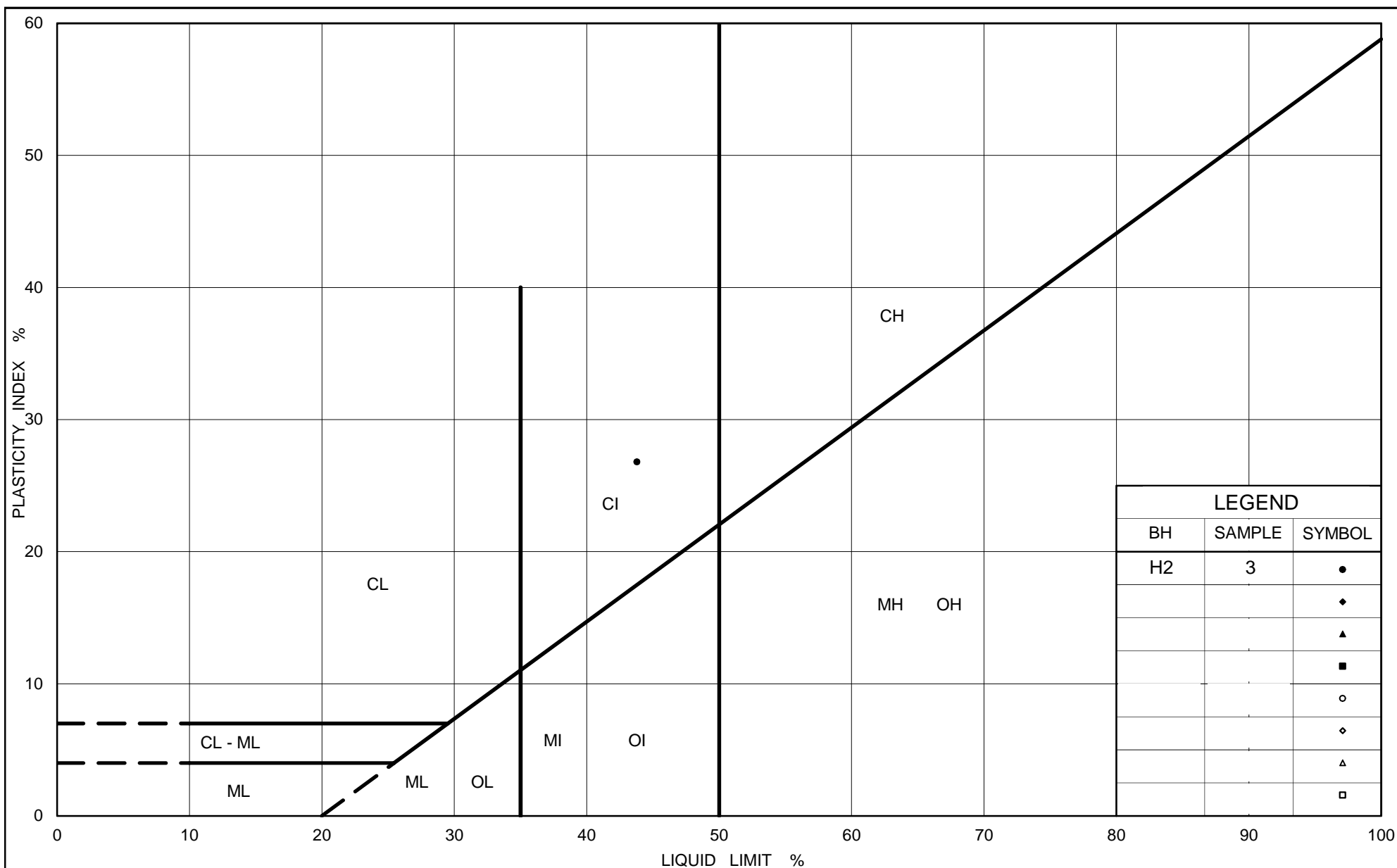
PROJECT 11-1184-0143		RECORD OF BOREHOLE No H1		SHEET 1 OF 1		METRIC															
G.W.P. 10-20011		LOCATION N 4861065.9 ; E 358912.0		ORIGINATED BY JCF																	
DIST Central HWY 401		BOREHOLE TYPE 150 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY PKS																	
DATUM Geodetic		DATE March 13, 2015		CHECKED BY LCC																	
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL		
83.5	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>p</sub> W W <sub>L</sub> 10 20 30			kN/m <sup>3</sup>					
0.0	TOPSOIL																				
83.0																					
0.5	Silty clay, trace sand, trace gravel, trace organics (FILL)		1	AS	-		83														
82.6	Light to dark brown		2	SS	19																
0.9	Moist																				
82.0	SILTY CLAY, some sand, some gravel		3A	SS	41		82						○						21 31 29 19		
1.5	Very stiff		3B	SS	41																
	Light to dark brown																				
	Moist																				
	CLAYEY SILT with sand, some gravel (TILL)		4	SS	29		81														
	Very stiff to hard																				
	Dark brown to dark grey		5	SS	33		80						○								
	Moist																				
							79														
			6	SS	58								○						20 48 26 6		
							78														
			7	SS	156		77														
76.5																					
7.0	SAND, some gravel, trace silt		8	SS	120		76						○								
	Very dense																				
	Dark grey																				
	Wet																				
75.4																					
8.1	END OF BOREHOLE																				
NOTE:																					
1. Water level in open borehole measured at a depth of 6.7 m (Elev. 76.8 m) upon completion of drilling.																					
2. Water level measured at a depth of 4.34 m (Elev. 79.2 m) below existing ground surface on August 4, 2016.																					

PROJECT 11-1184-0143		RECORD OF BOREHOLE No H2		SHEET 1 OF 1		METRIC							
W.O. 10-20011		LOCATION N 4860978.6 ; E 358909.8		ORIGINATED BY TD									
DIST Central HWY 401		BOREHOLE TYPE 150 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY PKS									
DATUM Geodetic		DATE March 12, 2015		CHECKED BY LCC									
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
82.5	GROUND SURFACE						20 40 60 80 100						
0.0	TOPSOIL						20 40 60 80 100						
0.2	Silty clay, trace to some sand, trace to some organics (FILL)		1	AS	-								
81.7	Firm												
0.8	Brown Moist		2	SS	8								
	SILTY CLAY, trace to some sand												
	Stiff to very stiff												
	Brown Moist		3	SS	13								
80.1													
2.4	CLAYEY SILT with sand, some gravel (TILL)		4A	SS	31								
	Hard		4B	SS									
	Brown Moist												
	- auger grinding on inferred cobbles/boulders between depths of 3.0 m and 6.1 m		5	SS	63								
	- becoming grey below a depth of 4.6 m		6	SS	68								
77.0													
5.5	SAND, some gravel, trace to some silt, trace clay												
	Very dense												
	Brown Moist		7	SS	81								
	- becoming wet below a depth of 6.1 m												
			8	SS	72								
73.4													
9.1	Silty SAND, trace clay, trace gravel		9	SS	117								
72.9	Very dense												
9.6	Brown Moist												
	END OF BOREHOLE												
NOTES: 1. Water level measured in borehole at a depth of 4.0 m (Elev. 78.5 m) upon completion of drilling, March 12, 2015. 2. Borehole caved to a depth of 4.6 m (Elev. 77.9 m) upon completion of drilling, March 12, 2015.													



# **APPENDIX B**

## **Geotechnical Laboratory Test Results**



# PLASTICITY CHART Silty CLAY

Figure No. B1

Project No. 11-1184-0143

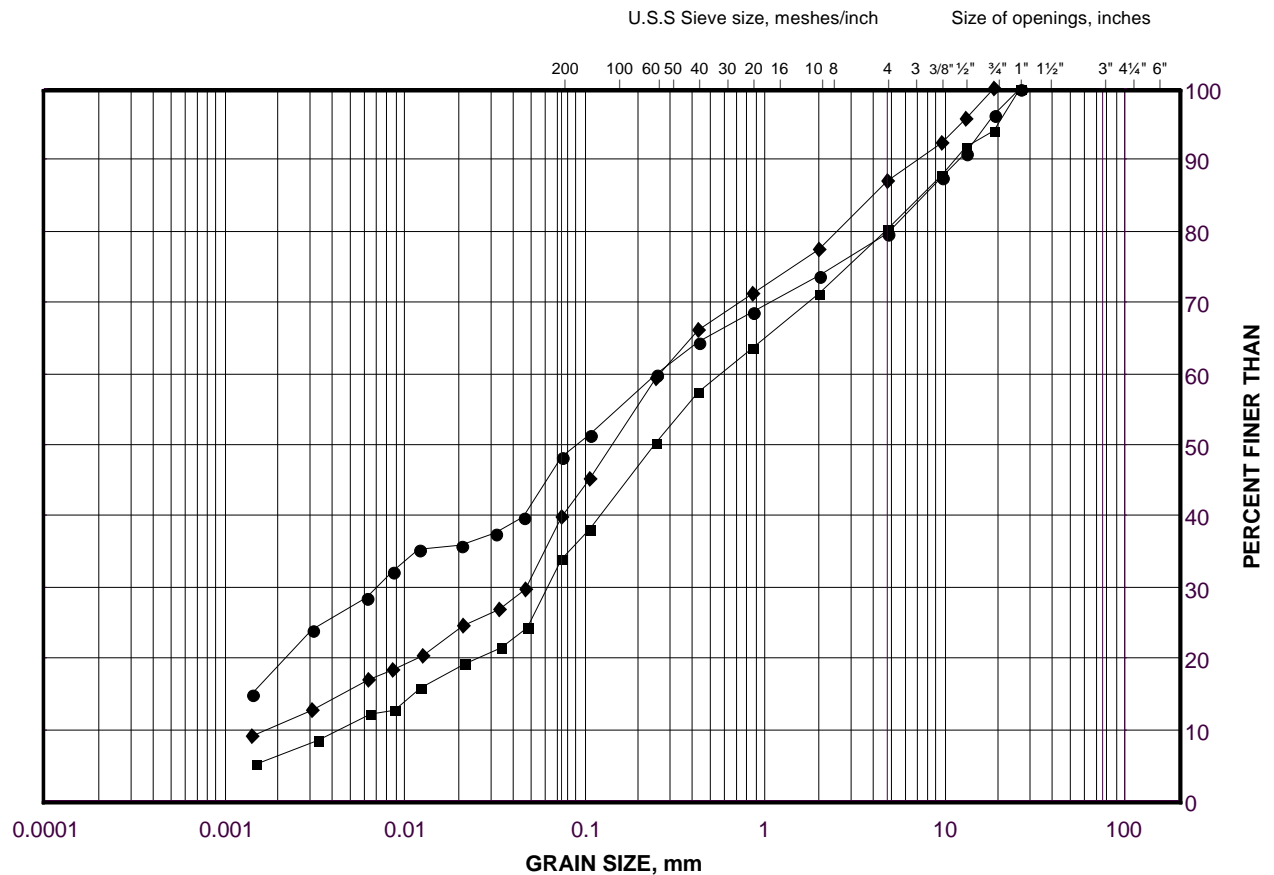
Checked By: MWK



# GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till

FIGURE B2



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

## LEGEND

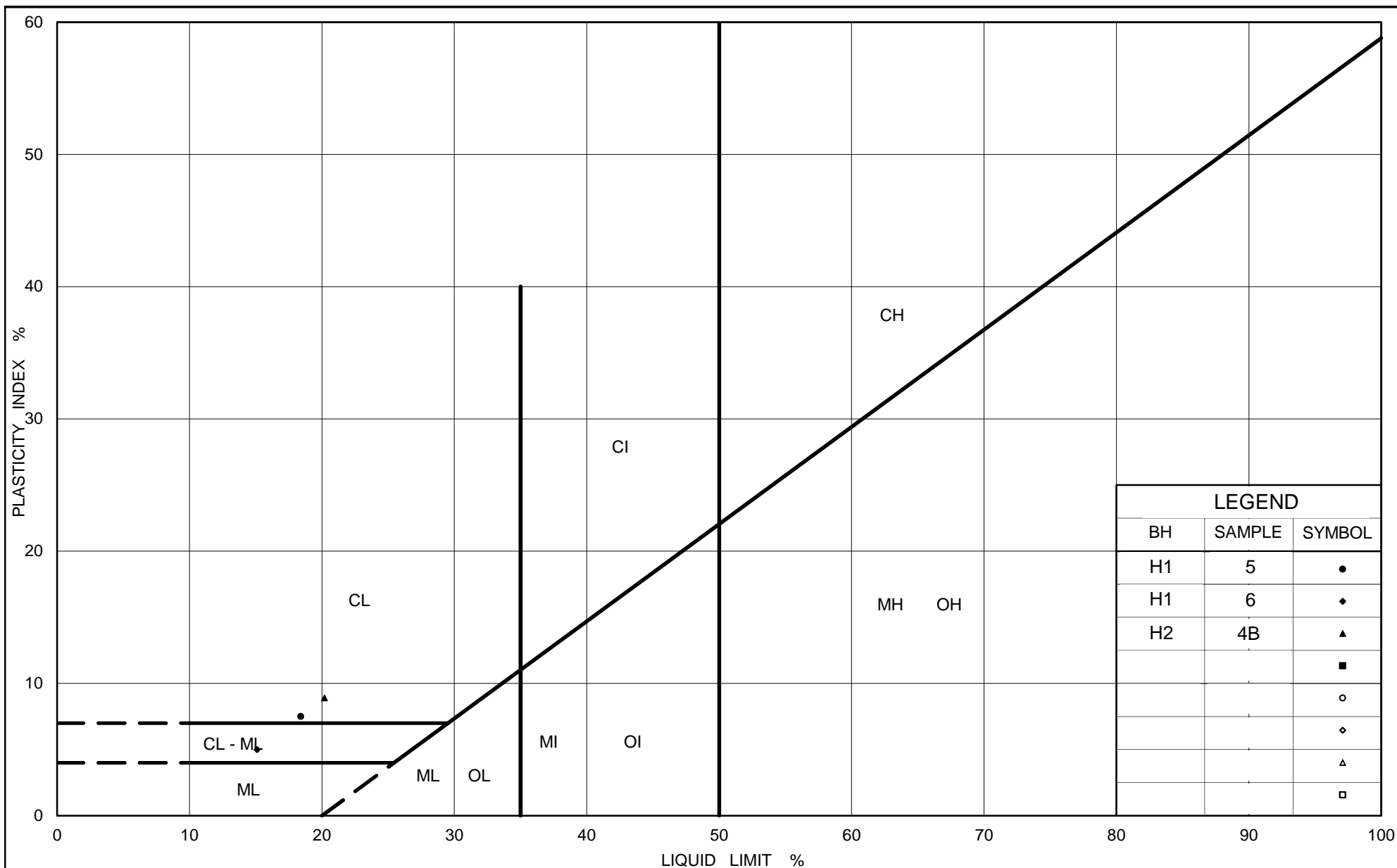
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	H1	3A	81.9
■	H1	6	78.7
◆	H2	6	77.7

Project Number: 11-1184-0143

Checked By: NK

**Golder Associates**

Date: 20-Jul-15



## PLASTICITY CHART Clayey Silt Till

Figure No. B3

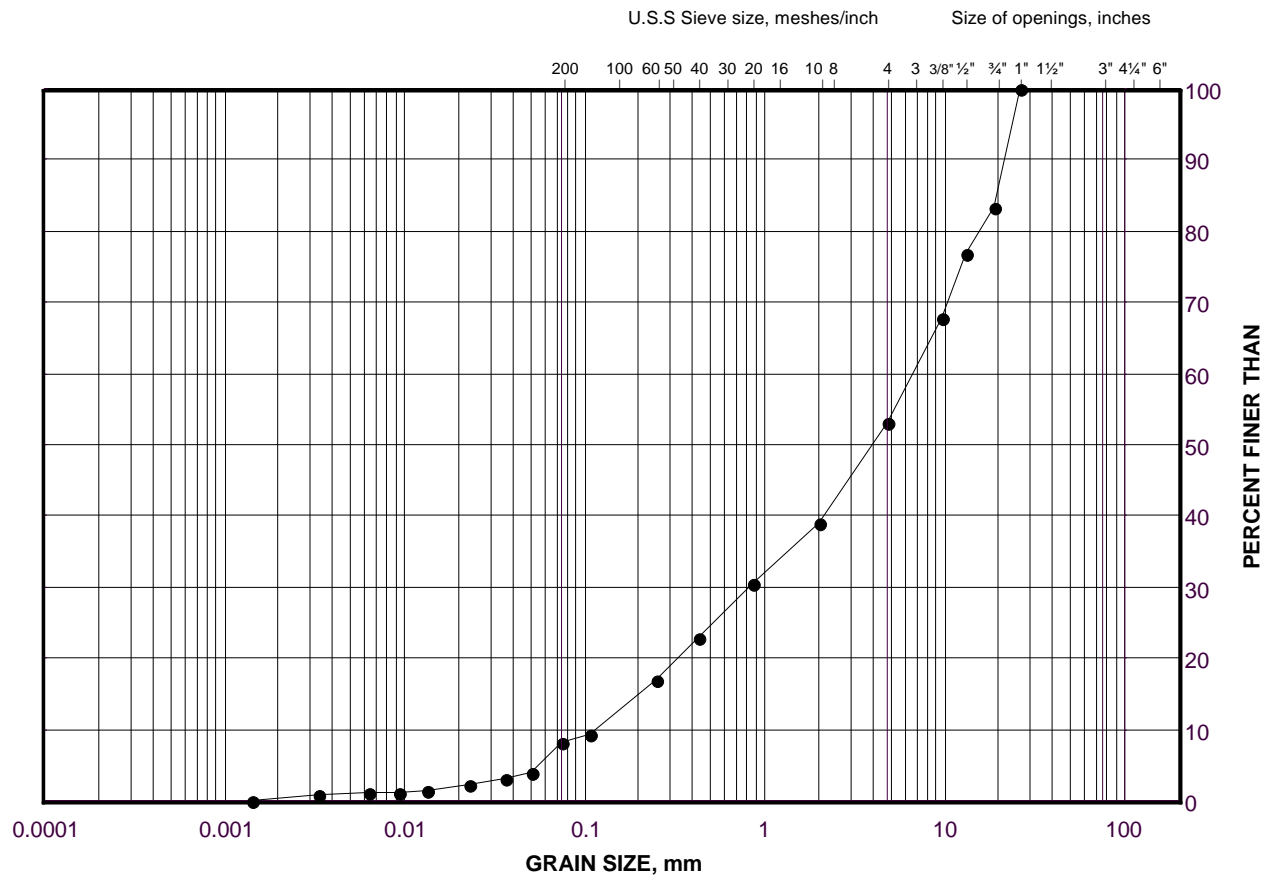
Project No. 11-1184-0143

Checked By: NK

# GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand

FIGURE B4



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	H2	8	74.8

Project Number: 11-1184-0143

Checked By: NK

**Golder Associates**

Date: 20-Jul-15

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