



November 2012

## FOUNDATION INVESTIGATION AND DESIGN REPORT

**LLOYDTOWN – AURORA ROAD UNDERPASS  
HIGHWAY 400 WIDENING FROM NORTH OF  
KING ROAD TO SOUTH CANAL ROAD  
MINISTRY OF TRANSPORTATION, ONTARIO  
G.W.P. 2835-02-00**

**Submitted to:**  
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REPORT





## Table of Contents

### **PART A - FOUNDATION INVESTIGATION REPORT**

<b>1.0 INTRODUCTION.....</b>	<b>1</b>
<b>2.0 SITE DESCRIPTION.....</b>	<b>1</b>
<b>3.0 INVESTIGATION PROCEDURES .....</b>	<b>1</b>
3.1 Previous Investigation.....	1
3.2 Current Investigation.....	2
<b>4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS .....</b>	<b>3</b>
4.1 Regional Geology .....	3
4.2 Subsurface Conditions.....	4
4.2.1 Asphalt.....	4
4.2.2 Topsoil .....	4
4.2.3 Fill .....	4
4.2.4 Sand and Silt to Silty Sand (Upper Deposit) .....	5
4.2.5 Clayey Silt to Silty Clay (Upper Deposit) .....	5
4.2.6 Clayey Silt Till .....	6
4.2.7 Sand and Silt to Silty Sand Till .....	6
4.2.8 Sand to Sand and Silt to Sandy Silt (Lower Deposit) .....	7
4.2.9 Clayey Silt (Lower Deposit).....	7
4.3 Groundwater Conditions .....	7
<b>5.0 CLOSURE.....</b>	<b>9</b>

### **PART B - FOUNDATION DESIGN REPORT**

<b>6.0 DISCUSSION AND ENGINEERING RECOMMENDATION .....</b>	<b>10</b>
6.1 General.....	10
6.2 Foundation Options .....	10
6.2.1 Strip or Spread Footing on Native Soil.....	12
6.2.1.1 Founding Elevations .....	12
6.2.1.2 Geotechnical Resistances .....	12
6.2.1.3 Resistance to Lateral Loads .....	13



## FOUNDATION REPORT – LLOYDTOWN – AURORA ROAD UNDERPASS – HIGHWAY 400 WIDENING G.W.P. 2835-02-00

6.2.1.4	Frost Protection .....	13
6.2.2	“Perched” Strip or Spread Footing .....	13
6.2.2.1	Founding Elevation .....	13
6.2.2.2	Geotechnical Resistances .....	14
6.2.2.3	Resistance to Lateral Loads .....	14
6.2.2.4	Frost Protection .....	14
6.2.3	Driven Steel H-Piles or Tube Piles .....	14
6.2.3.1	Pile Founding Elevation .....	15
6.2.3.2	Geotechnical Axial Resistances .....	15
6.2.3.3	Resistance to Lateral Loads .....	16
6.2.3.4	Frost Protection .....	18
6.2.4	Caissons .....	18
6.2.4.1	Founding Elevation .....	19
6.2.4.2	Geotechnical Axial Resistances .....	19
6.2.4.3	Resistance to Lateral Loads .....	19
6.2.4.4	Frost Protection .....	19
6.3	Lateral Earth Pressures for Design .....	20
6.3.1	Seismic Considerations .....	21
6.3.1.1	Site Coefficient .....	21
6.3.1.2	Seismic Analysis Coefficient .....	21
6.4	Retained Soil System (RSS) Walls .....	21
6.4.1	Founding Elevations .....	21
6.4.2	Global Stability .....	22
6.4.3	Geotechnical Resistances .....	23
6.4.4	Resistance to Lateral Loads .....	24
6.5	Approach Embankments .....	24
6.5.1	Subgrade Preparation and Embankment Construction .....	24
6.5.2	Approach Embankment Stability .....	25
6.5.3	Approach Embankment Settlement .....	26
6.5.3.1	Settlement of New Embankment Fill .....	27
6.5.3.2	Mitigation of Settlement .....	27



## FOUNDATION REPORT – LLOYDTOWN – AURORA ROAD UNDERPASS – HIGHWAY 400 WIDENING G.W.P. 2835-02-00

6.6	Construction Considerations.....	27
6.6.1	Open-Cut Excavation.....	27
6.6.2	Temporary Roadway Protection.....	28
6.6.3	Groundwater Control.....	28
6.6.4	Subgrade Protection .....	28
6.6.5	Ground and Groundwater Control for Caissons Installation .....	28
6.6.6	Obstructions During Pile Driving / Caisson Installation .....	28
7.0	CLOSURE.....	30

### REFERENCES

#### TABLES

Table 1	Foundation Alternatives for Lloydtown - Aurora Road Underpass
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#### DRAWINGS

Drawing 1	Highway 400, Lloydtown - Aurora Road Underpass – Borehole Locations and Soil Strata
Drawing 2	Highway 400, Lloydtown - Aurora Road Underpass – Soil Strata

#### FIGURES

Figure 1	RSS Wall Static Global Stability Analysis – 8.3 m High Wall
Figure 2	RSS Wall Static Global Stability Analysis – 4.1 m High Wall
Figure 3	RSS Wall Static Global Stability Analysis – 2.0 m High Wall
Figure 4	RSS Wall Seismic Stability Analysis – 8.3 m High Wall
Figure 5	RSS Wall Seismic Global Stability Analysis – 4.1 m High Wall
Figure 6	RSS Wall Seismic Global Stability Analysis – 2.0 m High Wall
Figure 7	East Approach Embankment – Static Global Stability Analysis
Figure 8	East Approach Embankment – Seismic Global Stability Analysis

### APPENDICES

#### APPENDIX A

Record of Borehole Sheets and Laboratory Test Results

List of Symbols and Abbreviations

Record of Borehole Sheets LA1 to LA6

Figure 1	Plasticity Chart – Clayey Silt (Fill)
Figure 2	Grain Size Distribution – Clayey Silt (Fill)
Figure 3	Grain Size Distribution – Sand and Silt to Silty Gravelly Sand (Fill)
Figure 4	Plasticity Chart – Clayey Silt (Upper Deposit)
Figure 5	Plasticity Chart – Clayey Silt (Till)
Figure 6	Grain Size Distribution – Clayey Silt (Till)
Figure 7	Plasticity Chart – Sand and Silt (Till)
Figure 8A & 8B	Grain Size Distribution – Sand and Silt Till to Silty Sand (Till)
Figure 9	Grain Size Distribution – Sand to Sand and Silt (Lower Deposit)
Figure 10	Plasticity Chart – Clayey Silt (Lower Deposit)
Figure 11	Grain Size Distribution – Clayey Silt (Lower Deposit)



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## FOUNDATION REPORT – LLOYDTOWN – AURORA ROAD UNDERPASS – HIGHWAY 400 WIDENING G.W.P. 2835-02-00

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### **APPENDIX B**

Record of Boreholes 87 and 88 and Figures 1 and 2, Golder Associates Ltd. Report No. 001-1122F, dated May 2001

### **APPENDIX C**

Non-Standard Special Provisions



# **PART A**

**FOUNDATION INVESTIGATION REPORT  
LLOYDTOWN – AURORA ROAD UNDERPASS  
HIGHWAY 400 WIDENING FROM NORTH OF  
KING ROAD TO SOUTH CANAL ROAD  
G.W.P. 2835-02-00**



## **1.0 INTRODUCTION**

Golder Associated Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the widening and replacement of the Lloydtown - Aurora Road underpass structure. The proposed work is part of the overall widening of Highway 400 from north of King Road to South Canal Road in the Regional Municipality of York, Ontario, including replacement of the 16<sup>th</sup> Side Road, Highway 9 and the southbound and northbound South Canal Bridges, culvert extensions and replacements, retaining walls, and the widening of high fill embankments and deep cuts.

The Terms of Reference for the foundation engineering services are outlined in the Terms of Reference of MTO's Request for Proposal, dated May 2008 that form part of the Consultant's Agreement (Number 2007-E-0002) for this project. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for this project, dated October 2010.

This report addresses the investigation carried out for the Lloydtown - Aurora Road underpass and its associated approach embankments. The purpose of this investigation is to establish the subsurface conditions at the location of the proposed replacement structure, including the associated approach embankments, by borehole drilling and laboratory testing on selected samples.

## **2.0 SITE DESCRIPTION**

The Lloydtown - Aurora Road underpass structure is located at the intersection of Highway 400 and Lloydtown - Aurora Road in the Regional Municipality of York, Ontario. The existing structure consists of an approximately 34 m long, 15 m wide single-span bridge, with the abutments supported on spread footings.

In general, the topography in the area of the overall project site consists of rolling terrain covered by agricultural fields and densely treed areas, with commercial facilities located along Highway 400. The existing natural ground surface at the Lloydtown - Aurora Road site is at approximately Elevation 304.5 m. The existing Highway 400 grade is slightly above this level, at about Elevation 304.8 m to 305.0 m in the immediate vicinity of the underpass.

Lloydtown - Aurora Road has been constructed on embankment fill that is between approximately 6 m and 7 m high, with the pavement grade at about Elevation 310.8 m to 311.0 m.

## **3.0 INVESTIGATION PROCEDURES**

### **3.1 Previous Investigation**

During the preliminary foundation investigation phase of the work two boreholes (Borehole Nos. 87 and 88) were advanced in October 2000 within the vicinity of the structure. The results of this investigation are presented in Golder's Preliminary Foundation Investigation Report (Report No. 001-1122F-7) dated May 2001, and a copy of the borehole records are presented in Appendix B. Reference to the subsurface conditions at these two borehole locations is made in the following sections of this report to augment the subsurface information gathered during the detail foundation investigation phase of the work.



### **3.2 Current Investigation**

The field work for the detail foundation investigation was carried out in October and November 2010, during which time a total of six boreholes (designated Boreholes LA1 to LA6) were advanced at the bridge site. One borehole was drilled near the proposed west abutment; two boreholes were drilled near the proposed centre pier location; one borehole was advanced near the east abutment; and one borehole was drilled at the toe of both the east and west approach embankments. The boreholes for the previous and current foundation investigation were advanced at the locations shown in plan on Drawing 1.

The field investigation was carried out using a D-90 track-mounted drill rig, supplied and operated by Walker Drilling Inc. of Utopia, Ontario. The boreholes were advanced using 210 mm outside diameter continuous flight hollow stem augers and 108 mm outside diameter continuous flight solid stem augers with wash boring as required. Soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586-08a). All the boreholes at the site were advanced into a stratum of equivalent SPT “N”-values equal to or greater than “100-blow” when corrected for the higher energy automatic hammer used during the current investigation. In general, the depths of the boreholes range from about 6.6 m to 18.9 m below existing ground surface, and are summarized below.

The groundwater conditions in the open boreholes were observed during the drilling operations and one piezometer was installed in Borehole LA2 to permit monitoring of the water level at this location. The groundwater conditions were also observed in the piezometers installed in Boreholes 87 and 88 during the previous (2000) investigation. The piezometer installed in Borehole LA2 consists of 50 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the screen sand pack was backfilled to the ground surface with bentonite pellets/grout. Piezometer installation details and water level readings are described on the Record of Borehole sheets presented following the text of the report. All boreholes in which standpipe piezometers were not installed were backfilled to ground surface with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372).

The field work was observed by members of Golder’s engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, directed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples.

The borehole locations and the ground surface elevations were surveyed by Callon Dietz, a professional surveying company retained by URS. The borehole locations, including the boreholes advanced during the preliminary field investigation, in MTM NAD 83 northing and easting coordinates, and the ground surface elevations referenced to geodetic datum, are summarized below and are shown on Drawing 1.





## FOUNDATION REPORT – LLOYDTOWN – AURORA ROAD UNDERPASS – HIGHWAY 400 WIDENING G.W.P. 2835-02-00

Borehole	Location (MTM NAD 83)		Ground Surface Elevation (m)	Depth Drilled (m)
	Northing (m)	Easting (m)		
LA1	4873466.0	297858.4	304.4	6.6
LA2	4873500.8	297864.6	310.8	18.9
LA3	4873501.2	297910.1	305.0	9.7
LA4	4873526.5	297904.2	304.8	17.4
LA5	4873525.8	297942.3	311.0	18.7
LA6	4873563.2	297951.0	304.3	6.7

The location and elevation of Boreholes 87 and 88 which were advanced as part of the preliminary study in October 2000, are presented below.

Borehole	Location (MTM NAD 83)		Ground Surface Elevation (m)	Depth Drilled (m)
	Northing (m)	Easting (m)		
87	4873552	297915	305.0	9.6
88	4873483	297887	305.0	9.6

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

This 23 km section of Highway 400 traverses, in a south–north direction, the physiographic regions known as South Slope, Oak Ridges Moraine and Simcoe Lowland, according to *The Physiography of Southern Ontario* (Chapman and Putman, 1984)<sup>1</sup>. Along Highway 400, the South Slope is present south of King Road, the Oak Ridge Moraines extends from north of King Road to south of Highway 9 and the Simcoe Lowlands occupy a 4 km wide strip extending from south of Highway 9 to Holland River. The Lloydtown - Aurora Road underpass structure is located within the Oak Ridges Moraine physiographic region.

The surficial soils of the South Slope region are generally cohesive tills. The Oak Ridges Moraine predominately consists of sand and gravel, although in the King Township area these soils are often overlain by till. It is understood that during grading for the initial construction of Highway 400 in this area, deep cuts exposed up to about 10 m of till overlying the sands and gravels.

The Holland River valley, which crosses Highway 400 in the vicinity of Highway 9 and South Canal Road, is located within the Simcoe Lowlands region. This valley extends to the southwest from Cook Bay at the south end of Lake Simcoe, and was once a shallow extension of the lake. The floor of the valley consists of peat, soft clays and loose sands. It is understood that during initial construction of Highway 400, a layer of peat about 2 m to 3 m thick was removed in order to construct the road upon the underlying sand and clay.

<sup>1</sup> Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000



A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

## **4.2 Subsurface Conditions**

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced for the detail foundation investigation together with results of the laboratory tests carried out on selected soil samples are provided on the Record of Borehole sheets in Appendix A. The Record of Boreholes 87 and 88 sheets are presented in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The interpreted stratigraphy in profile along Lloydtown - Aurora Road and in cross section at the abutment and pier locations is shown on Drawings 1 and 2 and is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, the subsurface conditions in the area consist of topsoil/asphalt and fill on the roadway and highway alignments, underlain in places by clayey silt or sand and silt. Till deposits are encountered below the fill or clayey silt/sand and silt deposits, underlain by a sand to sand and silt to sandy silt deposit, which in turn is underlain in places by a clayey silt deposit.

### **4.2.1 Asphalt**

A layer of asphalt about 0.1 m thick was encountered in Boreholes LA2 and LA5 advanced on the existing grade of Lloydtown - Aurora Road. In Boreholes LA3 and LA4, advanced through the inside shoulder of the northbound lane of Highway 400, the asphalt layer is about 0.4 m thick.

### **4.2.2 Topsoil**

A layer of topsoil about 0.2 m thick was encountered at the existing ground surface in Boreholes LA1, LA6, 87 and 88.

### **4.2.3 Fill**

At all boreholes drilled for this structure, with the exception of Borehole 88, fill was encountered underlying the topsoil or asphalt. In Boreholes LA2 and LA5 which were drilled along Lloydtown - Aurora Road through the existing fill embankment, the fill extends to depths of about 7.2 m and 6.6 m below ground surface (Elevation 303.6 m and 304.4 m), respectively. At the other borehole locations the fill extends to depths between about 0.6 m and 3.2 m below ground surface (Elevation 301.6 m and 304.4 m).

The fill material is variable in composition and thickness, and consists of cohesionless soil grading from sand and silt to silty sand to sandy silt to sand and gravel and cohesive fill consisting of clayey silt. The fill in places contains rootlets and organics (i.e. Boreholes LA2 to LA6 and 87) and sand and silt pockets (Boreholes LA1, LA4 and LA5).

The SPT "N"-values measured within the cohesionless portions of the fill generally range from 5 blows to 47 blows per 0.3 m of penetration, indicating a loose to dense relative density. Two SPT "N"-values of 63 blows per 0.3 m penetration and 97 blows per 0.23 m of penetration were encountered in Borehole LA2, within the



cohesionless fill and are attributed to the presence of gravel. The SPT “N”-values measured within the cohesive fill range from 4 blows to 14 blows per 0.3 m of penetration, suggesting a firm to stiff consistency.

Atterberg limits tests were carried out on three samples of the cohesive fill material and yielded liquid limits between about 16 per cent and 33 per cent, plastic limits between about 11 per cent and 16 per cent and corresponding plasticity indices between about 5 per cent and 17 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure 1 in Appendix A and indicate that this material is clayey silt of low plasticity. Grain size distribution tests were carried out on three samples of the cohesive fill and the results are shown on Figure 2 in Appendix A.

Grain size distribution tests were carried out on five samples of the cohesionless fill and the results are shown on Figure 3 in Appendix A.

The natural water content measured on samples of the cohesionless portions of the fill deposit range from 7 per cent to 14 per cent. The natural water content measured on samples of the cohesive fill ranges from 12 per cent to 27 per cent.

#### **4.2.4 Sand and Silt to Silty Sand (Upper Deposit)**

A cohesionless deposit of sand and silt, trace clay to silty sand containing some gravel, trace clay and organics was encountered in Boreholes LA2 and 88, below the fill deposit and the topsoil layer, respectively. The top of this deposit was encountered at about Elevation 303.6 m and Elevation 304.8 m and the base of the deposit extends to Elevation 302.1 and 302.7 m in Boreholes LA2 and 88, respectively.

The SPT “N”-values measured within the upper sand and silt to silty sand deposit range from 17 blows to 40 blows per 0.3 m of penetration, indicating a compact to dense relative density.

The natural water content measured on two samples of the cohesionless deposit is 12 per cent and 13 per cent.

#### **4.2.5 Clayey Silt to Silty Clay (Upper Deposit)**

Underlying the fill material in Boreholes LA5 and 87, a clayey silt to silty clay deposit containing trace to some gravel, trace to some sand, rootlets and organics was encountered at depths of 6.6 m and 3.9 m below ground surface, respectively, corresponding to Elevation 304.4 m. The thickness of the deposit is about 2.1 m and 3.3 m in Boreholes LA5 and 87, respectively.

The SPT “N”-values measured within the upper clayey silt to silty clay deposit range from 1 blow to 19 blows per 0.3 m of penetration, suggesting a soft to very stiff consistency.

An Atterberg limits test carried on one sample of this deposit yielded a liquid limit of about 24 per cent, a plastic limit of about 18 per cent and a plasticity index of 6 per cent. The results of the Atterberg limits test are shown on the plasticity chart on Figure 4 presented in Appendix A, and indicated that this material is a clayey silt of low plasticity.

The natural water content measured on samples within this deposit ranges from 9 per cent to 24 per cent. The organic content measured on a sample of this deposit is about 2 per cent.



#### **4.2.6 Clayey Silt Till**

A till deposit comprised of clayey silt with sand and trace gravel to clayey silt containing trace to some sand and gravel was encountered in Boreholes LA1, LA2, LA6, 87 and 88, underlying the fill deposit or the upper sand and silt to silty sand deposit or the upper clayey silt deposit. The top of this deposit was encountered between about Elevation 302.9 m and Elevation 301.1 m and its thickness ranges from about 0.8 m to 4.8 m. The clayey silt till deposit was also encountered at a depth of about 6.3 m below ground surface (Elevation 298.1 m) underlying the sand and silt till deposit, in Borehole LA1. This borehole was terminated within the clayey silt till deposit at a depth of 6.6 m below ground surface (Elevation 297.8 m).

The SPT “N”-values measured within the clayey silt till deposit range from 13 blows to 179 blows per 0.3 m of penetration, suggesting a stiff to hard consistency.

Atterberg limits tests were carried out on five samples of this deposit and yielded liquid limits between about 16 per cent and 31 per cent, plastic limits between about 9 per cent and 15 per cent and plasticity indices between about 5 per cent and 16 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure 5 in Appendix A and indicate that the material is a clayey silt of low plasticity.

Grain size distribution tests were carried out on two samples of this deposit and the results are shown on Figure 6 in Appendix A.

The natural water content measured within this deposit ranges from 8 per cent to 18 per cent.

#### **4.2.7 Sand and Silt to Silty Sand Till**

Underlying the clayey silt till deposit, upper clayey silt or the fill deposit, a till deposit comprised of sand and silt to silty sand containing trace to some gravel, trace to some clay, sand pockets and sand seams was encountered in Boreholes LA1 to LA6 and 88. The top of this granular till deposit was encountered between about Elevation 302.8 m and Elevation 300.6 m and the thicknesses of the deposit ranges from about 2.5 m to 4.8 m. Borehole LA6 was terminated within the sand and silt till deposit at a depth of about 6.7 m below ground surface (Elevation 297.5 m). In Borehole LA2, the augers were noted to be grinding on possible cobbles or boulders at depths between 11.3 m and 12.2 m below ground surface.

The SPT “N”-values measured within this till deposit range from 14 blows to 160 blows per 0.3 m of penetration, indicating a compact to very dense relative density. An SPT “N”-value of 9 blows per 0.3 m of penetration was recorded in Borehole LA2 at a depth of about 14 m below ground surface and is attributed to disturbance of the deposit due to groundwater inflow into the augers.

Atterberg limits tests were carried out on three samples of this deposit and yielded liquid limits of 15 per cent, plastic limits between about 11 per cent and 12 per cent and plasticity indices between about 3 per cent and 4 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure 7 in Appendix A and indicate that the sand and silt till exhibits a slight plasticity.

Grain size distribution tests were carried out on twelve samples within this deposit and the results are shown on Figures 8A and 8B in Appendix A.

The natural water content measured within this deposit ranges from 2 per cent to 16 per cent.



#### **4.2.8 Sand to Sand and Silt to Sandy Silt (Lower Deposit)**

Underlying the sand and silt to silty sand till deposit or the clayey silt till deposit, a cohesionless deposit comprised of sand to sand and silt to sandy silt, trace to some clay was encountered in Boreholes LA2 to LA5, 87 and 88. The top of this deposit was encountered between about Elevation 299.3 m and Elevation 296.0 m. Boreholes LA2, LA3, LA5, 88 and 87 were terminated within this deposit between about Elevation 295.5 m and Elevation 291.9 m. This deposit is 6.8 m thick where it was fully penetrated at Borehole LA4 and the base of the deposit extends to a depth of about 14.8 m below ground surface (Elevation 290.1 m).

The SPT “N”-values measured within this deposit range from 57 blows per 0.3 m of penetration to 171 blows per 0.25 m of penetration, indicating a very dense relative density.

The results of grain size distribution tests carried out on six samples of this deposit are shown on Figure 9 in Appendix A.

The natural water content measured on samples of the lower sand to sand and silt deposit ranges from 12 per cent to 21 per cent.

#### **4.2.9 Clayey Silt (Lower Deposit)**

A cohesive deposit comprised of clayey silt, containing trace to some sand and trace gravel was encountered underlying the lower sand to sand and silt deposit in Borehole LA4. The top of this deposit was encountered at about a depth of about 14.8 m below ground surface (Elevation 290.1 m) and Borehole LA4 was terminated within the deposit at a depth of about 17.4 m below ground surface (Elevation 287.5 m).

The SPT “N”-values measured within the clayey silt deposit are 72 blows and 87 blows per 0.3 m of penetration, suggesting a hard consistency.

An Atterberg limits test was carried out on one sample of this deposit and yielded a liquid limit of 24 per cent, a plastic limit of 15 per cent and plasticity index of 9 per cent. The results of the Atterberg limits test are shown on the plasticity chart on Figure 10 in Appendix A and indicate that material is a clayey silt of low plasticity.

The result of a grain size distribution test carried out on one sample of this deposit is shown on Figure 11 in Appendix A.

The natural water content measured within the lower clayey silt deposit is 14 per cent.

### **4.3 Groundwater Conditions**

Piezometers were installed in Borehole LA2 during the current investigation and in Boreholes 87 and 88 during the previous investigation to permit the monitoring of the groundwater levels at the site. In general, the overburden samples taken in the boreholes advanced in this area were moist. Details of the piezometer installation are shown on the Record of Borehole sheets presented in Appendix A and Appendix B. The water level in the piezometer in Borehole LA2 was measured on May 27, 2011 at a depth of about 1.5 m below ground surface and the tip of the water level finder contained bentonite corresponding to the depth of the top of the piezometer screen. Therefore, it is considered that the piezometer was damaged during installation. The groundwater levels measured in the piezometer installed during the previous investigation are summarized below.



## FOUNDATION REPORT – LLOYDTOWN – AURORA ROAD UNDERPASS – HIGHWAY 400 WIDENING G.W.P. 2835-02-00

Borehole No.	Ground Surface Elevation (m)	Stratum Sealed Into	Piezometer Tip Elevation (m)	Groundwater Elevation (m)	Date of Measurement
87	305.0	Clayey Silt with Sand Till/Sand	296.5	298.2	October 19, 2000
				298.7	December 20, 2000
				--*	January 19, 2001
88	305.0	Sand and Silt Till/Sand	298.0	299.8	October 20, 2000
				Dry	December 20, 2000
				Dry	January 19, 2001

\*Piezometer destroyed – unable to obtain water level

It should be noted that in Boreholes LA2 to LA5 and 88, drilling fluid was used to advance the boreholes between Elevation 298.3 m and Elevation 295.6 m due to “blowing” sands; as a result, water levels could not be determined in these boreholes upon completion of drilling.

It should also be noted that groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.



## **5.0 CLOSURE**

This Foundation Investigation Report was prepared by Ms. Olta Kociu, EIT and reviewed by Ms. Sandra McGaghran, P.Eng., a senior geotechnical engineer with Golder. Mr. Jorge Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent quality control review of the report.

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OK/TVA/SMM/JMAC/jl

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# **PART B**

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KING ROAD TO SOUTH CANAL ROAD  
G.W.P. 2835-02-00**





## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATION

### 6.1 General

This section of the report provides foundation engineering recommendations for the detail design of the Lloydtown - Aurora Road underpass structure replacement as part of Highway 400 widening from north of King Road to South Canal Road. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the structure foundations and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing Lloydtown - Aurora Road structure consists of a single-span bridge supported on shallow spread footings (shallow foundations). The replacement underpass structure is proposed to consist of two spans, each with a length of about 41.6 m. Based on the General Arrangement (GA) Drawing provided by URS on November 17, 2010, the proposed bridge deck surface varies between about Elevation 313.3 m and Elevation 313.5 m. The existing pavement surface of Lloydtown - Aurora Road in the vicinity of the proposed east and west abutments is at about Elevation 311 m. Therefore the grade of Lloydtown - Aurora Road will be raised by about 2.5 m. The existing ground surface at Highway 400 beneath the existing bridge is at about Elevation 305 m, and the ground surface at the toe of the existing west and east approach embankments is at about Elevation 304.5 m. Therefore the existing Lloydtown - Aurora Road embankment is between approximately 6 m and 7 m high, and the proposed approach embankments will be an additional 2.5 higher.

### 6.2 Foundation Options

The new bridge will consist of two spans, each approximately 41.6 m long. Within the vicinity of the foundation elements the subsurface soil conditions consist of existing fill, underlain by till deposits comprised of very stiff to hard clayey silt to silty clay and compact to very dense sand and silt, which in turn are underlain by a very dense cohesionless deposit of sand to sand and silt to sandy silt.

Shallow and deep foundations options have been considered for support of the abutments and the central pier. A summary of the advantages and the disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks/emergencies and relative costs is provided in Table 1 following the text of this report.

- **Strip or spread footings founded generally on dense to very dense sand and silt till or on very stiff to hard clayey silt till:** Strip or spread footings are considered as an alternative for the foundation elements at the west and east abutment and central pier, which allows for the usage of semi-integral abutments. Spread footings founded on the native soil would require excavation through the existing embankment fill; therefore this option may not be economical at the abutments, although it is considered feasible at the pier.



- **Strip or spread footing “perched” on a granular pad within the fill deposit:** This option could be adopted to support the abutment footings for an open structure with 2 horizontal to 1 vertical (2H:1V) front slopes. This option requires a longer span and may not be economical.
- **Steel H-piles driven to found within “100-blow” sand and silt to silty sand till or sand deposit:** Steel 310 x 110 H-piles driven to found within “100-blow” material are suitable and feasible for the support of abutments and the central pier foundation elements and allow for integral abutment construction.
- **Steel tube piles to found within the “100-blow” sand and silt to silty sand till or sand deposit:** Steel tube (pipe) piles (12 ¾ in. diameter and ¼ in. thick walls) could also be considered as a deep foundation option for support of the abutments and central pier, however, MTO does not allow the use of pipe piles for integral abutment construction. Pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation due to the presence of very dense/hard layers or cobbles and/or boulders within the glacially-derived soils at this site.
- **Caissons founded within very dense sand and silty to silty sand till or sand deposit:** Caisson foundations are suitable and feasible for the support of the proposed abutments and the central pier. For the abutment and pier foundation elements, temporary or permanent liners will be required during the installation to control groundwater, which will result in the caisson foundations being less a cost effective than the installation of driven steel H-piles. It is recommended to found the caissons above the “100-blow” material due to the artesian conditions encountered in the sand material at greater depth and therefore the geotechnical axial resistances are lower due to the shorter length of the caissons.

At the abutments, either “perched” footings or steel H-piles are preferred over spread footings founded on the native soils due the required depth of excavation to reach native soils and the resulting greater height of the abutment walls. At the central pier, spread footings would require temporary roadway protection in order to found the spread footings on the compact to dense sand and silt till deposit and are preferred if the geotechnical axial resistance available is adequate; otherwise, support of the pier on deep foundations will be required to achieve a higher capacity.

For deep foundations, the use of piles is preferred from a foundations perspective over caissons as the caissons would extend to the sand and silt till or into the underlying very dense sand material, as these deposits are water-bearing, which could present installation problems associated with water inflow at the base of the caisson during construction. Special construction procedures would be required for advancement of caissons through obstructions if encountered, as well as for control of groundwater inflow to reduce the potential for loosening of the soils at the base of the caissons.

It is noted that during the preliminary foundation investigation for this site the soil samples were obtained using a manually-operated safety hammer (i.e. rope cathead), whereas during the current foundation investigation the drill rig was equipped with an automated hammer with higher efficiency. The Standard Penetration Test (SPT) “N”-values from the current foundation investigation when corrected to 60 per cent efficiency of hammer energy transfer indicate that all the boreholes for the current foundation investigation were advanced into a stratum of equivalent SPT “N”-values equal to or greater than “100-blow”.

Recommendations for the various foundation options for the replacement of the Lloydtown - Aurora Road underpass are provided in the following sections. A summary comparison of the advantages, disadvantages and



relative costs associated with each of the feasible foundation options is presented in Table 1 following the text of this report.

## **6.2.1 Strip or Spread Footing on Native Soil**

### **6.2.1.1 Founding Elevations**

Strip or spread footings found on the hard clayey silt till or dense to very dense sand and silt till deposits is an adequate alternative as foundation elements at the abutments and central pier. The proposed finished grade of Highway 400 is between Elevation 305.0 m and Elevation 304.8 m as shown on the General Arrangement (GA) drawing, provided by URS. The proposed shallow foundations should be founded at a depth of 1.5 m below the proposed final surface grade, as a minimum requirement for the frost protection in accordance with Ontario Provincial Standard Drawing (OPSD) 3090.101-*Foundation Frost Penetration Depths for Southern Ontario*.

Due to the presence of existing embankment fill and very soft to firm clayey silt to silty clay soil below the proposed finished grade of Highway 400 from about Elevation 305.0 m to 301.6 m, and to ensure that the strip/spread footings are founded on competent soils, the following founding levels and stratum are recommended for strip or spread footings for the support of the west and east abutments and the central pier.

Foundation Element	Reference Borehole No.	Founding Stratum	Maximum Founding Elevation
West Abutment	LA1, LA2 and 88	Hard Clayey Silt Till	301.5 m
Central Pier	LA3, LA4 and 87	Dense to Very Dense Sand and Silt Till	301.0 m
East Abutment	LA5, LA6 and 87	Dense Sand and Silt Till	301.0 m

### **6.2.1.2 Geotechnical Resistances**

The following geotechnical resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) may be used for the design of a 3 m wide spread or strip footing placed on the properly prepared, undisturbed hard clayey silt till or undisturbed dense to very dense sand and silt till at or below the founding elevation as noted above.

Foundation Element	Founding Stratum	Maximum Founding Elevation	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm of settlement)
West Abutment	Hard Clayey Silt Till	301.5 m	600 kPa	350 kPa
Central Pier	Dense to Very Dense Sand and Silt Till	301.0 m	700 kPa	500 kPa
East Abutment	Dense Sand and Silt Till	301.0 m	600 kPa	350 kPa



These design values take into account the depth of footing embedment which is the depth of the footing relative to the proposed road grade of Highway 400. The geotechnical resistances should, be reviewed if the selected footing width or founding elevation differs from those given above.

The base of each footing excavation should be cleaned of loose / softened material. It is recommended that the founding level for the footings be inspected by geotechnical personnel immediately prior to pouring concrete to confirm the adequacy of the foundation conditions for the above noted geotechnical resistances. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a working slab of concrete (100 mm thick of 20 MPa compressive strength concrete) be placed in the excavation within three (3) hours to protect the integrity of the bearing stratum. This requirement can either be added as a note on the Contract Drawings or a sample NSSP is included for this item in Appendix C.

The geotechnical resistance provided above are given for the loadings that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*, using the curves for cohesive soils and non-cohesive soil.

### **6.2.1.3 Resistance to Lateral Loads**

Resistance to lateral force/sliding between the concrete footing and the subgrade should be calculated in accordance with the Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the generally hard clayey silt till and dense to very dense sand and silt till, the coefficient of friction  $\tan \Phi$ , can be taken as 0.45 and 0.51, respectively. This value is unfactored.

### **6.2.1.4 Frost Protection**

All footings should be provided with a minimum of 1.5 m of soil cover or equivalent thickness of insulation below the footings for frost protection, in accordance with Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*). As a guide, the MTO has adopted 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent to 0.3 m reduction in soil cover.

## **6.2.2 “Perched” Strip or Spread Footing**

### **6.2.2.1 Founding Elevation**

A “perched” footing founded on a Granular ‘A’ (SP110S13 – Aggregates) pad, at a depth of 1.5 m of the finished grade surface, is adequate and cost effective for these foundation elements at both abutments. For this option, sub-excavation of any existing topsoil, organic and loosened/soften material is recommended within the embankment footprint below the perched abutment, to minimize settlement due to embankment loading.

At the east abutment, up to about 6.5 m thick deposit of compact/firm to stiff sand and silt fill/clayey silt fill underlain by about 2.1 m thick stratum of very soft clayey silt was encountered, it is recommended that these deposits be sub-excavated and replaced with well compacted SP110S13 Granular ‘B’ Type II, prior to placing of the footing on Granular ‘A’ pad. The area to be sub-excavated, should be defined by a line extending from the toe of the Granular ‘A’ pad outward and downward at 1 horizontal to 1 vertical (1H:1V). The Granular ‘A’ pad should be a minimum of 2 m thick and should extend at least 1 m beyond the plan limits of the footing. The Granular ‘A’ pad should be constructed in accordance with OPSS 501 (Compacting) and SP 105S21 (Amendment to OPSS 501).



#### **6.2.2.2 Geotechnical Resistances**

Based on the above sub-excavation depths, and backfilling and pad construction procedures, a factored geotechnical resistance at ULS of 850 kPa and a geotechnical reaction at SLS of 350 kPa (for 25 mm of settlement) may be used for the detail foundation design, assuming 3 m wide footings.

The geotechnical resistances provided above are for the loadings that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *CHBDC and its Commentary*, using the curves for non-cohesive soil.

#### **6.2.2.3 Resistance to Lateral Loads**

Resistance to lateral force/sliding between the concrete footing and the compacted Granular 'A' pad should be calculated in accordance with the Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the compacted Granular 'A' pad, the coefficient of friction  $\tan \Phi$ , can be taken as 0.7. This value is unfactored.

#### **6.2.2.4 Frost Protection**

Spread footings perched on a Granular 'A' pad should be provided with a minimum of 1.5 m of soil cover for protection from frost penetration in accordance with OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*). As a guide, the MTO has adopted 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent for every 0.3 m reduction in soil cover.

It should be noted that the required thickness of conventional soil cover for frost protection of the footing (1.5 m) is measured perpendicular from the face of the abutment slope to the edge of the underside of the footing (it is not simply a vertical dimension when the footing is adjacent to a slope). Where the Granular 'A' pad is constructed with a 1 horizontal to 1 vertical (1H:1V) side slope, it is typical to cover the pad slope with a 2 horizontal to 1 vertical (2H:1V) conventional earth slope (to promote vegetation growth). If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation shall be installed to compensate for the lack of cover and provide protection from frost action.

#### **6.2.3 Driven Steel H-Piles or Tube Piles**

Based on the subsurface conditions encountered in the boreholes advanced in the vicinity of the proposed abutments and central pier, steel H-piles or steel tube piles driven to found within the very dense sand or very dense sand and silt till ("100-blow" soil) may be used for support of the abutments and the central pier.

For the installation of the steel H-piles or steel tube piles, consideration must be given to the potential presence of cobbles and boulders within the till deposits at the site. In this regard, steel H-piles are preferred over steel tube piles as tube piles are considered to pose a higher risk of "hanging-up" or being deflected away from their vertical or battered orientation during installation, due to their larger end area. It is recommended that the piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the pile. The steel H-piles should be reinforced with flange plates as per OPSD 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) for protection during driving as per OPSS 903 (Deep Foundations). Similarly, if steel tube piles are being considered, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe). The requirement for driving shoes should be included in the Contract Drawings.



Given the anticipated high water pressures in the very dense sand deposit, specialized construction techniques will be required to mitigate the possible upward flow of water along the pile shaft. It is recommended that a sand drainage/filter, possibly in combination with a geotextile and drainage to the adjacent ditches, be placed beneath the pile caps to minimize the migration of fines that may be transported along the piles during and after construction. The drainage/filter layer should consist of a minimum 0.5 m thick layer of concrete fine aggregate, meeting the gradation requirements of OPSS 1002 (Aggregates Concrete).

### **6.2.3.1 Pile Founding Elevation**

The steel H-Piles supporting the abutments and the central pier should be driven to found within the very dense sand or very dense sand and silt ("100-blow" soil). For design, the following pile tip elevations may be used based on the borehole results.

Foundation Element	Reference Borehole No.	Founding Stratum	Estimated Pile Tip Elevation
West Abutment	LA2	Very Dense Sand	295.5 m
Central Pier	LA4	Very Dense Sand to Sand and Silt	291.0 m
East Abutment	LA5	Very Dense Sand	294.5 m

There should be provision made in the contract for dealing with varying pile lengths.

### **6.2.3.2 Geotechnical Axial Resistances**

For steel HP 310x110 piles driven to the estimated tip elevations provided in Section 6.2.3.1, where a depth of 1.5 m of pile is embedded within the "100-blow" material, the factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS (for 25 mm of settlement) are given below. Due to the layout of the proposed bridge structure ("closed-end" structure), the depth of the piles at the abutments was considered relative to the elevation of the top of the front slope where it meets the abutment walls, whereas at the central pier, the pile length was considered relative to the underside of the pile cap.

Foundation Element	Founding Stratum	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
West Abutment	Very Dense Sand	1,400 kN	1,200 kN
Central Pier	Very Dense Sand to Sand and Silt	1,400 kN	1,200 kN
East Abutment	Very Dense Sand	1,400 kN	1,200 kN

Similar axial resistances may be used in the design for closed-end, concrete filled 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of (¼ in.).

It is recommended that a downdrag load of 100 kN be included in design for piles designed for the east abutment to account for the consolidation of the very soft clayey silt layer encountered in Borehole LA5 between about Elevation 304.4 m and 302.3 m.

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 set criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should





be verified in the field by using the Hiley formula (MTO Standard Drawing SS103-11) during the final stages of driving to achieve an ultimate capacity. The ultimate geotechnical axial resistance predicted from the Hiley formula should then be multiplied by a geotechnical resistance factor equal to 0.4 in accordance with Table 6.1 in the *CHBDC* to verify the factored ULS design value. Based on MTO experience with the Hiley formula in the Southern Ontario region, a resistance factor equal to 0.5 may be used for this project. The following Note 2 of MTO's Structural Manual or similar, should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation:

- Piles to be driven in accordance with Standard SS103-11 using an ultimate geotechnical resistance of 2,800 kN per pile, but should be driven to no higher than 1.5 m above the design pile tip elevations shown below at each abutment location:
  - West Abutment – Elevation 295.5 m
  - Central Pier – Elevation 291 m
  - East Abutment – Elevation 294.5 m

Assessment of ultimate geotechnical resistance by the Hiley formula should commence once the pile reaches a depth of not more than 1.5 m above the design pile tip elevation and at 0.5 m intervals of depth until the ultimate capacity is achieved. If the ultimate capacity as determined by the Hiley formula is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley formula should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48 hour wait period, the Contract Administrator should be notified and authorization obtained prior to driving the pile below the design pile tip elevation.

### **6.2.3.3 Resistance to Lateral Loads**

Resistance to lateral loading could be derived using vertical piles, with enhanced support offered by battered piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in front of the piles, whereas battered piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile.

The resistance to lateral loading in front of a vertical pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction ( $k_h$ ) is determined based on the equations given below (CFEM 1992 as noted in Section 6.8.7.1 of the *Commentary to the CHBDC, 2006*):

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} k_h \text{ is the coefficient of horizontal subgrade reaction (MPa/m);} \\ n_h \text{ is the constant of subgrade reaction (MPa/m);} \\ z \text{ is the depth (m) at any point along the pile; and;} \\ B \text{ is the pile diameter (m).} \end{array}$$

For cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{where} \quad \begin{array}{l} k_h \text{ is the coefficient of horizontal subgrade reaction (kPa/m);} \\ s_u \text{ is the undrained shear strength of the soil (kPa); and} \\ B \text{ is the pile diameter (m).} \end{array}$$



## FOUNDATION REPORT – LLOYDTOWN – AURORA ROAD UNDERPASS – HIGHWAY 400 WIDENING G.W.P. 2835-02-00

The following ranges for the value of  $n_h$  and  $s_u$  may be assumed in the structural analyses. Approximate elevation intervals are given below for each deposit at each foundation element, however, the deposit boundaries vary slightly at the abutments, and reference can be made to the borehole records and to the interpreted stratigraphic sections on Drawings 1 and 2 to assess the variation along each abutment.

Foundation Element	Soil unit	Elevation Interval (m)	$n_h$ (MPa/m)	$s_u$ (kPa)
West Abutment	Compact to Very Dense Silty Sand to Sand and Silt Fill	311.0 to 303.5	7.5	--
	Compact Sand and Silt	303.5 to 302.0	4.4	--
	Hard Clayey Silt Till	302.0 to 300.5	--	150
	Very Dense Sand and Silt Till	300.5 to 295.5	11	--
Central Pier	Loose to Compact Sand Fill	305.0 to 303.0	2.2	--
	Firm Clayey Silt Fill	303.0 to 301.5	--	35
	Compact to Very Dense Sand and Silt Till	301.5 to 298.5	7.5	--
	Very Dense Sand to Sand and Silt	298.5 to 291.0	11	--
East Abutment	Compact Sand and Silt Fill	311.0 to 308.0	2.2	--
	Firm to Stiff Clayey Silt Fill	308.0 to 304.5	--	75
	Very Soft Clayey Silt	304.5 to 302.0	--	25
	Dense to Very Dense Sand and Silt Till	302.0 to 299.0	11	--
	Very Dense Sand	299.0 to 294.5	11	--

The maximum factored lateral resistances recommended for design for a single vertical HP310x110 pile driven to the design pile tip elevations given in Section 6.2.3.1 is 120 kN at ULS and 50 kN at SLS for a lateral displacement of 10 mm at pile cap level, provided under Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC*. These values can be employed for piles supporting integral abutments below CSP filled with loose sand.

Group action for lateral loading should also 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 in the direction of loading by a reduction factor,  $R$  (NAVFAC DM-7.2, 1982), as follows:

Pile Spacing in direction of loading ( $d$ = Pile Diameter)	Subgrade Reaction Reduction Factor
8d	1.00
6d	0.70
4d	0.40
8d	1.00

The subgrade reaction reduction factor should be interpolated for pile spacing in between those provided above.





#### **6.2.3.4 Frost Protection**

All pile caps should be provide with a minimum of 1.5 m of soil cover or equivalent thickness of insulation below the cap for frost protection, in accordance with OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*). As a guide, the MTO has adopted 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent to 0.3 m reduction in soil cover.

#### **6.2.4 Caissons**

Given the artesian groundwater conditions at the site, if caissons are used for support of the proposed structure it is recommended that the caissons be founded at a higher elevation compared to the design tip elevations provided for the steel H-piles. During drilling of the boreholes, water and/or drilling fluid was required to be maintained inside the hollow stem augers at a level equivalent to or higher than ground surface in order to advance the borehole. Therefore it is recommended to found the base of the caissons at a higher elevation compared to the piles so as to minimize base disturbance.

The performance of caissons will depend upon the final cleaning and verification of the subsoil/quality (very dense sand and silt till or very dense sand) at the base of the caissons. Where possible each caisson excavation must be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. If caisson foundations are adopted for support of any of the foundation elements, a liner would be required to support the soils during construction, particularly through the fill material and water bearing sand as discussed further under Construction Considerations in Section 6.6 It is expected that the liner would be installed (and removed, if a temporary liner is used) using a vibratory hammer. In this case, vibration monitoring is recommended during liner installation and removal. The liner must be maintained tight to the sides of the bore to minimize seepage of water. In addition, the Ontario Occupational Health and Safety Act (2011) outlines appropriate safety procedures and requirements that must be implemented prior to entry of personnel into the caissons for inspection of the base or alternatively, the inspections may be carried out remotely using visual recording equipment.

The recommended founding elevation of the caissons as provided in Section 6.2.4.1 is intended to maintain the base of the caissons above the groundwater level; however during advancement of the caisson there may be a requirement to maintain a head of water and or drilling fluid within the liner to prevent basal heave and disturbance of the very dense water-bearing sand and/or dense to very dense sand and silt till. Further, it is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to address the need for control of the ground and groundwater during caisson construction as recommended under Construction Considerations in Section 6.6.

If there is water infiltration such that there is standing water within the caisson excavation prior to concrete placement, the concrete must be placed using tremie techniques. After initial placement of concrete at the bottom of the caisson, the tremie discharge point should be maintained a minimum of 1 m below the surface of the wet concrete during placement.



#### 6.2.4.1 Founding Elevation

For design, the following caisson base elevations and strata may be assumed based on the borehole results.

Foundation Element	Boreholes No.	Founding Stratum	Estimated Caisson Founding Elevation
West Abutment	LA1, LA2 and 88	Very Dense Sand to Sand and Silt Till	298.5 m
Central Pier	LA3, LA4 and 87	Very Dense Sand and Silt Till	299.0 m
East Abutment	LA5 and LA6	Very Dense Sand/ Sand and Silt Till	299.0 m

#### 6.2.4.2 Geotechnical Axial Resistances

The following provides the recommended factored axial geotechnical resistance at ULS and geotechnical reaction at SLS (for 25 mm of settlement) for caissons socketted to the design elevations given in Section 6.2.4.1. Since the proposed bridge is a “closed-end” structure the effective vertical stress used to calculate the resistances are based on the grade elevation of Highway 400 and as such the resistances are lower than values typically given for caissons in similar soil conditions.

Foundation Element	Caisson Diameter	Founding Stratum	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
West Abutment	0.9 m	Very Dense Sand and Silt Till / Very Dense Sand	1,200 kN	1,000 kN
Centre Pier	1.2 m		2,150 kN	1,800 kN
East Abutment	1.5 m		3,400 kN	2,800 kN

#### 6.2.4.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons (based on subgrade reaction theory), and the reductions due to group effects, may be determined as per Section 6.2.3.3.

The maximum factored lateral resistances at ULS of 120 kN and maximum lateral resistances at SLS of 50 kN (for 10 mm of horizontal deflection at pile cap level) are recommended for 0.9 m diameter caissons, based on Table C6.4 of the *Commentary on CHBDC*. Values for alternative caisson diameters can be developed if larger diameter caisson foundations are adopted for support of foundation elements at this site.

#### 6.2.4.4 Frost Protection

All caisson caps should be provide with a minimum of 1.5 m of soil cover or equivalent thickness of insulation below the cap for frost protection, in accordance with unless the caissons are extended to the underside of the bridge superstructure OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario). As a guide, the MTO has adopted 25 mm (1 inch) of rigid polystyrene foam insulation as equivalent to 0.3 m reduction in soil cover.



## 6.3 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls/ RSS retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill in accordance with SP110S13 Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSP 3101.150 (*Wall, Abutments Backfill*) and OPSP 3121.150 (*Walls Retaining, Backfill*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS 501. Other surcharge loadings should be accounted for in the design as required.
- For restrained structures the granular fill may be placed either in a zone with the width equal to at least 1.5 m behind the back of the walls (Figure C6.20 (a) of the *Commentary* to the CHBDC). For unrestrained structures, the granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Case B in Figure C6.20 (b) of the *Commentary* to the CHBDC).
- For restrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of earth fill :

	Earth Fill
Soil Unit Weight	20 kN/m <sup>3</sup>
Coefficient of static lateral earth pressure	
Active, $K_a$	0.33
At rest, $K_o$	0.50

- For unrestrained structures, where the pressures are based on SP110S13 granular fill behind the wall, the following parameters (unfactored) may be assumed:

	Granular 'A'	Granular 'B' Type II
Soil Unit Weight	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficient of static lateral earth pressure		
Active, $K_a$	0.27	0.27
At rest, $K_o$	0.43	0.43



If the wall support and superstructure allow lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (such as for a rigid frame structure), at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

A restrained structure is typically a concrete box culvert or a rigid frame bridge where the rotational and/or horizontal movement is not sufficient to mobilize the active centre pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

### **6.3.1 Seismic Considerations**

#### **6.3.1.1 Site Coefficient**

For seismic design purposes, the Site Coefficient,  $S$ , for this site, based on experience and considering the guidelines in Section 4.4.6 of the *CHBDC* may be taken as 1.2, consistent with Soil Profile Type II.

#### **6.3.1.2 Seismic Analysis Coefficient**

The potential for seismic (earthquake) loading must also be taken into account in the design of abutment stems/retaining walls in accordance with Section 4.6 of the *CHBDC*, as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. According to Table A3.1.1 of the *CHBDC*, this site is located in Seismic Performance Zone 1. The site-specific zonal acceleration ratio of the Aurora-Newmarket area is 0.05. Based on experience, for the subsurface conditions at this site, a 20 per cent amplification of the ground motion may occur (i.e. Site Coefficient,  $S=1.2$  for Soil Profile II from Table 4.4 of *CHBDC*), resulting in an increase in the peak horizontal ground acceleration (PHA) from 0.05 g to 0.06 g at the ground surface. Based on Section 4.4.4 of the *CHBDC*, this bridge structure is assigned Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC*, no seismic analysis is required for structures located in Seismic Performance Zone 1.

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

It is understood that mechanically-reinforced soil retaining systems (retained soil system or RSS walls) are proposed as wing walls/retaining walls on both sides of the west and east abutments to retain the Lloydtown - Aurora Road embankment fill (refer to Drawing 1). The RSS retaining walls are to be designed for high performance and appearance in accordance with MTO Special Provision (SP) 599S22 (Retained Soil System).

### **6.4.1 Founding Elevations**

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. The existing embankment fill generally extends to between about Elevation 304.4 m and 302.9 m in the boreholes near the abutments. The footing and the RSS mass should be founded below any topsoil, loose fill or unsuitable native soils (such as the very soft clayey silt encountered between Elevation 304.4 m and 302.3 m in Borehole LA5). In this regard, it will be necessary to place the facing footing below the firm clayey silt fill encountered in Borehole LA1 in the vicinity of the northwest wall, and below the very soft



clayey silt encountered near the east abutment. The following sub-excavation elevations may be used in the RSS wall design:

<b>RSS Wall Location</b>	<b>Sub-excavation Elevation</b>
Northwest Abutment Area	302.9 m
East Abutment Area	302.3 m

The underside of the RSS wall adjacent to the abutments is proposed to be at about Elevation 304.8 m; therefore, based on the above recommended sub-excavation levels, up to 2.5 m of very soft to firm soils will have to be removed from below the RSS wall facing footing.

The facing footing should be placed on a 300 mm thick layer of compacted SP 110S13 Granular 'A', as detailed in Figure 5.2, MTO RSS Wall Design Guidelines (September 2008). The compacted granular pad should extend at least 1.0 m beyond the outside edge of the facing footing, then outward and downward at 1H:1V. Where sub-excavation of fill and unsuitable native soils has been carried out, the Granular A pad and the reinforced soil mass can be constructed immediately on top of the native subgrade soils at the elevations identified above, which would consist of very stiff to hard clayey silt till or compact sand and silt at the west abutment, and dense to very dense sand and silt till or stiff clayey silt till at the east abutment. Alternatively, the thickness of the granular pad can be increased to raise the grade after sub-excavation and the facing footing and reinforced soil mass can be founded at a higher elevation. Where compacted granular fill is used to replace the subexcavated soil, this material should extend outward and downward from the base of the reinforced soil mass or compacted granular pad at 1H:1V to the sub-excavation elevations given above.

Since the height of the RSS walls decreases with increasing distance away from the bridge abutments, consideration could be given to partial sub-excavation of the very soft to firm soils to 1.5 m below the proposed underside of the facing footing for the RSS wall, and providing a 1.5 m thick granular pad constructed of compacted SP110S13 (Aggregates) Granular 'A' or Granular 'B' Type II material. This compacted granular pad should extend at least 1.0 m beyond the outside edge of both sides of the facing footing, then outward/downward at 1H:1V. The compacted Granular 'A' or Granular 'B' Type II as well as the reinforced soil mass should be keyed into the existing embankment by benching into the embankment fill, as per OPSD 208.010 (Benching of Earth Slopes).

#### **6.4.2 Global Stability**

The static and seismic global slope stability of RSS walls adjacent to the Lloydtown - Aurora Road underpass structure has been analyzed using the commercially available program SLIDE, produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety. A target Factor of Safety of 1.3 against deep-seated global instability of the RSS walls is normally used for the design under static conditions; under seismic conditions, a target Factor of Safety of 1.1 is used. These Factors of Safety are considered appropriate for the RSS walls at this site, considering the design requirements and the field data available.

The soil parameters used in the analyses, as given below, were estimated from empirical correlations using the results of in-situ Standard Penetration Tests (Bowles, 1984) and geotechnical classification testing. The groundwater table was taken as Elevation 300.0 m in the analyses.



Soil Type	Unit Weight (kN/m <sup>3</sup> )	Undrained Shear Strength (kPa)	Angle of Internal Friction, $\phi'$ (degrees)
Existing embankment fill	20	--	30
Compact sand and silt	19	--	32
Very stiff to hard clayey silt till	21	150 kPa	34
Dense to very dense sand and silt till	21	--	35

Three RSS wall sections were analyzed for the different wall heights as shown on the drawings provided by URS, dated November 17, 2010. In these analyses, the height of the RSS wall was considered to extend from the top of the pavement elevation to the underside of the lowest panel (top of the front facing footing). The analysis was carried out using a minimum of 0.8 m of soil cover over the front facing footing and a 2H:1V embankment side slope in front of the wall. If the wall configuration changes during the course of the detail design and is different from that assumed above, further stability analyses should be completed as the results are sensitive to the buried depth of wall and the presence of the 2H:1V embankment side slope at the base of the wall.

Given the required RSS wall height(s), the minimum reinforced width of RSS wall required to obtain a Factor of Safety equal to 1.3 or greater against deep-seated global instability has been calculated. The ratio of minimum reinforced mass width to reinforced wall height for three RSS wall heights is provided below and the results of the analyses are shown on Figures 1 to 3.

RSS Wall Height	Ratio of Minimum Reinforced Mass Width to Wall Height
8.3 m	0.7
4.1 m	1.6
2.0 m	2.0

The above ratios for walls with a height of 4.1 m or less are greater than what is typically used by the wall designers (i.e., a ratio of 0.67 is typically adopted), because of the presence of the 2H:1V slope in front of the wall. The contract drawings will need to specify the (non-standard) width of the reinforced soil mass.

Under seismic loading conditions, using a seismic coefficient of 50 per cent of the site-specific design peak horizontal ground acceleration (PHA) equal to 0.03 g, the Factor of Safety is greater than 1.1. The results of example seismic slope stability analyses for the three cases of reinforced mass width to wall height are shown on Figures 4 to 6.

### 6.4.3 Geotechnical Resistances

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass, as recommended in Section 6.4.2, the factored geotechnical resistances at ULS and the geotechnical reaction at SLS (for 25 mm of settlement) given below may be used for assessment of the reinforced mass founded on the properly prepared compacted granular fill, or on the native soil subgrade at the sub-excavation elevations given above.





Wall Height	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
8.3 m	350 kPa	250 kPa
4.1 m	225 kPa	200 kPa
2.0 m	250 kPa	225 kPa

#### 6.4.4 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the compacted granular fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction,  $\tan \phi'$ , between the compacted granular fill of the RSS wall and the properly prepared subgrade may be taken as 0.55.

### 6.5 Approach Embankments

The current pavement grade behind the proposed new abutment locations is at about Elevation 311 m, and the proposed grade following the highway widening and construction of the two-span replacement structure is approximately Elevation 313.5 m. The natural ground surface at the site is at about Elevation 304.5 m. Therefore, the existing 6.5 m high embankment will be raised by about 2.5 m and widened. The overall height of the new embankment will be about 9 m.

Based on the results of the boreholes drilled at this site, the existing approach embankments are founded on compact sand and silt and very stiff to hard clayey silt till near the west abutment, and on very soft clayey silt and stiff clayey silt till near the east abutment. These deposits are underlain by till deposits consisting of hard clayey silt and dense to very dense sand and silt.

#### 6.5.1 Subgrade Preparation and Embankment Construction

The fill comprising the existing approach embankments consists of layers of firm to stiff cohesive soils and loose but generally compact to very dense cohesionless materials. This existing embankment fill is considered to be appropriate for incorporation into the new approach embankments, with placement of 2.5 m of additional fill on top of the existing, and widening to the north and south. However, to improve the performance of the embankment as related to reducing the potential for post-construction settlement and to achieve stability of the embankment, it is recommended that prior to the placement of the additional fill, all topsoil, organic matter and soft/loose fill should be stripped from below the approach embankment areas. Embankment fill should be placed and compacted in accordance with SP 206S03 (Excavation and Grading), OPSS 501 (Compacting) and SP 105S21 (Amendment to OPSS 501).

In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where embankment slopes are greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m, consistent with OPSD 202.010 (Slope Flattening). To reduce the potential for erosion of the embankment side 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. The erosion protection should be in accordance with OPSS 572 (relocated to OPSS 804) (*Seed and Cover*).



### 6.5.2 Approach Embankment Stability

Static and seismic slope stability analyses of the proposed approach embankments were carried out with the commercially available program Slide (produced by Rocscience Inc.) to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally used in the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this site.

The soil parameters used in the analysis, as given below, were estimated from empirical correlations suggested by proposed by Kulhawy and Mayne (1990) and the CHBDC (2006) using the results of in-situ Standard Penetration Tests (SPT) and geotechnical classification testing. For the purpose of analysis, earth fill have been considered for the construction of the approach embankment with side slopes at 2H:1V. The groundwater table in the analyses was taken to be Elevation 300.0 m.

Approach Embankment	Soil Type	Unit Weight (kN/m <sup>3</sup> )	Undrained Shear Strength (kPa)	Angle of Internal Friction, $\phi'$ (degrees)
West Approach (Borehole LA2)	New Embankment Fill	21	--	35
	Silty Sand to Sand and Silt Fill	20	--	30
	Compact Sand and Silt	19	--	32
	Very Stiff to Hard Clayey Silt Till	21	150	34
	Very Dense Sand and Silt Till	21	--	35
	Very Dense Sand	21	--	34
East Approach (Borehole LA5)	New Embankment Fill	21	--	35
	Sand and Silt / Clayey Silt Fill	20	--	30
	Very Soft Clayey Silt	18	25	
	Dense to Very Dense Sand and Silt Till	21	--	35
	Very Dense Sand	21	--	34

At the west approach embankment, assuming appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the proposed 9 m high embankment with side slopes maintained at 2H:1V will have a Factor of Safety of greater than 1.3 against deep-seated slope instability.

At the east approach embankment, a 2.1 m thick layer of very soft clayey silt deposit (where present) may impact the global stability of the embankment, if encountered within the footprint of the embankment widening. This deposit was encountered in Borehole LA5, which was drilled through the existing embankment; it was not encountered in Borehole LA6 near the north toe of the east approach embankment. This deposit does not have to be subexcavated from below the existing embankment. However, in order to achieve a factor of safety of greater than 1.3 against deep-seated slope instability for the section of widening of the east approach embankment, any soft clayey silt soil will have to be subexcavated where such material is present within the footprint of the widening. Sub-excavation of the soft clayey silt soil and backfilling of the excavated area during the embankment widening should be carried out in accordance with OPSS 209 so as to not affect the stability of the existing embankment. It is recommended that a provision be made in the Contract Documents for a variable quantity of sub-excavation of this clayey silt deposit at the east approach embankment.





Provided that such soft clayey silt (where present) is subexcavated from below the embankment widening footprint and replaced with new embankment fill material prior to constructing the widened embankments, the proposed 9 m high east approach embankment with side slopes maintained at 2H:1V will have a Factor of Safety of greater than 1.3 against deep-seated slope instability (refer to Figure 7).

Under seismic loading conditions with yield peak horizontal ground acceleration (PHA) equal to 0.03g, the Factor of Safety is greater than 1.1 as shown on Figure 8.

### **6.5.3 Approach Embankment Settlement**

Settlement of the approach embankments at the site will occur due to compression of the new embankment fill (approximately 2.5 m required to raise the grade on top of the existing embankment, plus the 9 m high widened portions of the embankment), as well as compression of the existing embankment fill and underlying native soils due to the increased embankment height and load. The compression for the approach embankments was modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). The values of the parameters given are based on the soil conditions encountered in Borehole LA-5 drilled at the location of the proposed east abutment as this area is considered as worst case due to the presence of the very soft clayey silt deposit encountered beneath the existing fill embankment. The groundwater table in the analyses was taken to be Elevation 300.0 m.

<b>Soil Deposit</b>	<b>Bulk Unit Weight</b>	<b>Estimated Deformation Properties</b>
Sand and Silt / Clayey Silt Fill	20 kN/m <sup>3</sup>	E = 20 MPa
Very Soft Clayey Silt	18 kN/m <sup>3</sup>	$m_v = 1 \times 10^{-3} \text{ kPa}^{-1}$
Dense to Very Dense Sand and Silt Till	21 kN/m <sup>3</sup>	E = 65 MPa
Very Dense Sand	21 kN/m <sup>3</sup>	E = 100 MPa

Below the east approach embankment, where the 2.1 m thick very soft clayey silt deposit would remain in place below the existing embankment footprint, the results of the analyses indicate that a total settlement between about 130 mm and 270 mm would occur, with the lower bound occurring at the centreline of the existing embankment and the upper bound occurring near the toe of the existing embankment/crest of the new embankment. This settlement is estimated to be comprised of up to about 15 mm to 20 mm of immediate settlement due to compression of the cohesionless sand and silt till and sand, and between about 120 mm and 250 mm of time-dependent settlement due to primary consolidation of the near-surface very soft clayey silt deposit. The "immediate" settlements are expected to occur during or shortly after construction in response to the placement of the new embankment fill.

Based on an estimated coefficient of consolidation ( $C_v$ ) equal to  $7 \times 10^{-3} \text{ cm}^2/\text{s}$  estimated for the very soft cohesive layer and the imposed loading conditions, and assuming two-way drainage of the approximately 2.1 m thick clayey silt deposit, it is estimated that about 90% of the primary consolidation settlement will be completed within one month. The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 5 mm per log-cycle of time, corresponding to about 15 mm over a twenty-year (20-year) period following completion of construction.



Below the west approach embankment, the results of the analyses indicate that a total settlement of up to about 10 mm would occur at the road centreline. This settlement is expected to occur rapidly (i.e. during or shortly after construction) in response to filling based on the cohesionless or heavily overconsolidated nature of the underlying soils.

#### **6.5.3.1 Settlement of New Embankment Fill**

Provided that the existing embankments have been constructed of clean and suitable earth fill and/or granular fill of similar composition as encountered at Boreholes LA2 and LA5, and the new fill to raise and widen the embankments is comprised of suitable earth or granular fill meeting the requirements of and placed and compacted in accordance with SP 206S03, the settlement of the additional 2.5 m of fill to be placed to raise the existing approach embankments and the 9 m high wedge of widened embankment on the north and south sides of the existing embankments is expected to be less than about 25 mm, and this settlement is expected to occur relatively quickly, during and immediately following construction.

#### **6.5.3.2 Mitigation of Settlement**

As discussed in Sections 6.5.3 and 6.5.3.1, some time-dependent settlement is anticipated below the east approach embankment. It is estimated that 90 per cent of the primary consolidation settlement will be completed within about one month. However, in order to meet the settlement performance criterion for the embankment and to eliminate the need for instrumentation and settlement monitoring during and after construction, it is recommended that the east approach embankment be preloaded for a period of not less than three months to allow the majority of the compression/consolidation settlement to occur prior to final grading and paving. If the construction schedule can accommodate this preload period, the magnitude of remaining primary consolidation settlement and the secondary consolidation settlement is estimated to be about 10 mm within the first twenty (20) years following completion of construction.

In addition, as discussed under Section 6.5.2 regarding embankment stability, sub-excavation of soft clayey silt soils is required where these are encountered within the footprint of the embankment widening.

## **6.6 Construction Considerations**

### **6.6.1 Open-Cut Excavation**

The foundation excavations at the abutments for spread footings or pile cap construction will extend through existing fill and into the till deposit(s). Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill materials are classified as Type 3 soil and the till is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical. As discussed in Section 6.6.1, stability and settlement mitigation options include removal of the near surface clayey silt layer at the east approach embankment. If the cohesive layer is to be removed from within the approach embankment footprint, sub-excavation of the clayey layer to a depth of up to 2.1 m below existing ground surface will be required prior to placement of backfill / embankment fill. In addition, all topsoil, organic matter and soft/loose fill should be stripped from below the proposed approach embankment areas, in accordance with SP 206S03.



### **6.6.2 Temporary Roadway Protection**

It is expected that temporary excavation support will be required to maintain traffic lanes in operation on Lloydtown - Aurora Road during construction of the new abutments and retaining walls. The following soil parameters may be used for the design of the temporary support system in accordance with OPSS 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided that any adjacent utilities can tolerate this magnitude of deformation. Assuming the system will be retaining the existing embankment fill; surcharge loads must be included in the design:

- Unit Weight of Soil,  $\gamma = 20 \text{ kN/m}^3$  and
- Earth Pressure Coefficient,  $K_a = 0.33$
- Groundwater Elevation = 300 m

### **6.6.3 Groundwater Control**

The groundwater level measured on October 20, 2000 in the standpipe piezometers installed in the sand deposit in Boreholes 87 and 88 underlying the till deposit at the site range between about 5 m to 6 m below the existing grade of Highway 400, corresponding to Elevation 300 m. Therefore, it is expected that the base of excavations for spread footings founded on the hard clayey silt till and or compact to dense sand and silt till will be about 1 m to 2 m above the groundwater level. Some groundwater may be “perched” within the existing granular fill at the site, and therefore some water inflow should be expected into the foundation excavations, particularly during wet months, however, it is anticipated that water inflow can be handled by pumping from filtered sump pumps placed at the base of the excavation.

### **6.6.4 Subgrade Protection**

The soils exposed at the footing or pile cap subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a working slab of concrete be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. An NSSP, such as the example presented in Appendix C, should be included in the Contract Documents for this item.

### **6.6.5 Ground and Groundwater Control for Caissons Installation**

As discussed in Section 6.2.4, running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons and basal heave could occur where water-bearing cohesionless soils are present at the caisson base. If caisson foundations are adopted for support of any of the foundation elements temporary or permanent caisson liners would be required to support the soils during construction and permit inspection and cleaning of the caisson base. It is recommended that an NSSP be included in the Contract Documents to warn the contractor of these conditions and the need to control the ground and groundwater during caisson construction; an example NSSP is presented in Appendix C.

### **6.6.6 Obstructions During Pile Driving / Caisson Installation**

It is anticipated that cobbles and/or boulders will be encountered within the till deposits, as noted in Borehole LA2 advanced at this site; which may affect the installation of steel H-piles and/or caissons. It is recommended that flange plate reinforcement or driving shoes be used on all steel H-piles to facilitate driving into the hard



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## **FOUNDATION REPORT – LLOYDTOWN – AURORA ROAD UNDERPASS – HIGHWAY 400 WIDENING G.W.P. 2835-02-00**

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clayey silt till and very dense sand and silt till. In addition it is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils; an example NSSP is presented in Appendix C.



## FOUNDATION REPORT – LLOYDTOWN – AURORA ROAD UNDERPASS – HIGHWAY 400 WIDENING G.W.P. 2835-02-00

### 7.0 CLOSURE

This Foundation Design Report was prepared by Ms. T. Veronica Ayetan, P.Eng., a geotechnical engineer, and Ms. Sandra McGaghran, P.Eng., a senior geotechnical engineer with Golder. Mr. Jorge Costa, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.

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### Ontario Provincial Standard Specifications (OPSS)

- |           |   |
|-----------|---|
| OPSS 209  | Construction Specification for Embankments Over Swamps and Compressible Soils |
| OPSS 501  | Construction Specification for Compacting                                     |
| OPSS 539  | Construction Specification for Temporary Protection Systems                   |
| OPSS 572  | (Relocated to OPSS 804) Construction Specification for Seed and Cover         |
| OPSS 903  | Construction Specification for Deep Foundations                               |
| OPSS 1002 | Material Specification for Aggregates - Concrete                              |

### Ontario Provincial Standard Drawings (OPSD)

- |               |   |
|---------------|---|
| OPSD 202.010  | Slope Flattening  |
| OPSD 208.010  | Benching of Earth Slopes                                  |
| OPSD 3000.100 | Foundation Piles – Steel H-Pile Driving Shoe              |
| OPSD 3001.100 | Foundation, Piles – Steel Tube Pile Driving Shoe          |
| OPSD 3090.101 | Foundation Frost Penetration Depths for Southern Ontario  |
| OPSD 3101.150 | Walls Abutment, Backfill – Minimum Granular Requirements  |
| OPSD 3121.150 | Walls Retaining, Backfill – Minimum Granular Requirements |

### Contract Design Estimating and Documentation (CDED)

- |           |  |
|-----------|--|
| SP 110S13 | Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material |
| SP 206S03 | Excavation and Grading; Excavation for Pavement Widening                                     |
| SP 105S21 | Amendment to OPSS 501  |
| SP 599S22 | Retained Soil System, Wall/Slope, High Performance   |
- Occupational Health and Safety Act and Regulations, Construction Projects (O.Reg 213191), 2011.



## FOUNDATION REPORT – LLOYDTOWN – AURORA ROAD UNDERPASS – HIGHWAY 400 WIDENING G.W.P. 2835-02-00

**TABLE 1**  
**COMPARISON OF FOUNDATION ALTERNATIVES**  
**LLOYDTOWN - AURORA ROAD UNDERPASS - HIGHWAY 400 WIDENING G.W.P. 2853-02-00**

Option	Rank	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<b>Strip or Spread Footing</b> on dense to very dense sand and silt till or very stiff clayey silt till	3	<ul style="list-style-type: none"> <li>Feasible for support of abutments and pier.</li> </ul>	<ul style="list-style-type: none"> <li>Relative ease of construction; and,</li> <li>Negligible post-construction settlement.</li> </ul>	<ul style="list-style-type: none"> <li>Approximately up to 10 m of sub-excavation required at the abutments and up to about 4 m of sub-excavation below proposed highway road grade to reach competent material at the pier;</li> <li>Traffic protection system required during construction;</li> <li>Groundwater control required (can be controlled by pumping from sumps depending on the time of the year);</li> <li>Lower bearing capacities compared to deep foundation options; and,</li> <li>Does not allow for integral abutment construction if used at the abutments.</li> </ul>	<ul style="list-style-type: none"> <li>Lower relative costs than deep foundations; and,</li> <li>Sub-excavation up to about 4.0 m of fill and surficial soils required.</li> </ul> <p>(3 m wide x 30 m long x 1.0 m thick x 3 footings) @ \$ 600 / m<sup>3</sup> + (10 m deep x 12 m wide x 30 m long x 2 abutments) @ \$ 10 / m<sup>3</sup> + (4 m deep x 5 m wide x 30 m long x 1 pier) @ \$ 10 / m<sup>3</sup> ≈ \$ 240,000</p>	<ul style="list-style-type: none"> <li>Moderate risk of loosening of subgrade soil due to ponded water, but can be mitigated with concrete working slab; and,</li> <li>Potential traffic disruption during construction.</li> </ul>
<b>Strip or Spread Footing “perched”</b> on Granular ‘A’ pad in approach embankment fill	2	<ul style="list-style-type: none"> <li>Feasible for support of abutments.</li> </ul>	<ul style="list-style-type: none"> <li>Negligible post-construction settlement; and,</li> <li>Footing subgrade would not be disturbed by groundwater.</li> <li>Reduce depth of existing embankment excavation compared to footings founded at lower founding elevation.</li> </ul>	<ul style="list-style-type: none"> <li>Traffic protection system required during construction;</li> <li>Lower bearing capacities compared to deep foundation options;</li> <li>Longer bridge spans required; and,</li> <li>Does not allow for integral abutment construction</li> </ul>	<ul style="list-style-type: none"> <li>Low cost option; and,</li> <li>Relatively lower cost for excavation of existing embankment fill.</li> </ul> <p>(3 m wide x 30 m long x 1 m thick x 2 footings) @ \$ 600 / m<sup>3</sup> + (2 m deep x 4 m wide x 30 m long x 2 abutments) @ \$ 10 / m<sup>3</sup> ≈ \$ 115,000 plus Granular ‘A’ Pad.</p>	<ul style="list-style-type: none"> <li>Must ensure proper compaction of Granular ‘A’ pad to minimise any post-construction settlement; and,</li> <li>Potential traffic disruption during construction.</li> </ul>
<b>Steel H-Piles</b> driven within “100-blow” material.	1	<ul style="list-style-type: none"> <li>Feasible for support of abutments and pier.</li> </ul>	<ul style="list-style-type: none"> <li>Higher geotechnical axial resistance, compared to spread footings;</li> <li>Negligible post-construction settlement; and,</li> <li>Can be used for support of conventional or integral abutments.</li> </ul>	<ul style="list-style-type: none"> <li>Requires 30 m long excavation for pile cap;</li> <li>Traffic protection system required during construction;</li> <li>Long piles may be required to reach “100-blow” materials; and,</li> <li>Requirement for sand filter and geotextile beneath the pile caps to reduce potential of migration of fines that may be carried due to artesian groundwater condition along the piles.</li> </ul>	<ul style="list-style-type: none"> <li>Higher cost than spread footings; and,</li> <li>Installation costs could be impacted by presence of obstructions.</li> </ul> <p>Assume (36 piles x 8 m long) @ \$ 250 / m<sup>3</sup> ≈ \$ 72,000 plus excavation and pile cap costs of about \$ 232,000.</p>	<ul style="list-style-type: none"> <li>Potential traffic disruption during construction;</li> <li>Negligible risk of post-construction settlement of underpass structure, or of differential settlement of foundation elements;</li> <li>Risk of encountering obstructions that could impact pile installation; and,</li> <li>Potentially less costly maintenance over life of the structure than semi-integral abutment structures.</li> </ul>





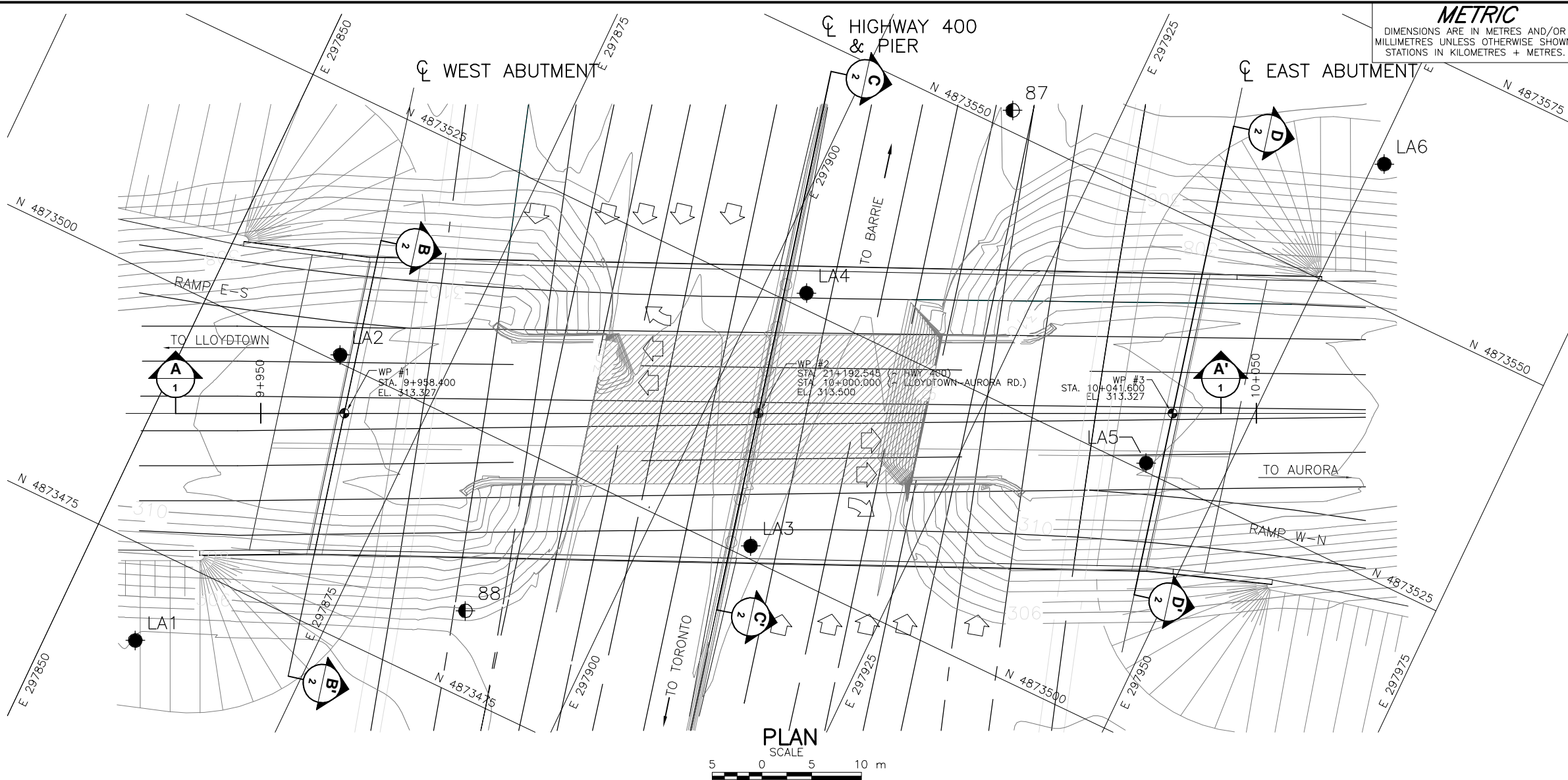
## FOUNDATION REPORT – LLOYDTOWN – AURORA ROAD UNDERPASS – HIGHWAY 400 WIDENING G.W.P. 2835-02-00

Option	Rank	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<b>Steel Tube Piles</b> (closed-end, concrete filled) driven within "100-blow" material.	4	<ul style="list-style-type: none"> <li>Feasible for support of abutments and pier.</li> </ul>	<ul style="list-style-type: none"> <li>Higher geotechnical axial resistance, compared to spread footings;</li> <li>Negligible post-construction settlement; and,</li> <li>Can be used for support of conventional or integral abutments provided the pile size can accommodate the lateral resistance required for such abutment design.</li> </ul>	<ul style="list-style-type: none"> <li>Requires sub-excavation for cap construction;</li> <li>Traffic protection system required during construction;</li> <li>Long piles may be required to reach "100-blow" materials;</li> <li>Greater disturbances to immediately adjacent ground due to larger base area if end is closed;</li> <li>Requires staged construction for driving, cleaning and concrete filling of tube;</li> <li>Greater potential for crumpling if obstructions encountered;</li> <li>Requirement for sand filter beneath the centre pier pile caps to reduce potential of migration of fines that may be carried along the piles due to high groundwater table present at the site; and,</li> <li>MTO does not allow the use of pipe piles for integral abutment design.</li> </ul>	<ul style="list-style-type: none"> <li>Higher cost than spread footings;</li> <li>Cost for steel tube (pipe) piles slightly higher than for steel H-piles; and,</li> <li>Installation costs could be impacted by presence of obstructions.</li> </ul> <p>Assume same cost as steel H-piles ≈ \$ 305,000.</p>	<ul style="list-style-type: none"> <li>Potential traffic disruption during construction;</li> <li>Negligible risk of post-construction settlement of underpass structure, or of differential settlement of foundation elements; and,</li> <li>Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; resulting in piles "hanging up".</li> </ul>
<b>Caissons</b> founded within very dense sand and silt till and very dense sand	5	<ul style="list-style-type: none"> <li>Feasible for support of abutments and pier.</li> </ul>	<ul style="list-style-type: none"> <li>Sub-excavation is not required (no cap at/below ground surface);</li> <li>Higher geotechnical axial resistance compared to spread footings and piles; so reduced number of deep foundation elements compared to steel H- or tube piles; and,</li> <li>Negligible post-construction settlement.</li> </ul>	<ul style="list-style-type: none"> <li>Potential for blow-out of the caisson base due to the presence of the silt to sand and silt deposits under high hydrostatic head;</li> <li>Caissons could encounter obstructions (cobbles and boulders) during installation;</li> <li>Need for temporary or permanent liners;</li> <li>Cleaning of the base below the water table could be difficult;</li> <li>Potential requirement for placement of concrete by tremie method;</li> <li>Traffic protection system required during construction;</li> <li>Not suitable for integral abutment design for the standard MTO tube size; and,</li> <li>Greater risk of encountering obstructions due to larger size of drill hole required.</li> </ul>	<ul style="list-style-type: none"> <li>More costly option than steel H-piles;</li> <li>Installation cost could be impacted by presence of cobbles and boulders; and,</li> <li>Installation cost could be impacted by need for liner and also the need to maintain a head of water inside the liner to counter-balance the water pressures in the very dense sand, and for tremie concrete placement.</li> </ul> <p>Assume (8 caissons / element x 8 m long x 3) @ \$ 2,000 / m<sup>3</sup> ≈ \$ 384,000.</p>	<ul style="list-style-type: none"> <li>Risk of disturbance of water-bearing sand and silt till soils, requiring special construction procedures including use of temporary or permanent liners;</li> <li>Significant traffic disruption during construction due to space required for caisson drilling equipment;</li> <li>Negligible risk of post-construction settlement of overpass structure, or of differential settlement of foundation elements; and,</li> <li>Risk of encountering obstructions that could impact caisson installation/costs.</li> </ul>

Prepared By: TVA

Reviewed By: JMAC





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

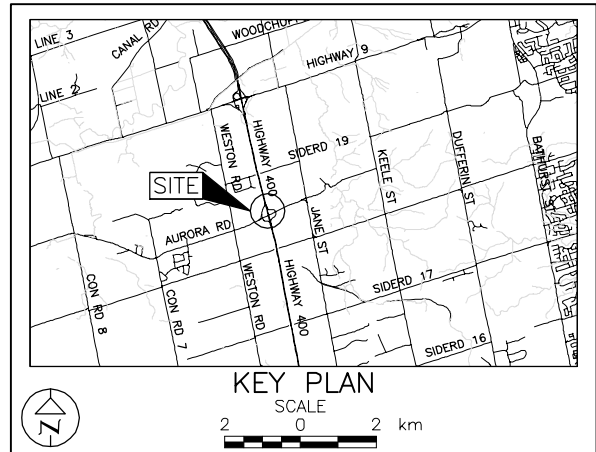
CONT No.  
GWP No. 2835-02-00

HIGHWAY 400  
LLOYDTOWN - AURORA ROAD UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

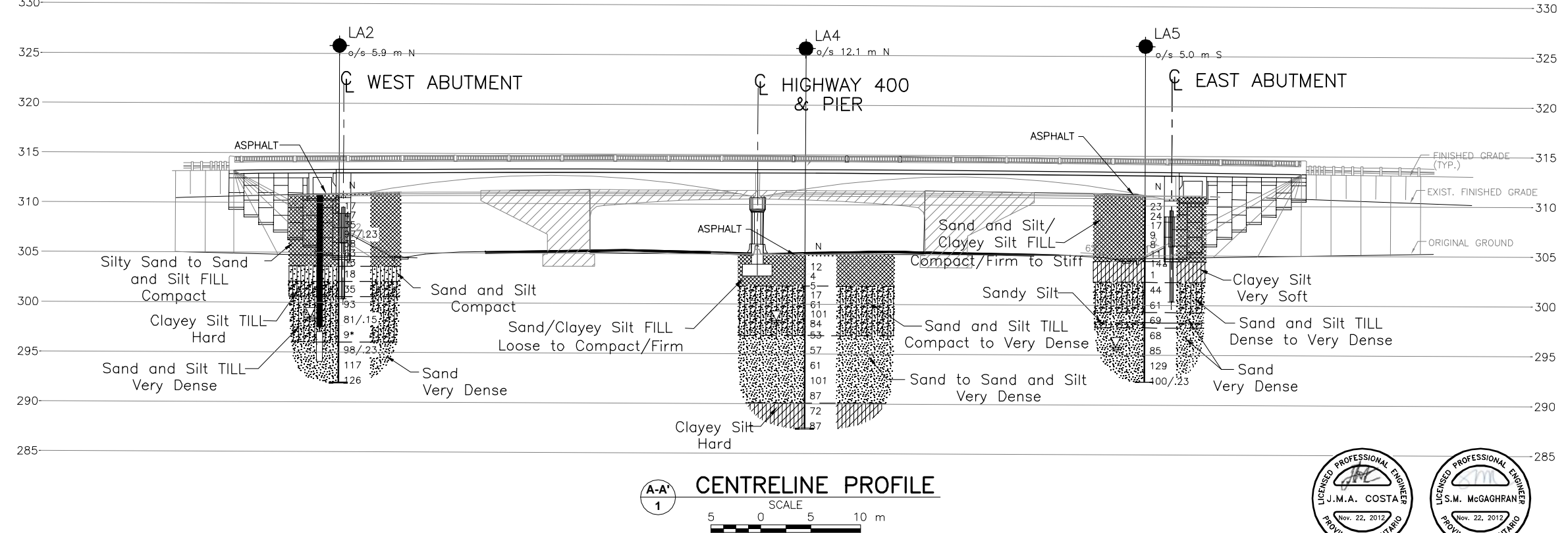
**Golder Associates**

**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation, Golder Associates Ltd. Report No. 001-1122F-7, dated May, 2001
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on May 28, 2011
- WL upon completion of drilling



BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
87	305.0	4873552.0	297915.0
88	305.0	4873483.0	297887.0
LA1	304.4	4873466.0	297858.4
LA2	310.8	4873500.8	297864.6
LA3	305.0	4873501.2	297910.1
LA4	304.8	4873526.5	297904.2
LA5	311.0	4873525.8	297942.3
LA6	304.3	4873563.2	297951.0

**NOTES**

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

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE			
Base plans provided in digital format by URS, drawing file no. Aurora Rd Underpass GA.dwg, received November 17, 2010.			
Geocres No. 31D-550			
NO.	DATE	BY	REVISION
PROJECT NO. 09-1111-0018			
HWY. 400	PROJECT NO. 09-1111-0018	DIST.	
SUBM'D. TT	CHKD. SMM	DATE: 11/22/2012	SITE:
DRAWN: JFC	CHKD. SMM	APPD. JMAC	DWG. 1



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

CONT No.  
GWP No. 2835-02-00



HIGHWAY 400  
LLOYDTOWN - AURORA ROAD UNDERPASS  
SOIL STRATA

SHEET



**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



**KEY PLAN**  
SCALE  
2 0 2 km

**LEGEND**

- Borehole - Current Investigation
- ⊕ Borehole - Previous Investigation, Golder Associates Ltd. Report No. 001-1122F-7, dated May, 2001 Seal
- ⊕ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ≡ WL in piezometer, measured on May 28, 2011
- ≡ WL upon completion of drilling

**BOREHOLE CO-ORDINATES**

No.	ELEVATION	NORTHING	EASTING
87	305.0	4873552.0	297915.0
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LA5	311.0	4873525.8	297942.3
LA6	304.3	4873563.2	297951.0

**NOTES**

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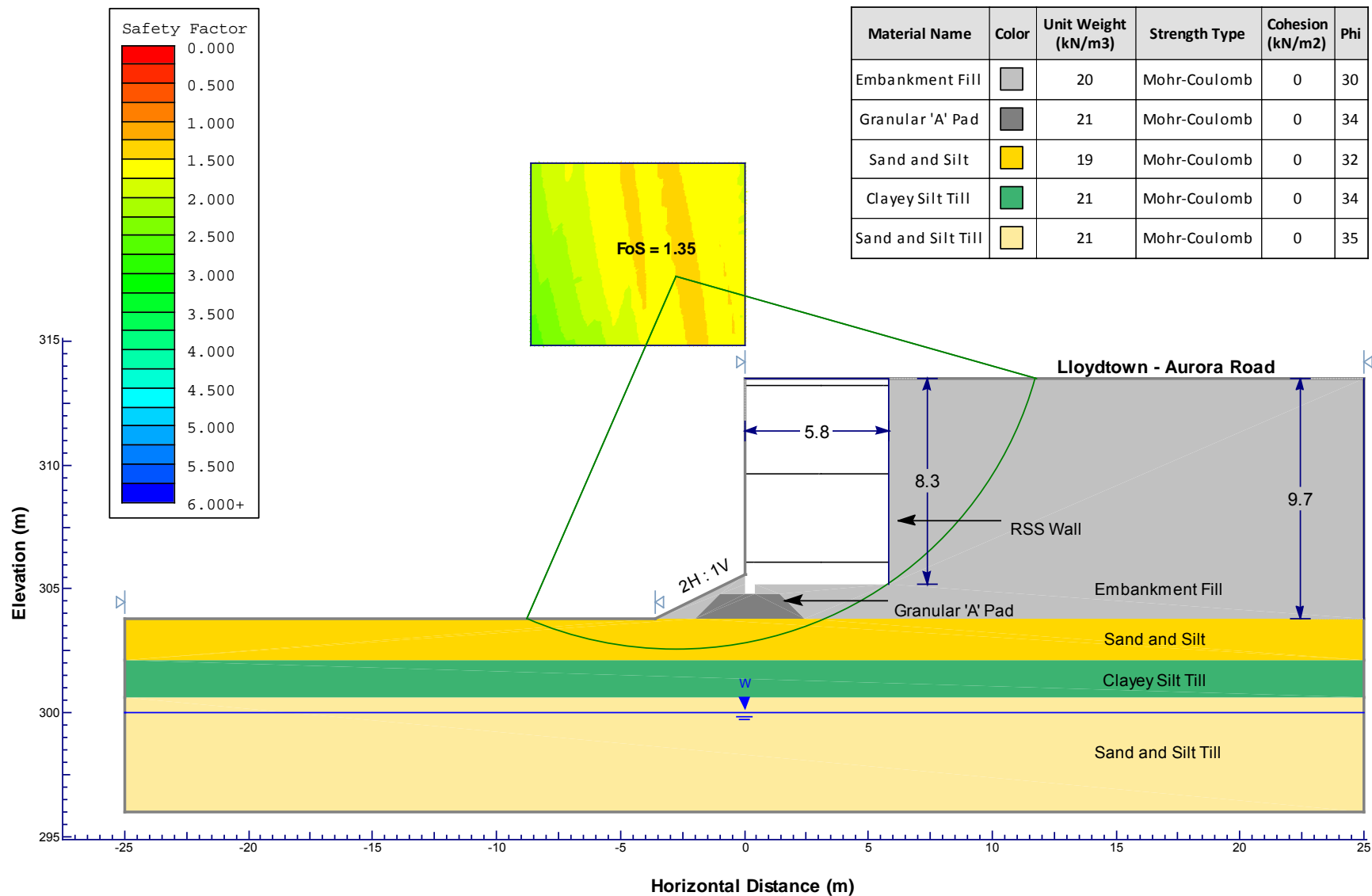


NO.	DATE	BY	REVISION
Geocres No. 31D-550			
HWY. 400	PROJECT NO. 09-1111-0018	DIST.	
SUBM'D. TT	CHKD. SMM	DATE: 11/22/2012	SITE:
DRAWN: JFC	CHKD. SMM	APPD. JMAC	DWG. 2



# Lloydtown – Aurora Road Underpass – Hwy 400 Widening RSS Wall Static Global Stability – 8.3 m High Wall

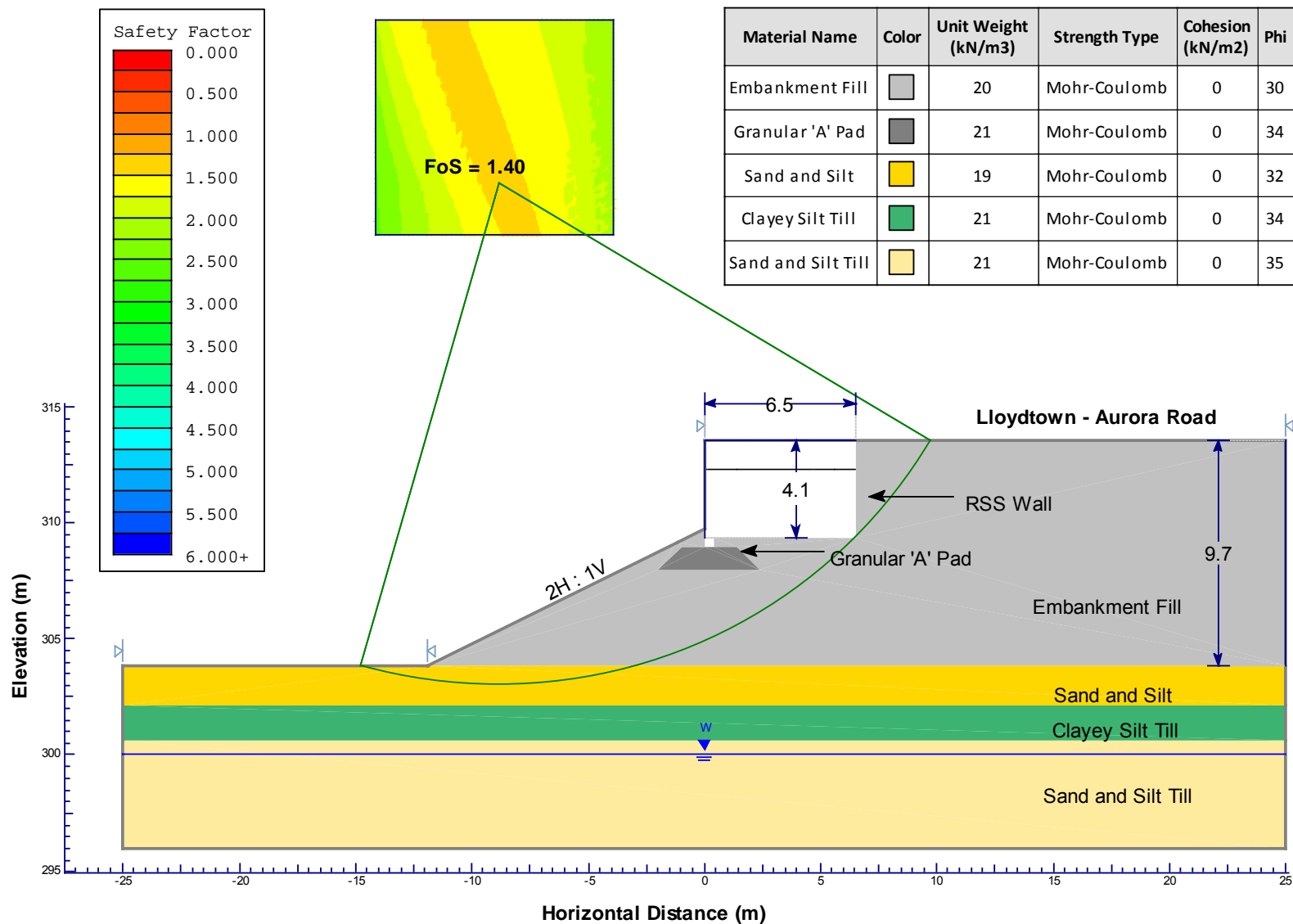
Figure 1





# Lloydtown - Aurora Road Underpass – Hwy 400 Widening RSS Wall Static Global Stability – 4.1 m High Wall

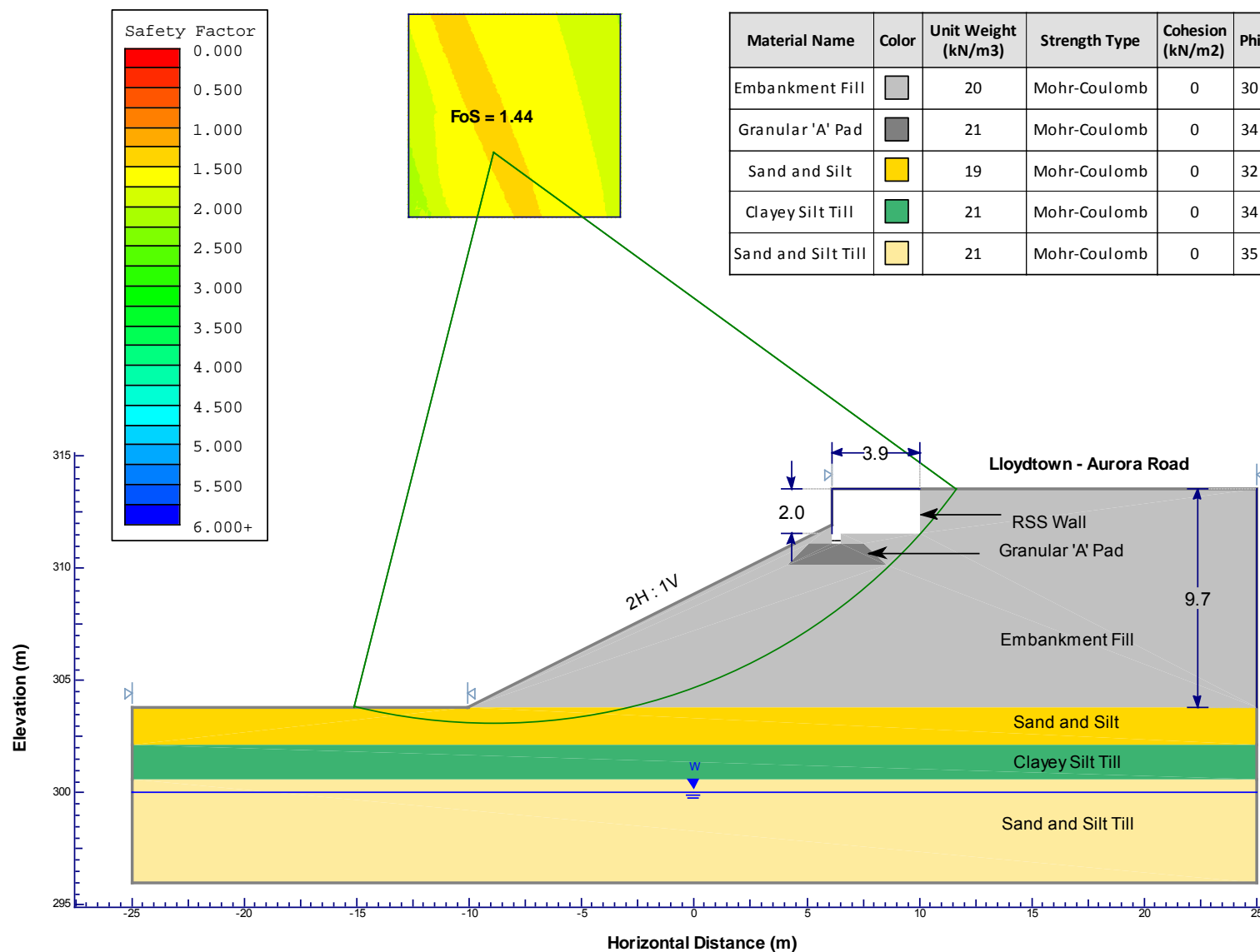
Figure 2





# Lloydtown - Aurora Road Underpass – Hwy 400 Widening RSS Wall Static Global Stability – 2.0 m High Wall

Figure 3

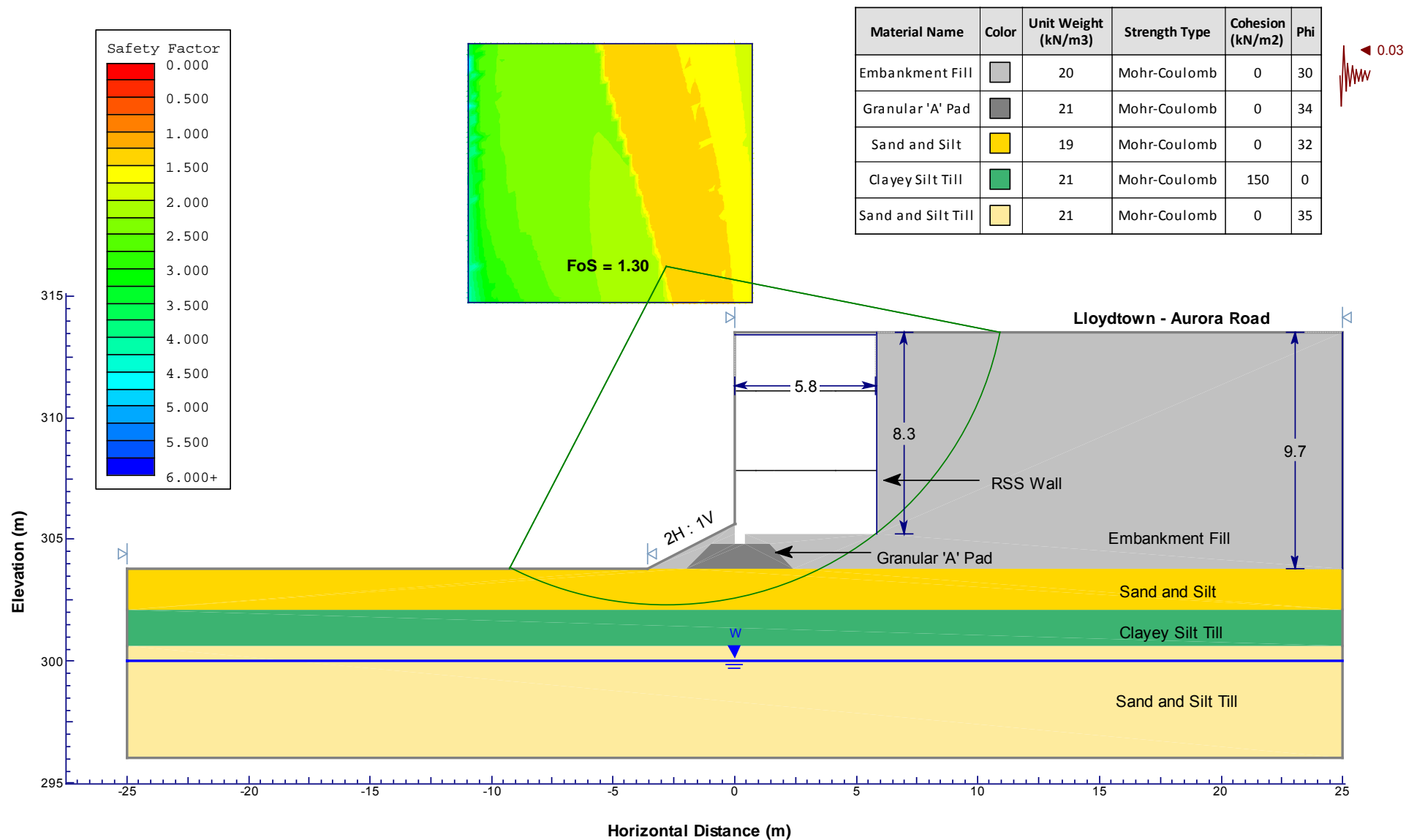






# Lloydtown - Aurora Road Underpass – Hwy 400 Widening RSS Wall Seismic Global Stability – 8.3 m High Wall

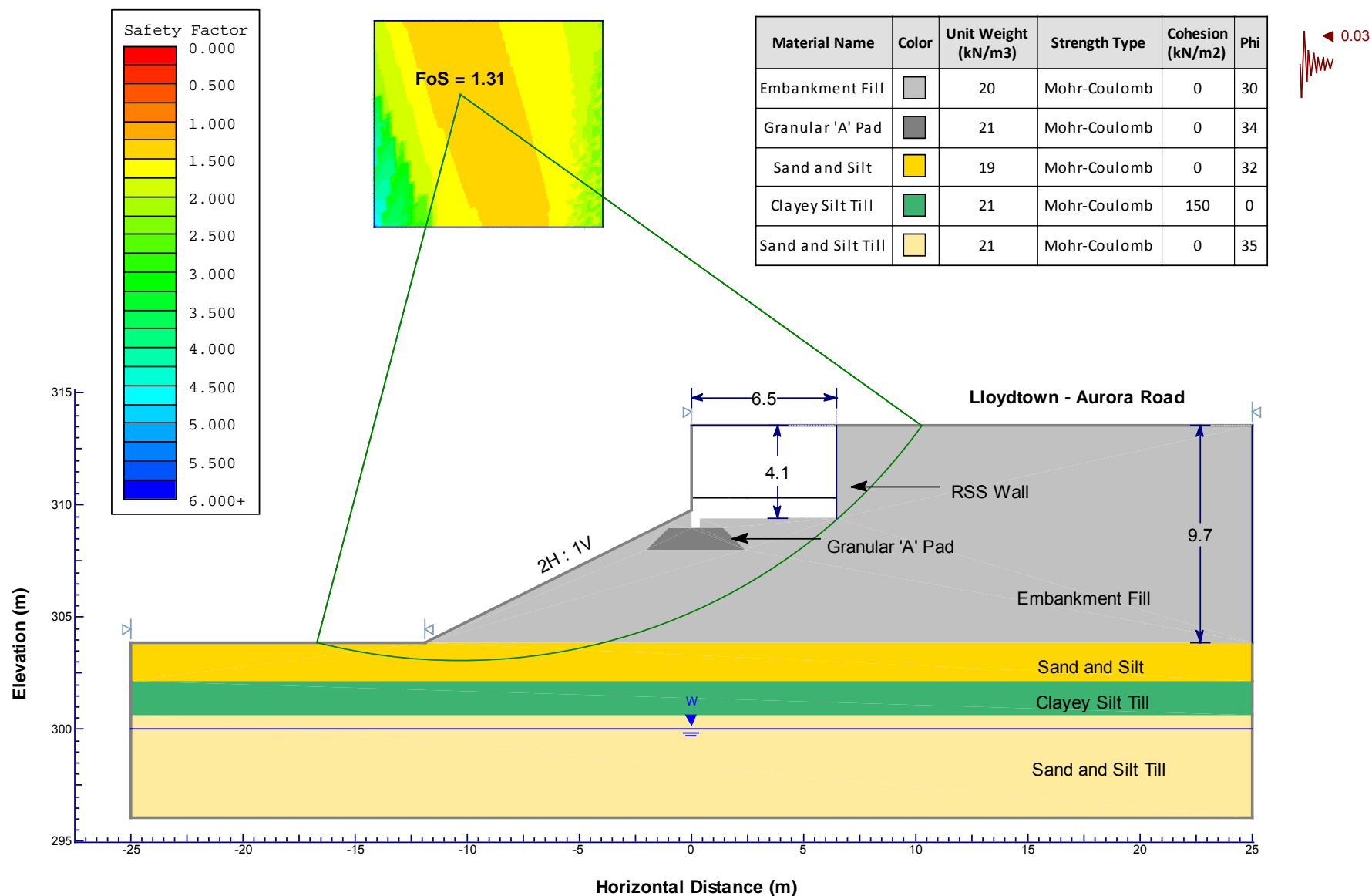
Figure 4





# Lloydtown - Aurora Road Underpass – Hwy 400 Widening RSS Wall Seismic Global Stability – 4.1 m High Wall

Figure 5

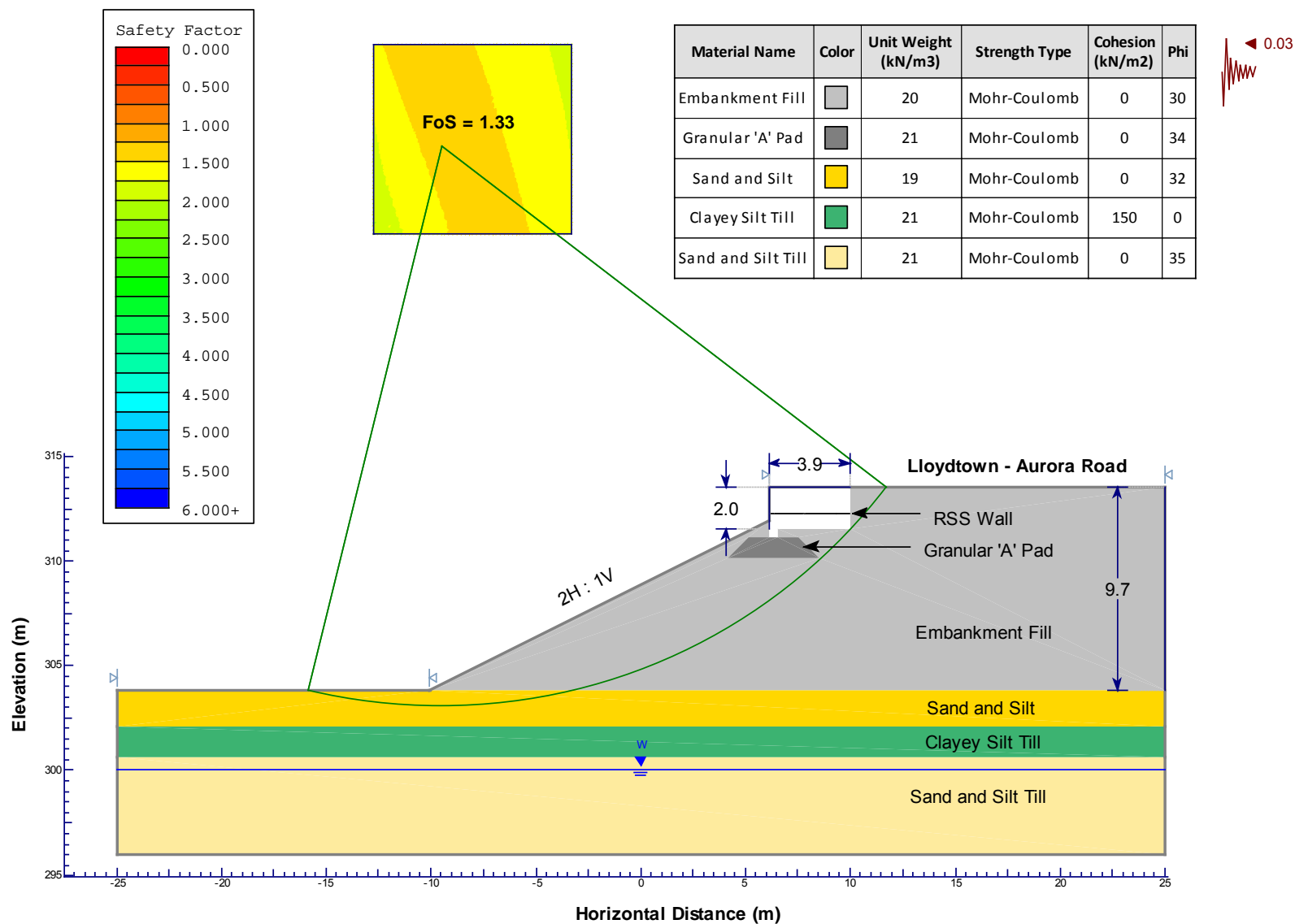






# Lloydtown - Aurora Road Underpass – Hwy 400 Widening RSS Wall Seismic Global Stability – 2.0 m High Wall

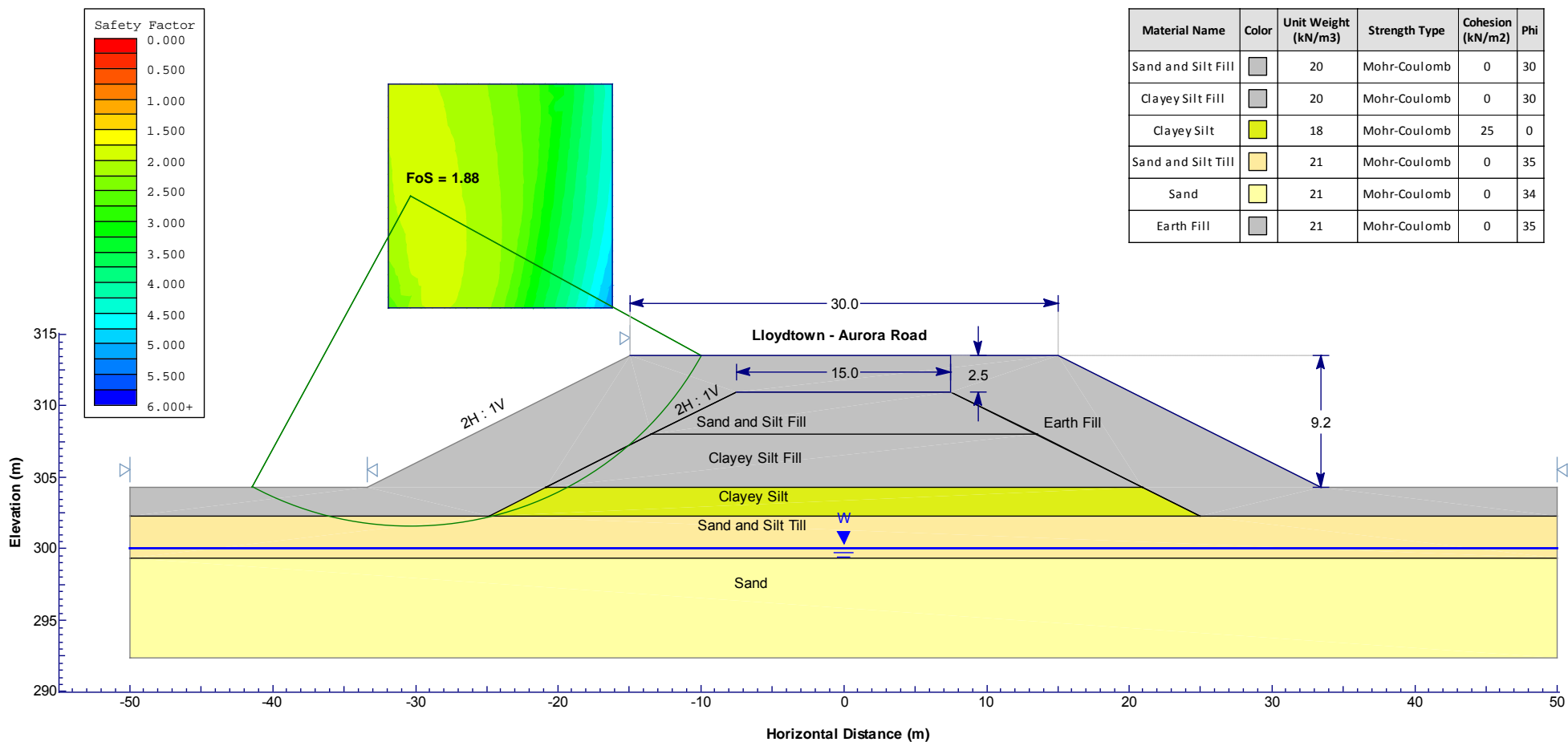
Figure 6





# Lloydtown – Aurora Road Underpass – Hwy 400 Widening East Approach Embankment Static Global Stability

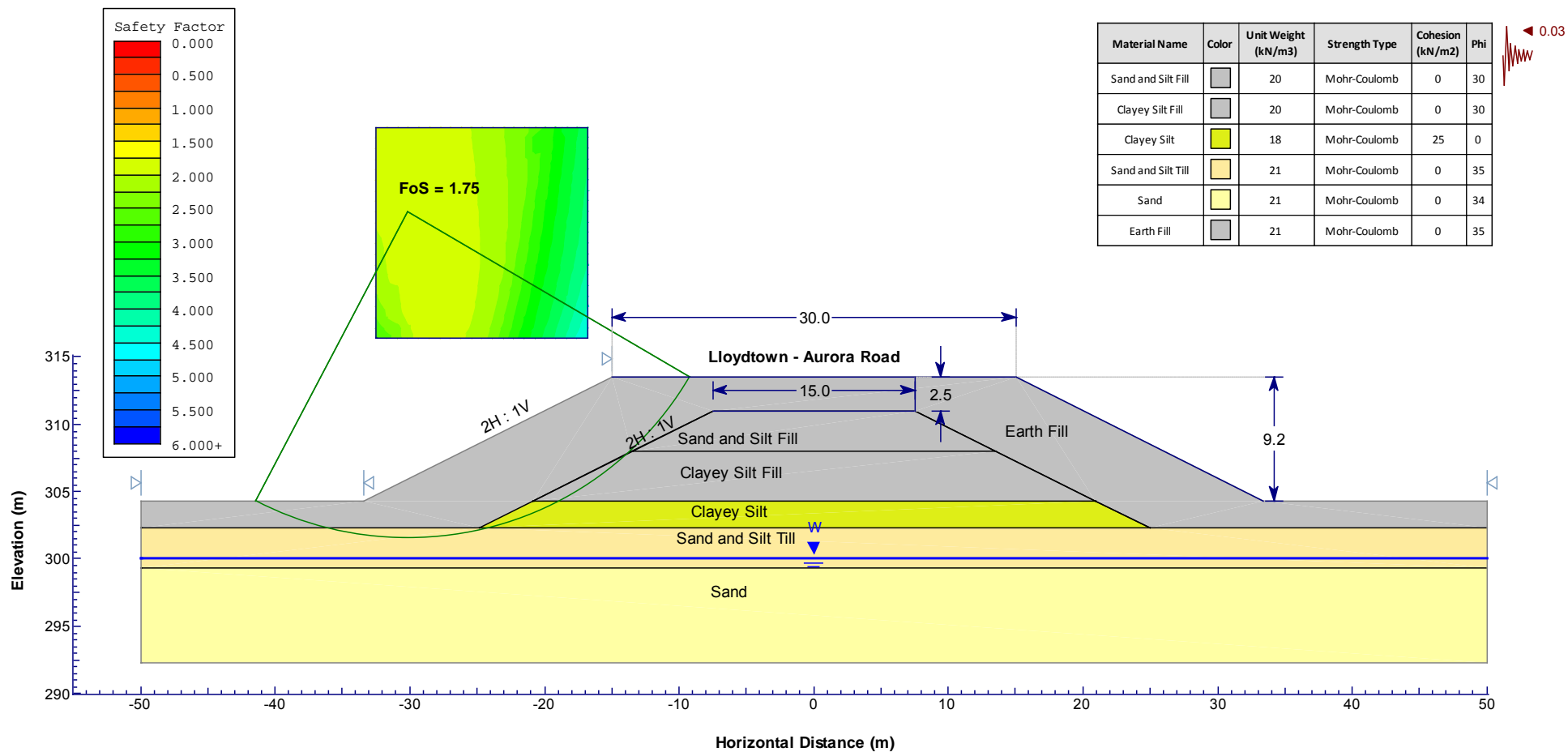
Figure 7





# Lloydtown - Aurora Road Underpass – Hwy 400 Widening East Approach Embankment Seismic Global Stability

Figure 8





# **APPENDIX A**

## **Record of Borehole Sheets and Laboratory Test Results**



## 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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
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.).

<b>PH:</b>	Sampler advanced by hydraulic pressure
<b>PM:</b>	Sampler advanced by manual pressure
<b>WH:</b>	Sampler advanced by static weight of hammer
<b>WR:</b>	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) 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 <sub>4</sub>	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

Percent 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 (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



## 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

### 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

#### (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

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

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

PROJECT		RECORD OF BOREHOLE		No LA1		SHEET 1 OF 1		METRIC						
G.W.P. 09-1111-0018		LOCATION		N 4873466.0 ; E 297858.4		ORIGINATED BY		CS						
DIST Central HWY 400		BOREHOLE TYPE		108 mm Outside Diameter Continuous Flight Solid Stem Auger		COMPILED BY		SKB						
DATUM Geodetic		DATE		October 25, 2010		CHECKED BY		SMM						
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						
304.4	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100					
0.0	TOPSOIL													
0.2	Clayey silt, some sand, trace gravel, containing rootlets (FILL) Firm Brown Moist		1	SS	8		304							
			2	SS	7									
303.0							303							
1.5	CLAYEY SILT with SAND, trace gravel (TILL) Very stiff to hard Brown Moist		3	SS	19									1 31 50 18
			4	SS	44		302							
301.4														
3.0	SAND and SILT to Silty SAND, trace clay, trace gravel, some sand pockets and sand seams (TILL) Very dense Brown Moist		5	SS	84		301							
			6	SS	89		300							4 53 37 6
			7	SS	160									
							299							
298.2			8A											
297.9	CLAYEY SILT, some sand, trace gravel (TILL) Hard Brown Moist		8B	SS	179/28		298							5 66 23 6
6.6	END OF BOREHOLE													
NOTE: 1. Water level in open borehole at a depth of 5.6 m below ground surface (Elevation 298.8 m) upon completion of drilling.														



PROJECT 09-1111-0018		<b>RECORD OF BOREHOLE No LA2</b>		SHEET 1 OF 2		<b>METRIC</b>	
G.W.P. 2835-02-00		LOCATION N 4873500.8 ; E 297864.6		ORIGINATED BY CS/TT			
DIST Central HWY 400		BOREHOLE TYPE 210 mm Outside Diameter Continuous Flight Hollow Stem Auger		COMPILED BY SKB			
DATUM Geodetic		DATE October 26-28, 2010		CHECKED BY SMM			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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>		
							20	40	60	80	100						
310.8	GROUND SURFACE																
0.0	ASPHALT																
0.3	GRANULAR FILL																
	Silty sand, trace clay, trace gravel (FILL)																
	Compact																
	Brown																
	Moist																
309.4			1	SS	17												
1.5	Sand and silt, trace to some clay, trace gravel (FILL)																
	Compact to very dense																
	Brown																
	Moist																
	- Occasional sandy silt and clayey silt layers/pockets between the depths of 1.5 m and 5.6 m (Elev. 309.3 m and 305.2 m)																
			2	SS	47												
			3	SS	35												
			4	SS	97/23												
			5	SS	38												
			6	SS	63												
			7	SS	13												
	- Containing rootlets and sand seams/pockets between the depths of 5.6 m and 7.2 m (Elev. 305.2 m and 303.6 m)																
303.6																	
7.2	SAND and SILT, trace clay, occasional silty sand layers																
	Compact																
	Grey																
	Moist																
302.1			8	SS	18												
8.7	CLAYEY SILT, trace to some sand, trace gravel, sand lenses (TILL)																
	Hard																
	Brown																
	Moist																
300.6			9	SS	35												
10.2	SAND and SILT, trace to some clay, trace gravel (TILL)																
	Very dense																
	Brown																
	Moist																
	- Augers grinding between 11.3 m and 12.2 m depth																
			10	SS	93												
			11	SS	81/15												
			12	SS	9*												
296.0																	
14.8																	

Continued Next Page

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

GTA-MTO 001 0911110018.GPJ GAL-GTA.GDT 11/22/12 SIB

PROJECT		RECORD OF BOREHOLE		No LA2		SHEET 2 OF 2		METRIC							
G.W.P. 09-1111-0018		LOCATION		N 4873500.8 ; E 297864.6		ORIGINATED BY		CS/TT							
DIST Central HWY 400		BOREHOLE TYPE		210 mm Outside Diameter Continuous Flight Hollow Stem Auger		COMPILED BY		SKB							
DATUM Geodetic		DATE		October 26-28, 2010		CHECKED BY		SMM							
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 $\gamma$ 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							
	--- CONTINUED FROM PREVIOUS PAGE ---														
291.9	SAND, some silt Very dense Brown Wet		13	SS	98/23										
			14	SS	117										0 84 14 2
			15	SS	126										0 84 14 2
292															
18.9	END OF BOREHOLE														
	NOTES:  1. * SPT "N" Value considered to be affected by sample disturbance due to groundwater inflow to borehole.  2. A hydrostatic head of water and drilling fluid was required inside the augers at a depth of 15.2 m below ground surface (Elev. 295.6 m) in order to advance the borehole due to "blowing " sands.  3. Water level in open borehole at a depth of 12.3 m below ground surface (Elev. 298.5 m) during drilling on October 27, 2010.  4. Borehole advanced using drilling mud; water level not measured upon completion of drilling as it is not reflective of in-situ water conditions.  5. Water level measurement in the piezometer: Date      Depth (m)      Elev. (m) 11/25/10      1.8      309.0 12/02/10      1.7      309.1  6. Piezometer damaged - unable to obtain water level reading on May 27, 2011.														

PROJECT 09-1111-0018		<b>RECORD OF BOREHOLE No LA3</b>		SHEET 1 OF 1		<b>METRIC</b>	
G.W.P. 2835-02-00		LOCATION N 4873501.2 ; E 297910.1		ORIGINATED BY TT			
DIST Central HWY 400		BOREHOLE TYPE 108 mm Outside Diameter Continuous Flight Solid Stem Auger, Wash Boring		COMPILED BY SKB			
DATUM Geodetic		DATE November 2, 2010		CHECKED BY SMM			

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 (%)				
								20	40	60	80	100	w <sub>p</sub>	w	w <sub>L</sub>		
305.0	GROUND SURFACE																
0.0	ASPHALT																
304.7																	
0.4	Sand, some silt, trace gravel, trace clay, containing pockets of clayey silt and organics (FILL) Loose Brown Moist to wet		1	SS	6												
			2	SS	5												
302.8																	
2.2	SAND and SILT, trace clay, trace to some gravel, containing sand seams and pockets (TILL) Compact to dense Brown Moist becoming wet below 3.1 m depth		3	SS	14												
			4A	SS	22												
			4B														
			5	SS	46												
			6	SS	49												
298.6			7	SS	44												
6.5	SAND, some silt Dense to very dense Brown Moist																
			8	SS	97												
295.3			9	SS	61												
9.7	END OF BOREHOLE																
	NOTE:  1. A hydrostatic head of water and drilling fluid was required inside the hollow stem augers at a depth of 6.7 m below ground surface (Elev. 298.3 m) in order to advance the borehole due to "blowing " sands; water level could not be determined upon completion of drilling.																

PROJECT <u>09-1111-0018</u>		<b>RECORD OF BOREHOLE No LA4</b>		SHEET 1 OF 2		<b>METRIC</b>	
G.W.P. <u>2835-02-00</u>		LOCATION <u>N 4873526.5 ; E 297904.2</u>		ORIGINATED BY <u>TT</u>			
DIST <u>Central</u> HWY <u>400</u>		BOREHOLE TYPE <u>210 mm Outside Diameter Continuous Flight Hollow Stem Auger, Wash Boring</u>		COMPILED BY <u>SKB</u>			
DATUM <u>Geodetic</u>		DATE <u>November 1-2, 2010</u>		CHECKED BY <u>SMM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT  $\gamma$ 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
304.8	GROUND SURFACE																			
0.0	ASPHALT																			
304.5																				
0.4	Sand, some silt, trace gravel, trace clay, containing clayey silt layers (FILL) Loose to compact Brown Moist		1	SS	12		304													
303.0							303													
1.8	Clayey silt, trace to some sand, trace gravel (FILL) Firm Brown Moist		2	SS	4															
			3	SS	5		302													
301.6																				
3.2	SAND and SILT, trace gravel, trace clay (TILL) Compact to very dense Brown Moist		4	SS	17		301													
			5	SS	61															
							300													
			6	SS	101															
							299													
	- Containing sand pockets between the depths of 6.1 m and 6.7 m (Elev. 298.7 m and 298.1 m)		7	SS	84															
							298													
							297													
296.8			8	SS	53															
8.0	SAND, some silt, trace clay to SAND and SILT Very dense Brown Wet						296													
			9	SS	57		295													
							294													
			10	SS	61															
							293													
			11	SS	101		292													
							291													
			12	SS	87															
290.0							290													
14.8																				

Continued Next Page

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

GTA-MTO 001 0911110018.GPJ GAL-GTA.GDT 11/22/12 SIB

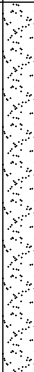

PROJECT		09-1111-0018		RECORD OF BOREHOLE No LA4		SHEET 2 OF 2		METRIC									
G.W.P.		2835-02-00		LOCATION		N 4873526.5 ; E 297904.2		ORIGINATED BY									
DIST		Central HWY 400		BOREHOLE TYPE		210 mm Outside Diameter Continuous Flight Hollow Stem Auger, Wash Boring		COMPILED BY									
DATUM		Geodetic		DATE		November 1-2, 2010		CHECKED BY									
								SMM									
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									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
	CLAYEY SILT, trace to some sand, trace gravel Hard Grey Moist		13	SS	72												
287.4			14	SS	87												
17.4	END OF BOREHOLE																
	NOTE: 1. A hydrostatic head of water and drilling fluid was required inside the augers at a depth of 6.5 m below ground surface (Elev. 298.3 m) in order to advance the borehole due to "blowing" sands; water level could not be determined upon completion of drilling.																

PROJECT		RECORD OF BOREHOLE		No LA5		SHEET 1 OF 2		METRIC															
G.W.P. 09-1111-0018		LOCATION		N 4873525.8 ; E 297942.3		ORIGINATED BY		TT															
DIST Central HWY 400		BOREHOLE TYPE		210 mm Outside Diameter Continuous Flight Hollow Stem Auger, Wash Boring		COMPILED BY		SKB															
DATUM Geodetic		DATE		October 29 & November 19, 2010		CHECKED BY		SMM															
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			20	40						60	80	100	20	40	60	80	100	10
311.0	GROUND SURFACE																						
0.0	ASPHALT																						
0.1	Sand and silt, trace to some clay, trace gravel (FILL) Compact Brown Moist		1	SS	23																		
			2	SS	24																		
			3	SS	17																		
308.0																							
3.0	Clayey silt with sand, trace gravel, containing rootlets and sand and silt pockets (FILL) Stiff Brown Moist		4	SS	9																		
			5	SS	8																		
			6	SS	11																		
305.9																							
5.1	Sandy silt, trace gravel (FILL) Brown to grey Moist																						
305.4																							
5.6	Clayey silt with sand, containing sandy silt seams (FILL) Stiff Brown to grey Moist		7A																				
			7B	SS	14																		
304.4																							
6.6	CLAYEY SILT, trace to some sand, trace gravel, containing rootlets and organics between the depths of 6.6 m and 8.1 m (Elev. 304.4 m and 302.9 m) Very soft Grey Moist		8A																				
			8B																				
302.3																							
8.7	SAND and SILT, trace to some clay, trace gravel (TILL) Dense to very dense Brown Moist		9	SS	44																		
			10	SS	61																		
299.3																							
11.7	SAND, some silt Very dense Brown Moist		11	SS	69																		
298.3																							
12.8	Sandy SILT, trace clay Brown Moist																						
297.7																							
13.3	SAND, trace to some silt Very dense Brown Wet		12	SS	68																		

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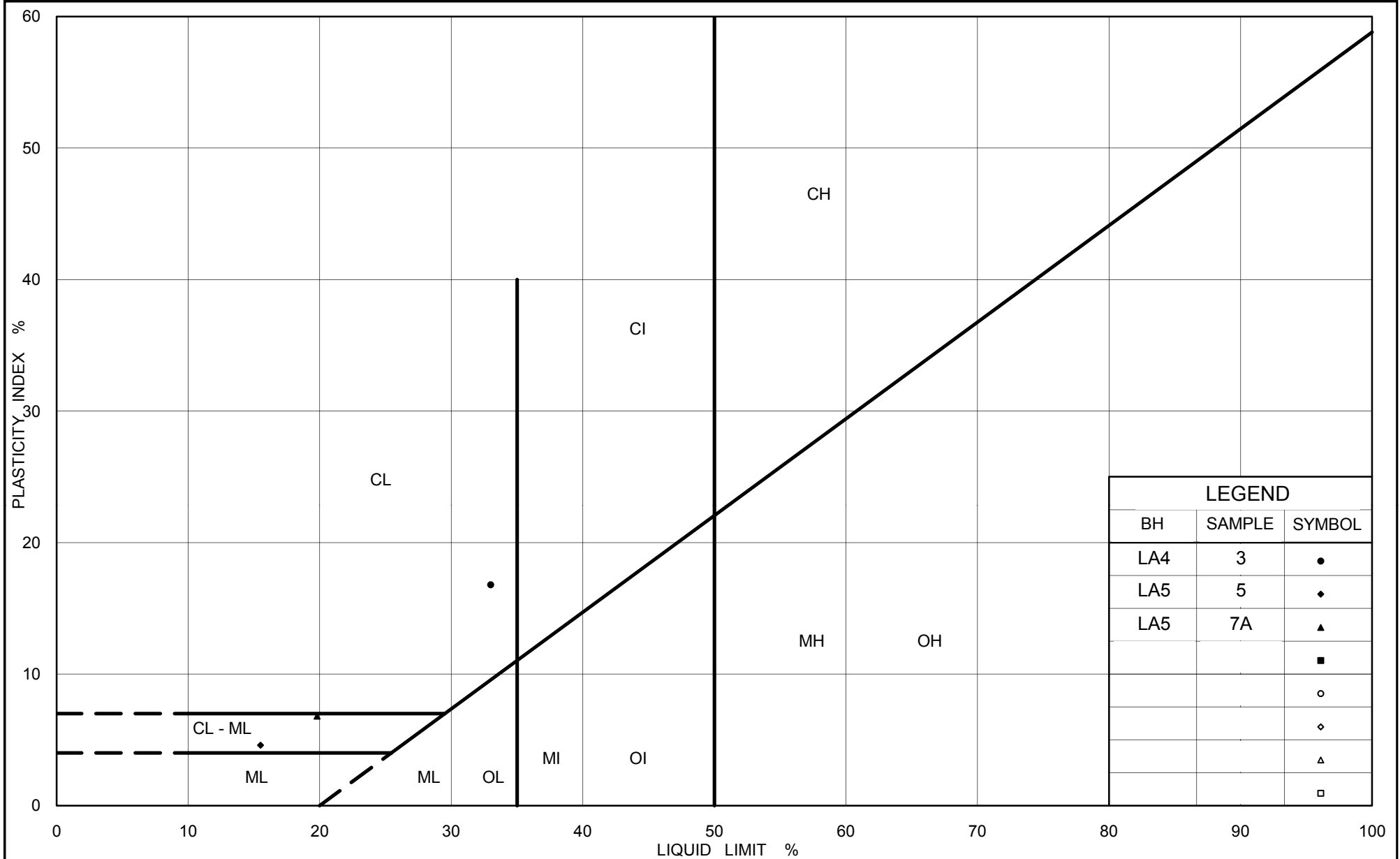
+ <sup>3</sup>, × <sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

GTA-MTO 001 0911110018.GPJ GAL-GTA.GDT 11/22/12 SIB

PROJECT		RECORD OF BOREHOLE		No LA5		SHEET 2 OF 2		METRIC					
G.W.P. 09-1111-0018		LOCATION		N 4873525.8 ; E 297942.3		ORIGINATED BY		TT					
DIST Central HWY 400		BOREHOLE TYPE		210 mm Outside Diameter Continuous Flight Hollow Stem Auger, Wash Boring		COMPILED BY		SKB					
DATUM Geodetic		DATE		October 29 & November 19, 2010		CHECKED BY		SMM					
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)	γ	GR SA SI CL	
--- CONTINUED FROM PREVIOUS PAGE ---													
292.3	SAND, trace to some silt Very dense Brown Wet		13	SS	85		295						
			14	SS	129		294						
							293						
18.7	END OF BOREHOLE NOTE:  1. A hydrostatic head of water and drilling fluid was required inside the augers at a depth of 15.2 m below ground surface (Elev. 295.8 m) in order to advance the borehole due to "blowing" sands; water level could not be determined upon completion of drilling.		15	SS	100/23								



PROJECT		09-1111-0018		RECORD OF BOREHOLE No LA6		SHEET 1 OF 1		METRIC								
G.W.P.		2835-02-00		LOCATION		N 4873563.2 ; E 297951.0		ORIGINATED BY CS								
DIST		Central HWY 400		BOREHOLE TYPE		210 mm Outside Diameter Continuous Flight Hollow Stem Auger		COMPILED BY SKB								
DATUM		Geodetic		DATE		October 22, 2010		CHECKED BY SMM								
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								
304.3	GROUND SURFACE															
0.0	TOPSOIL															
0.2	Silty gravelly sand, trace to some clay, containing rootlets (FILL) Loose to compact Brown Moist		1	SS	7											
303.0			2	SS	20											27 40 25 8
1.3	CLAYEY SILT, some sand, trace gravel (TILL) Stiff Brown Moist		3	SS	13											
302.2																
2.1	SAND and SILT, trace to some clay, trace gravel (TILL) Compact to very dense Brown Moist		4	SS	16											4 39 48 9
			5	SS	32											
			6	SS	64											3 49 40 8
			7	SS	89											
			8	SS	70											
297.6																
6.7	END OF BOREHOLE NOTE:  1. Water level in open borehole at a depth of 5.4 m below ground surface (Elevation 298.9 m) upon completion of drilling.															



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## PLASTICITY CHART

### Clayey Silt Fill

Figure No. 1

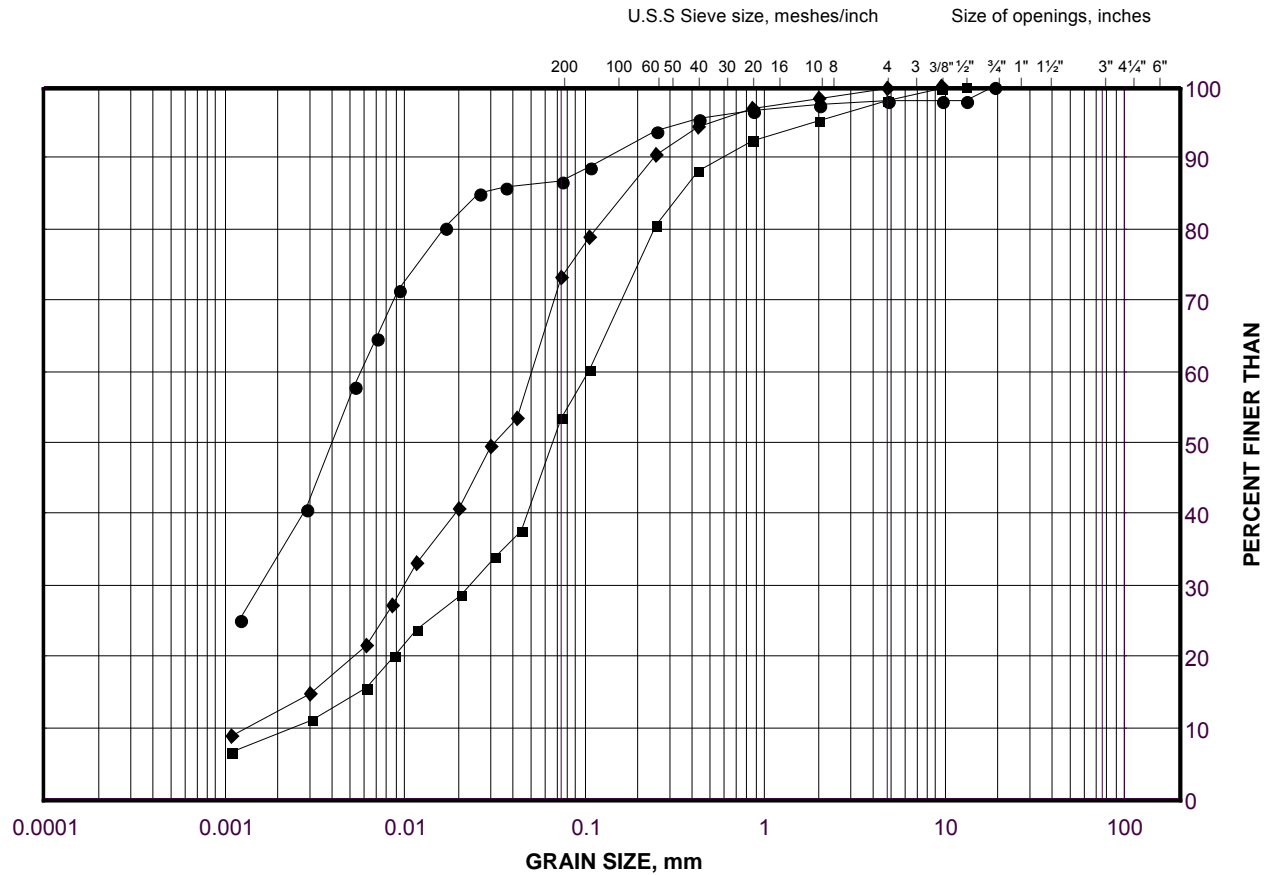
Project No. 09-1111-0018-1

Checked By: SMM

# GRAIN SIZE DISTRIBUTION

Clayey Silt Fill

FIGURE 2



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	LA4	3	302.2
■	LA5	5	306.9
◆	LA5	7A	304.8

Project Number: 09-1111-0018-1

Checked By: SMM

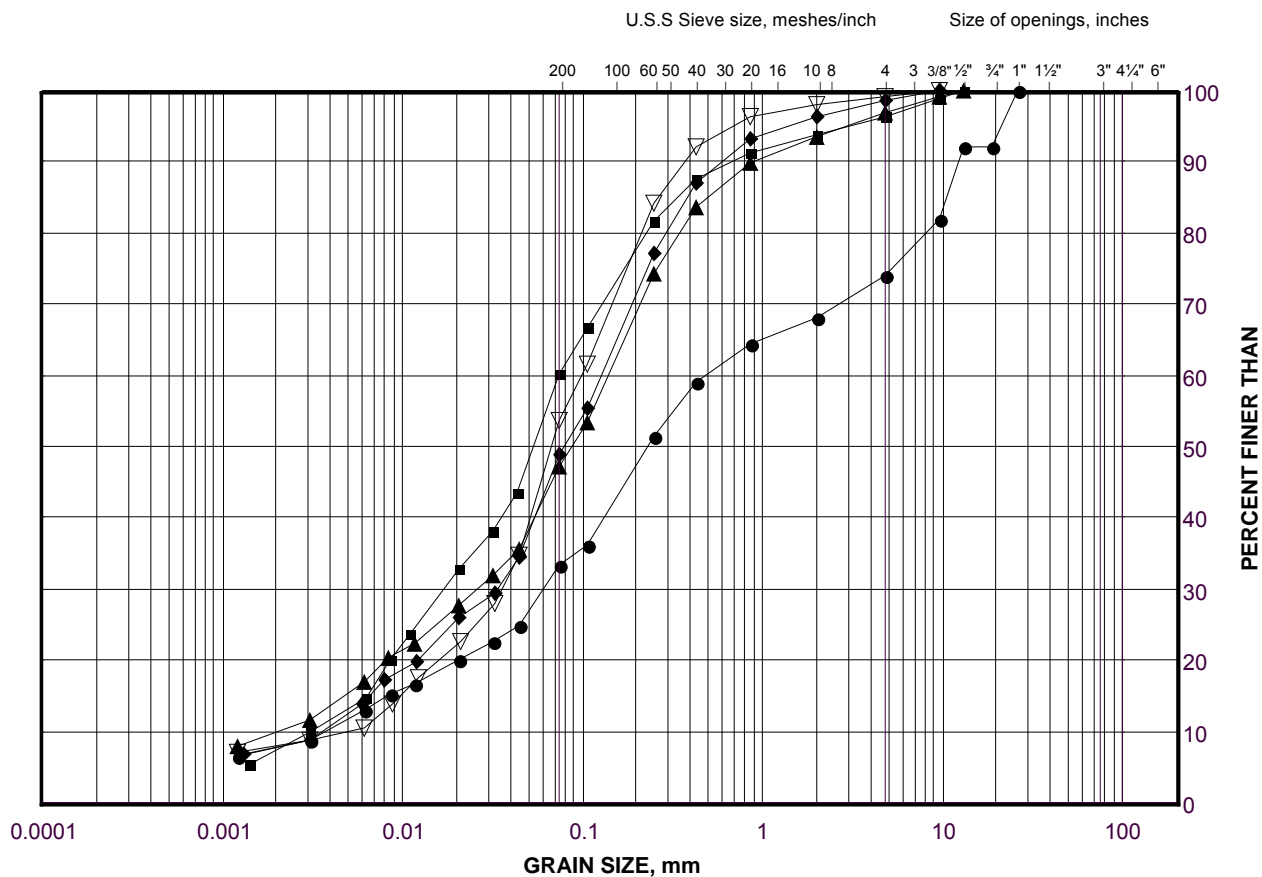
**Golder Associates**

Date: 07-Mar-11

# GRAIN SIZE DISTRIBUTION

Sand and Silt Fill to Silty Gravelly Sand Fill

FIGURE 3



## LEGEND

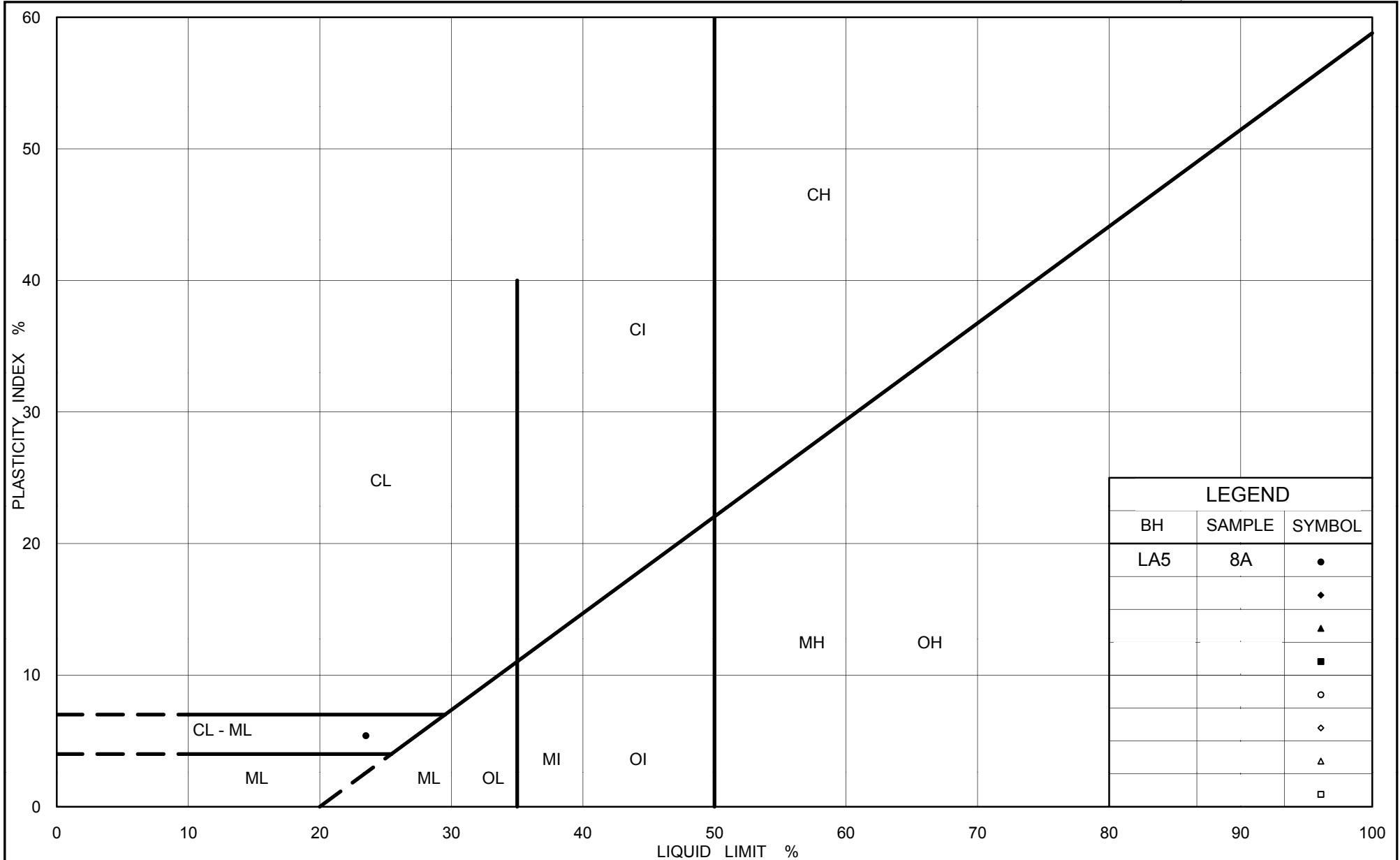
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	LA6	2	303.2
■	LA5	3	308.4
◆	LA2	3	308.2
▲	LA2	6	305.9
▽	LA2	7	304.4

Project Number: 09-1111-0018-1

Checked By: SMM

**Golder Associates**

Date: 07-Mar-11



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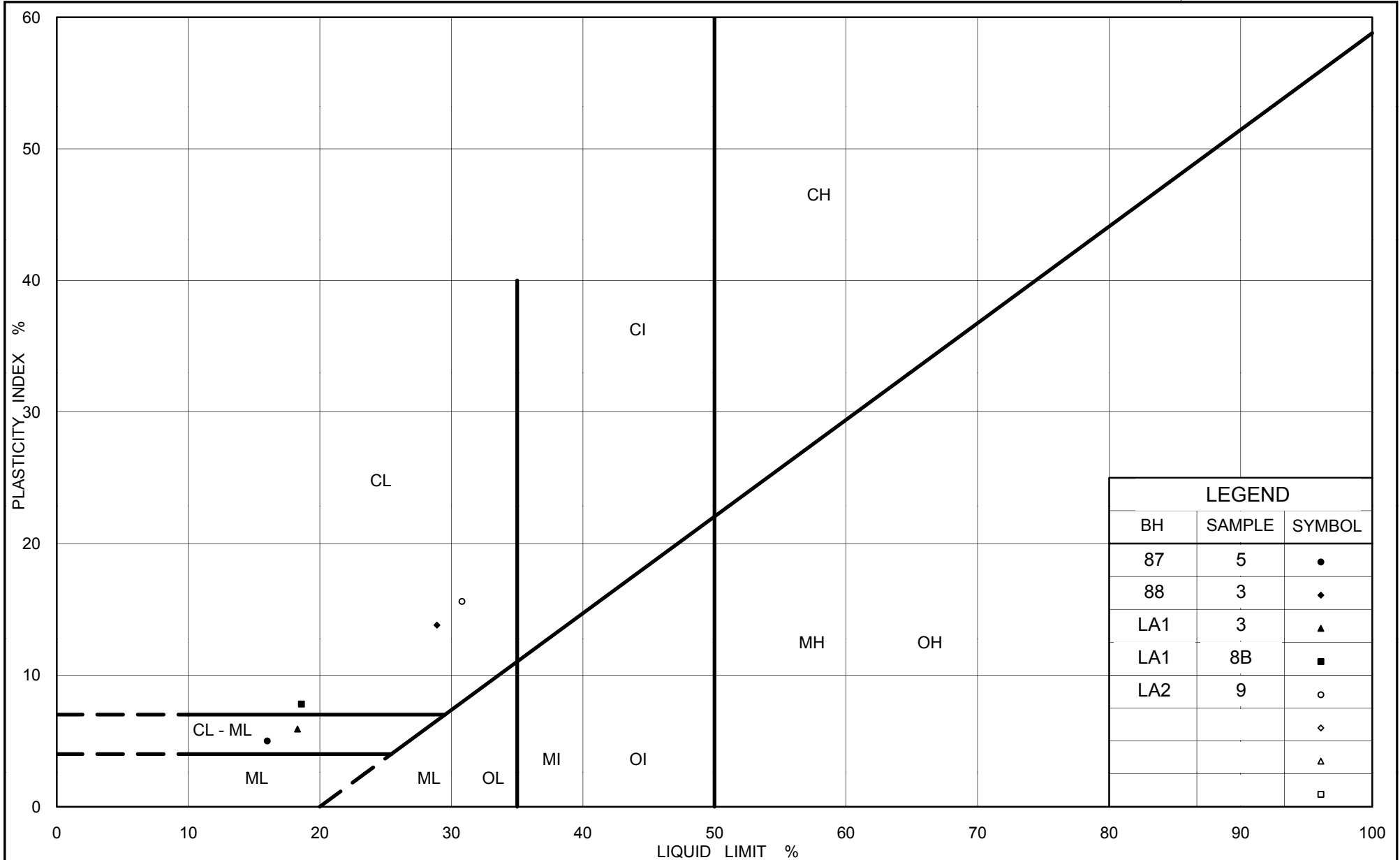
## PLASTICITY CHART

### Clayey Silt (Upper Deposit)

Figure No. 4

Project No. 09-1111-0018-1

Checked By: SMM



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## PLASTICITY CHART

### Clayey Silt Till

Figure No. 5

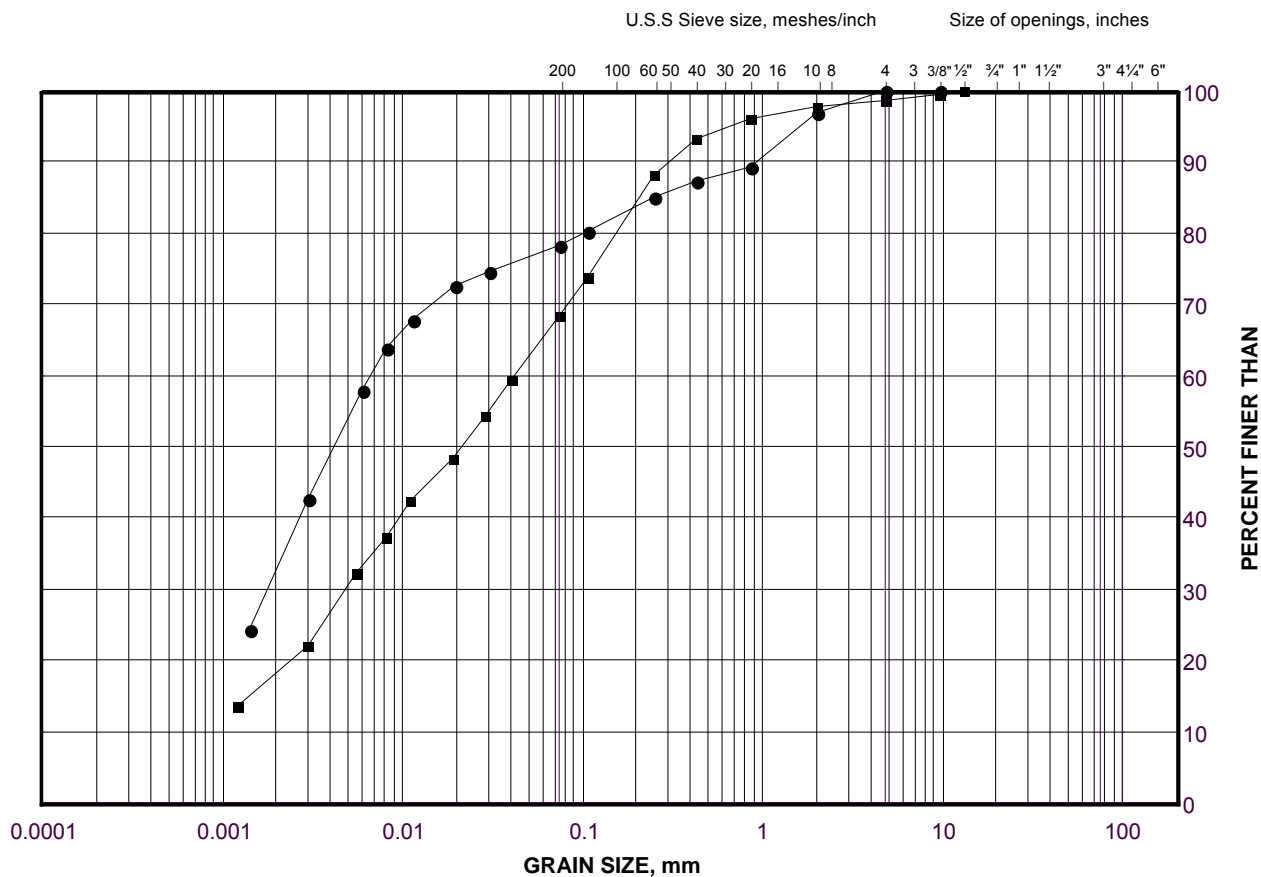
Project No. 09-1111-0018-1

Checked By: SMM

# GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE 6



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	88	3	302.4
■	LA1	3	302.6

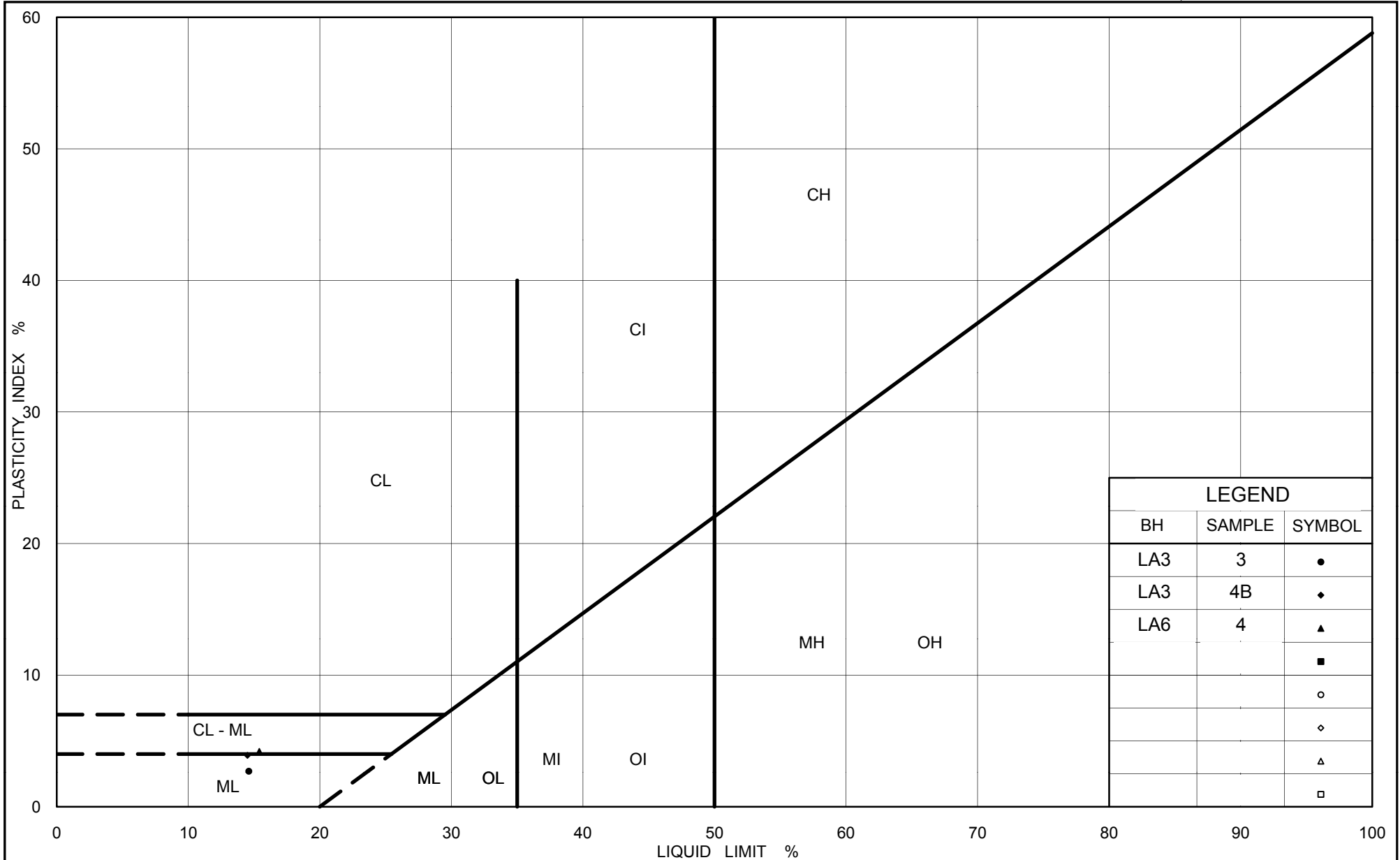
Project Number: 09-1111-0018-1

Checked By: SMM

**Golder Associates**

Date: 07-Mar-11





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## PLASTICITY CHART Sand and Silt Till

Figure No. 7

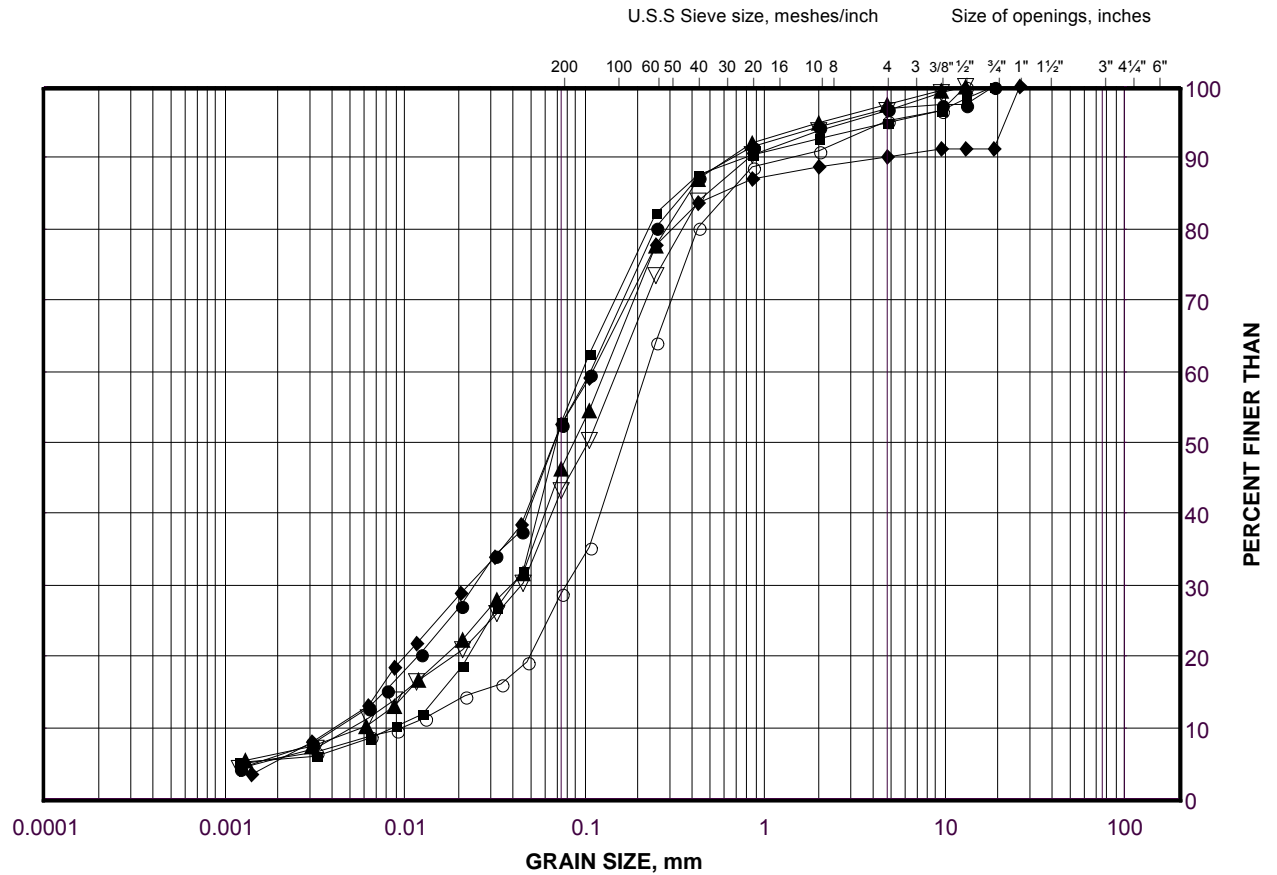
Project No. 09-1111-0018-1

Checked By: SMM

# GRAIN SIZE DISTRIBUTION

Sand and Silt Till to Silty Sand Till

FIGURE 8A



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	LA2	10	299.8
■	LA2	12	296.8
◆	LA3	3	302.4
▲	LA3	6	300.1
▽	LA1	6	300.3
○	LA1	8A	298.2

Project Number: 09-1111-0018-1

Checked By: SMM

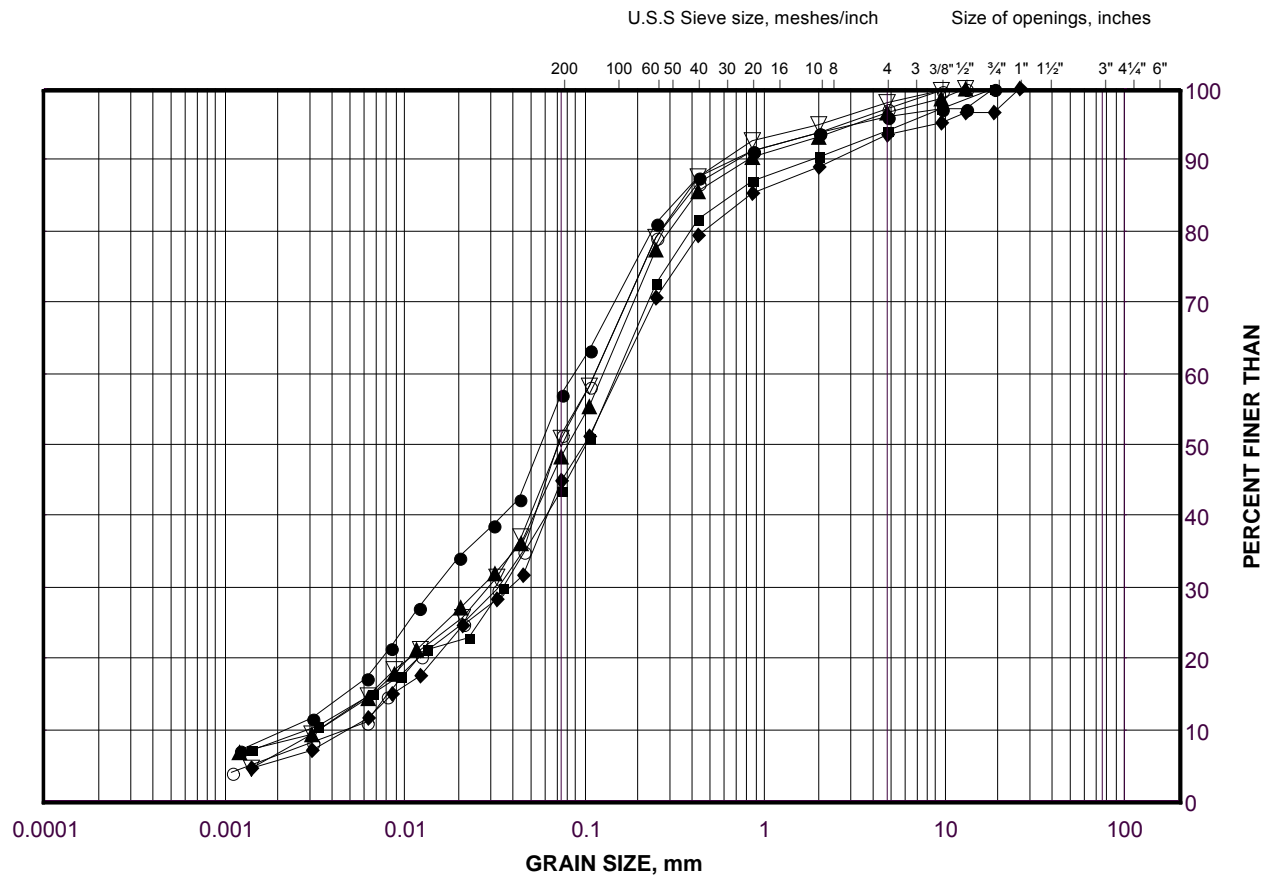
**Golder Associates**

Date: 07-Mar-11

# GRAIN SIZE DISTRIBUTION

Sand and Silt Till to Silty Sand Till

FIGURE 8B



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	LA6	4	301.7
■	88	5	300.9
◆	LA4	5	300.7
▲	LA6	6	300.2
▽	LA4	7	298.4
○	LA5	9	301.6

Project Number: 09-1111-0018-1

Checked By: SMM

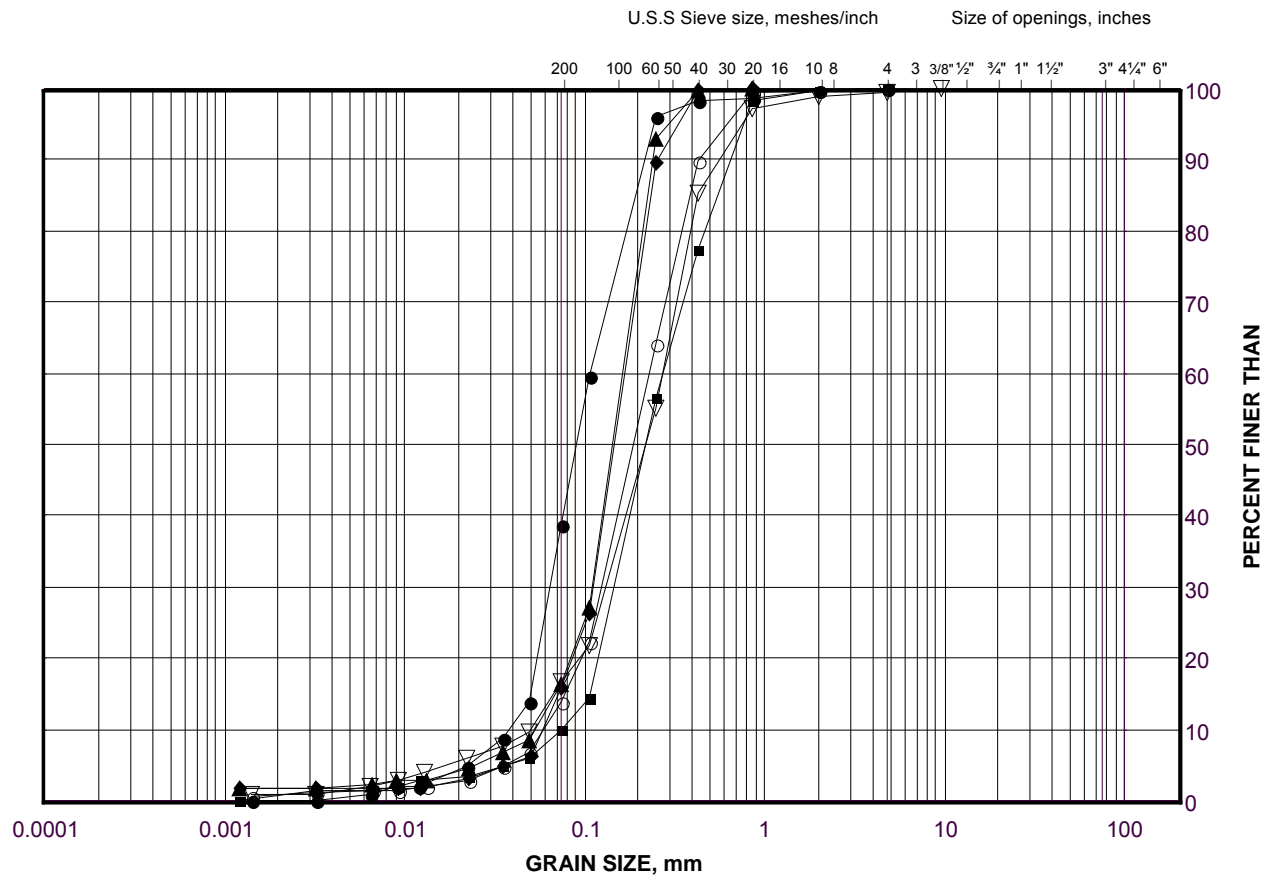
**Golder Associates**

Date: 29-Nov-10

# GRAIN SIZE DISTRIBUTION

Sand to Sand and Silt (Lower Deposit)

FIGURE 9



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

## LEGEND

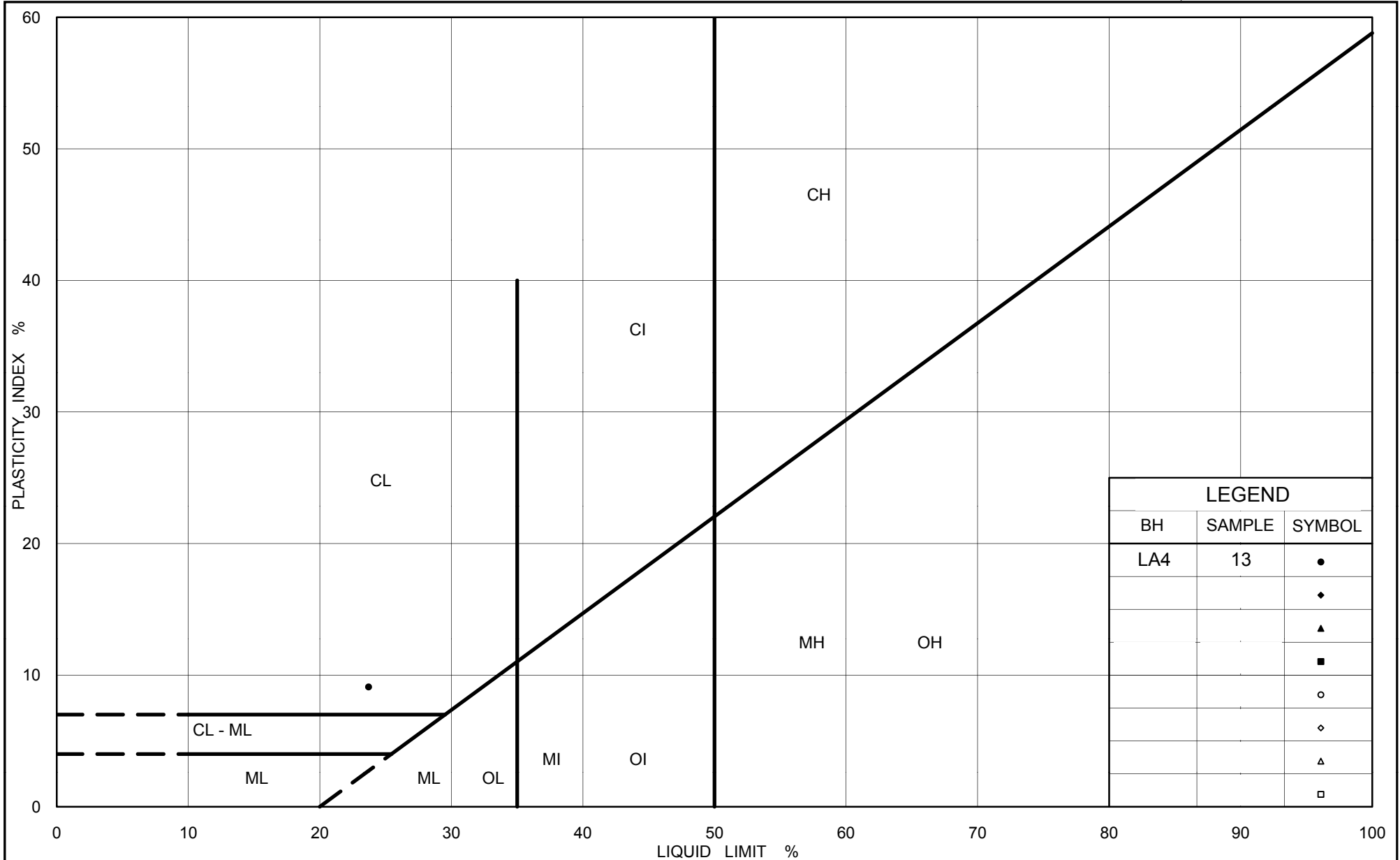
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	LA4	11	292.3
■	LA5	13	295.5
◆	LA2	14	293.7
▲	LA2	15	292.2
▽	LA3	8	297.1
○	LA4	9	295.4

Project Number: 09-1111-0018-1

Checked By: SMM

**Golder Associates**

Date: 07-Mar-11



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Ontario

## PLASTICITY CHART

### Clayey Silt (Lower Deposit)

Figure No. 10

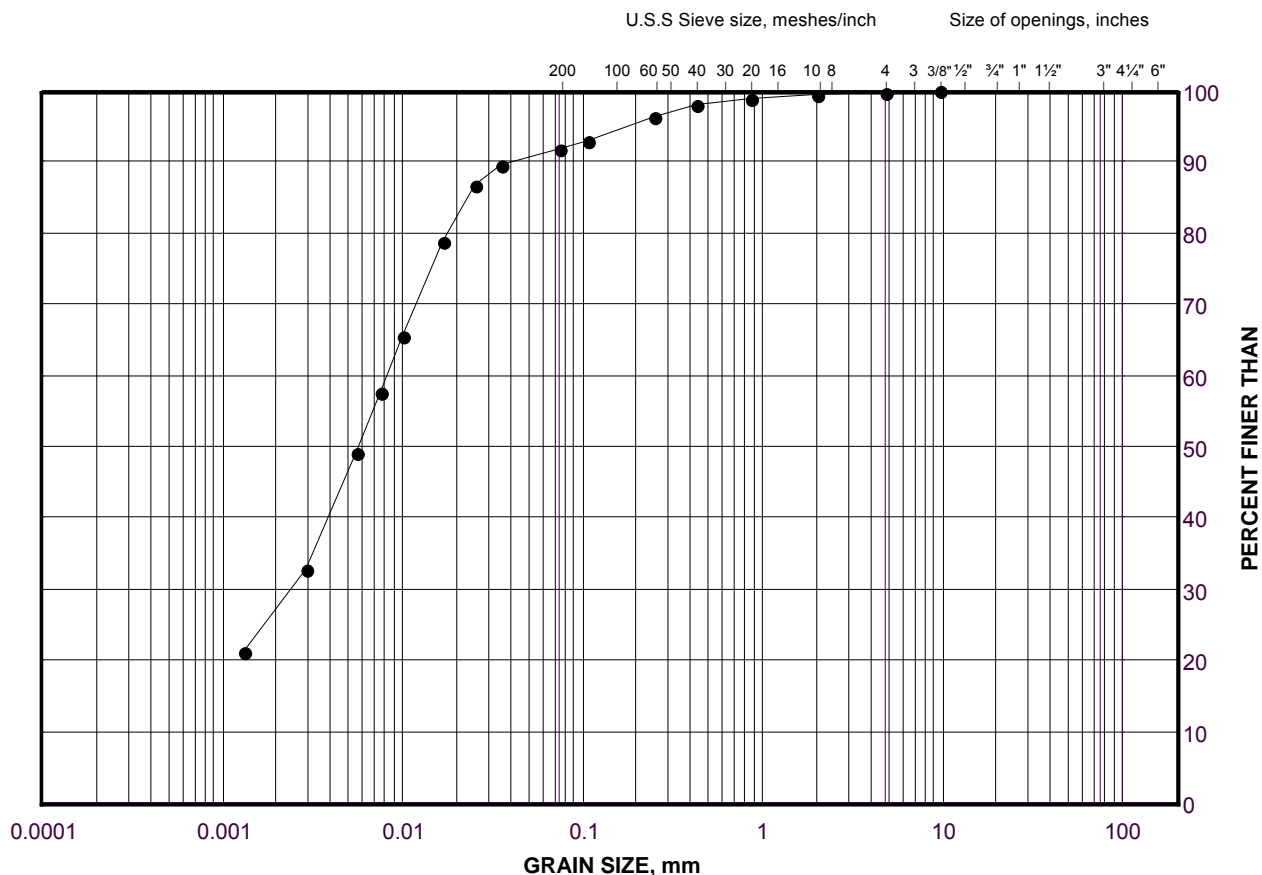
Project No. 09-1111-0018-1

Checked By: SMM

# GRAIN SIZE DISTRIBUTION

Clayey Silt (Lower Deposit)

FIGURE 11



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	LA4	13	289.3

Project Number: 09-1111-0018-1

Checked By: SMM

**Golder Associates**

Date: 07-Mar-11



# **APPENDIX B**

**Record of Boreholes 87 and 88 and Figures 1 and 2, Golder Associates Ltd. Report No. 001-1122F, dated May 2001**

PROJECT 001-1122F				RECORD OF BOREHOLE No 87				1 OF 1		METRIC							
W.P. 222-97-00				LOCATION N 4873552; E 297915				ORIGINATED BY AZ									
DIST Central HWY 400				BOREHOLE TYPE 108mm I.D. Hollow Stem Augers				COMPILED BY LCC									
DATUM Geodetic				DATE October 19, 2000				CHECKED BY ASP									
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									
305.0	GROUND SURFACE						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)					
0.0	Topsoil																
0.2	Sand and Gravel (Fill)																
304.4																	
0.6	Silty Clay, some sand, trace to some gravel, trace black organics Firm to very stiff Brown Moist		1	SS	19		304										
			2	SS	4		303										
			3	SS	8		302										
			4	SS	5		301										
301.1																	
3.9	Clayey Silt with sand, trace gravel (Till) Moist becoming wet at 7.3m depth Hard Brown		5	SS	42		300										
			6	SS	136		299										
	Pockets of sand to silty sand from 6.1m depth.		7	SS	103		298										
			8	SS	139		297										
296.3							296										
8.7	Sand Very dense Brown Wet		9	SS	171/25												
295.5																	
9.6	END OF BOREHOLE																
Note: Water level in open borehole at 7.3m depth (Elev.298.7m) on completion of drilling operations. Water level in piezometer at 6.8m depth (Elev.298.2m) after installation on October 19, 2000. Water level in piezometer at 6.3m depth (Elev. 298.7m) on December 20, 2000. Piezometer destroyed - unable to obtain water level on January 19, 2001.																	

ON MOT 001-1122.GPJ ON MOT.GDT 19/3/01



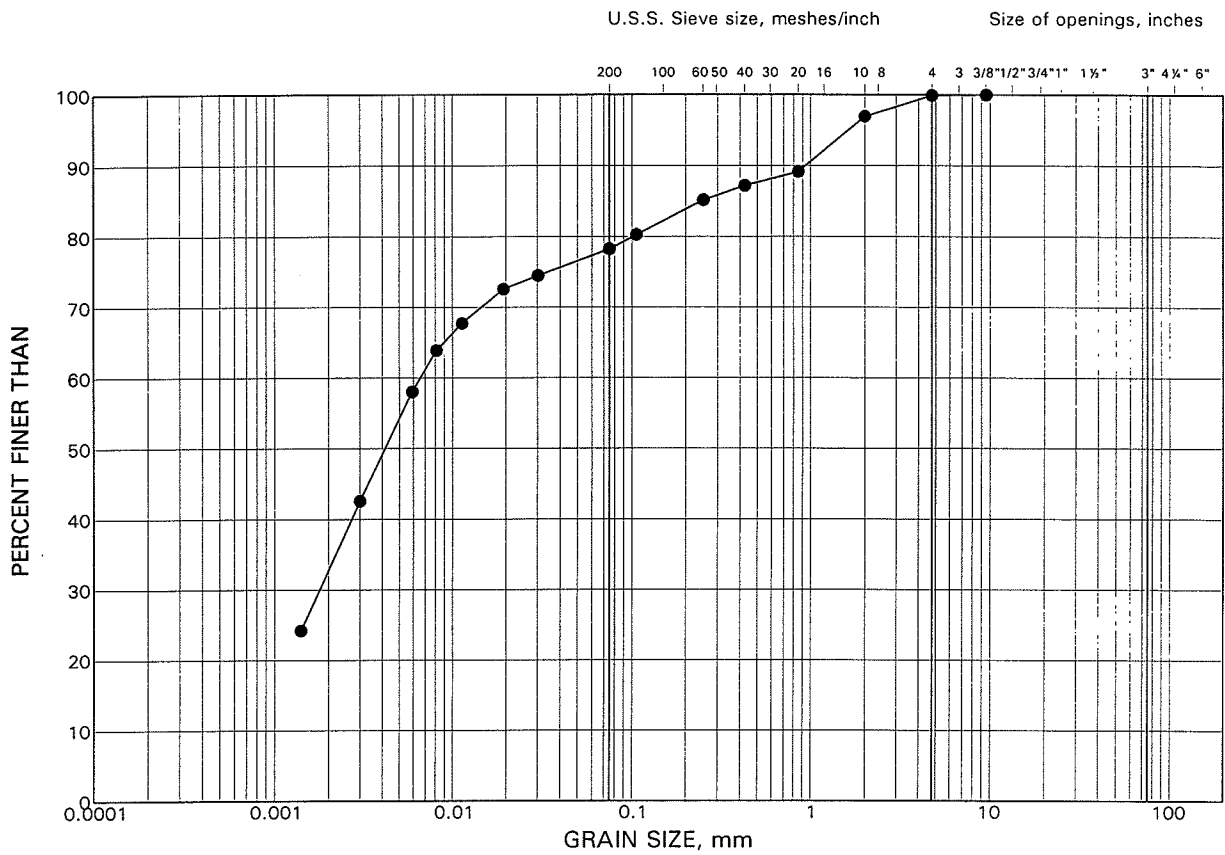
PROJECT 001-1122F				RECORD OF BOREHOLE No 88				1 OF 1		METRIC				
W.P. 222-97-00				LOCATION N 4873483; E 297887				ORIGINATED BY AZ						
DIST Central HWY 400				BOREHOLE TYPE 108mm I.D. Hollow Stem Augers				COMPILED BY LCC						
DATUM Geodetic				DATE October 19, 2000				CHECKED BY ASP						
SOIL 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						
305.0	GROUND SURFACE													
0.0	Topsoil													
0.2	Silty Sand, some gravel, trace clay, trace organics Compact to dense Brown becoming grey at 1.1m depth Moist		1	SS	40									
			2	SS	17									
302.7														
2.3	Silty Clay, trace sand, trace gravel (Till) Stiff to very stiff Mottled brown to brown Moist		3	SS	14									0 22 45 33
			4	SS	29									
301.2														
3.8	Sand and Silt, trace to some gravel, trace to some clay (Till) Very dense Brown Moist		5	SS	107									6 51 35 8
			6	SS	160									
298.7														
6.3	Sand, trace silt, trace gravel Very dense Brown Wet		7	SS	120									
			8	SS	150									
295.4														
9.6	END OF BOREHOLE		9	SS	149									
	Note: Drilling mud was used below 7.5m depth to minimize "blowback" of sands into hollow stem augers. Water level on completion of drilling at 5.4m depth (Elev. 299.6m). Water level in piezometer at 5.2m depth (Elev. 299.8m) after installation on October 20, 2000. Piezometer dry on December 20, 2000 and January 19, 2001.													

ON MOT 001-1122.GPJ ON MOT.GDT 20/3/01

# GRAIN SIZE DISTRIBUTION

Silty Clay Till

FIGURE 1



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

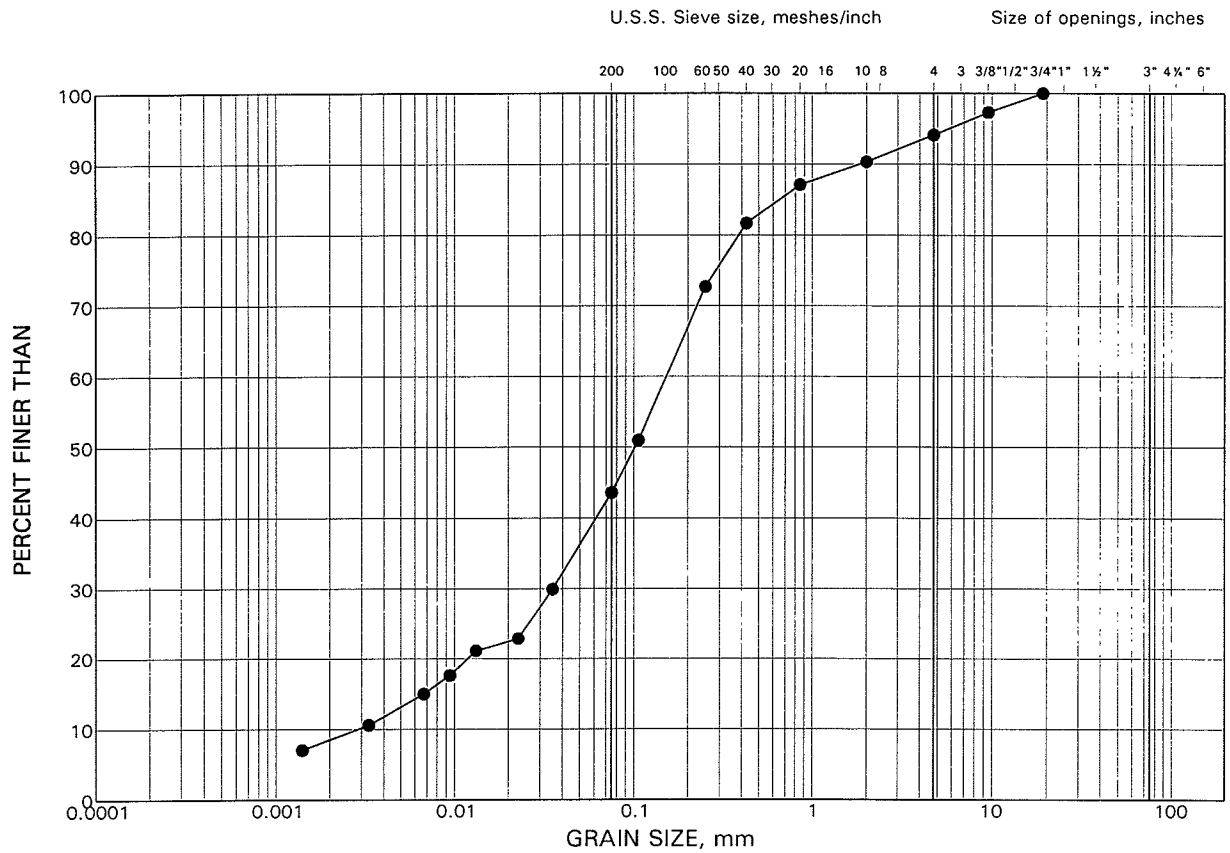
## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	88	3	2.9

# GRAIN SIZE DISTRIBUTION

Sand and Silt Till

FIGURE 2



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	88	5	4.4



# **APPENDIX C**

## **Non-Standard Special Provisions**

## **SUBGRADE PROTECTION - Item No.**

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### **Non-Standard Special Provision**

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The subgrade soils for the footing or pile cap subgrade level may be susceptible to disturbance and loosening from construction traffic and ponded water.

If the concrete for the footings on the native or engineered fill soil cannot be poured immediately after excavation and within three hours of its inspection and approval, a working mat of lean concrete or mass concrete, with minimum thickness of 100 mm, should be placed on the foundation subgrade in general accordance with OPSS 904. The lean concrete shall have a compressive strength of 20 MPa. A minimum 75 mm thick uncompacted levelling pad consisting of Granular 'A' material or fine aggregates (meeting the grading requirements specified in OPSS 1002) should be provided on top of the lean concrete mat.

### **Basis of Payment**

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

## **GROUND AND GROUNDWATER CONTROL - Item No.**

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Non-Standard Special Provision

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Running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons and basal heave could occur where water-bearing cohesionless soils are present at the caisson base. If caisson foundations are adopted for support of any of the foundation elements temporary or permanent caisson liners would be required to support the soils during construction and permit inspection and cleaning of the caisson base. The Contractor is to design and install an appropriate measures to control the groundwater during caisson construction.

### **Basis of Payment**

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

**OBSTRUCTIONS - Item No.**

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**Non-Standard Special Provision**

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The approximately 4.6 m thick sand and silt till deposit encountered at about 10.2 m below ground surface (between about Elevation 300.6 m and 296.0 m) contains cobbles and boulders as indicated in the Record of Borehole sheet LA2. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for driving Steel H-Piles or caissons and pre-augering for deep foundations.

**Basis of Payment**

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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