October 2012

FOUNDATION INVESTIGATION AND DESIGN REPORT

Crown Hill Overpass Replacement
Highway 400 NBL Rehabilitation, Highway 400-Highway 11 Interchange, Simcoe County
GWP 2179-10-00

Submitted to:
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REPORT

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PART A

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the detail design for the proposed replacement of the Crown Hill overpass associated with the overall interchange improvements, in the County of Simcoe, Ontario.

The initial terms of reference and scope of work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) dated May 2008, and in Section 6.8 of MH's *Technical Proposal*. The original terms of reference and scope of work for this portion of the foundation investigation included a contingency item for widening of the existing Highway 400 Northbound Lanes (NBL) ramp structure. Subsequent to the RFP, MTO endorsed the replacement of the existing Highway 400 NBL overpass structure at the Crown Hill interchange on a new alignment. Amended Terms of Reference for the foundation engineering services were provided by MTO on February 22, 2012, and the scope of the revised foundation engineering services was outlined in Golder's letter dated March 2, 2012.

2.0 SITE DESCRIPTION

The existing overpass structure carrying Highway 400 NBL over Highway 11 is located about 3 km north of the Duckworth Street interchange in Barrie, Ontario at the location shown on the Key Plan on Drawing 1. The proposed replacement overpass is located approximately 35 m (centerline to centerline) southwest of the existing overpass structure.

In general, the overall surface topography in the area is gently sloping, and the natural ground surface varies from approximately Elevation 230 m near the southern end of the site, to Elevation 242 m near the northern end of the structure site.

The Highway 400 NBL embankments at this site vary in height from approximately 5 m to 9 m relative to the natural ground surface. The pavement surface of the existing Highway 400 NBL is at approximately Elevation 242 m at the south end of the existing structure, and approximately Elevation 248 m beyond the north end of the existing structure. The existing embankment side slopes are generally oriented at about 2 horizontal to 1 vertical (2H:1V), with the slope faces generally well vegetated.

3.0 SUBSURFACE INVESTIGATION

3.1 Previous Investigation

In March 2011 Golder completed a foundation investigation and design report entitled "Widening of Deep Cuts and High Fill Embankments, Highway 400 NBL Rehabilitation between Highway 11 and Highway 93, Simcoe County, G.W.P. 2039-06-00", March 2011. Several boreholes were advanced near the proposed alignment of the new overpass structure. Boreholes 09-F-6 and 09-F-10 have been included in Appendix A and are used in this report to supplement the information collected during the current investigation. The locations of these boreholes are shown on Drawing 1.

3.2 Current Investigation

The field work for the subsurface investigation for the proposed Crown Hill overpass structure was carried out between March 28 and April 18, 2012, during which time nine boreholes were advanced using both track-mounted and truck-mounted drill rigs, supplied and operated by Canadian Soil Drilling of Midhurst, Ontario. Two boreholes were advanced at each of the north and south abutments (Boreholes 12-01, 12-02, 12-08, and 12-09, respectively), and one borehole was advanced near each of the north and south piers (Boreholes 12-04 and 12-06, respectively). A total of three boreholes were advanced between the abutments and piers to provide information for use by the Contractor for design of temporary falsework support (Boreholes 12-03, 12-05, and 12-07). Due to site access constraints associated with sloping ground conditions adjacent to the existing structure and its approach embankments, several of the boreholes had to be relocated outside of the footprints of the proposed foundation units to permit suitable and safe drilling access. The locations of Boreholes 12-01 to 12-09 are shown on Drawing 1.

The boreholes were advanced to depths ranging from 6.6 m to 34.0 m below existing ground surface using hollow stem auger and mud rotary drilling methods. Soil samples were generally obtained in the boreholes at 0.75 m and 1.5 m intervals of depth (excluding one sample interval of 3.0 m in Borehole No. 12-09 at a depth of 30 m) using 50 mm outer diameter split-spoon samplers driven by a manual hammer, in accordance with the Standard Penetration Test (SPT) procedure.

Each of the foundation boreholes, excluding Borehole 12-09, was terminated after ‘effective refusal’, defined in the MTO Terms of Reference as 3 m of penetration into materials that have Standard Penetration Test (SPT) ‘N’-values greater than 100 blows per 0.3 m of penetration. Borehole 12-09 was terminated at a depth of 34 m due to penetration into a wet, very dense sand and gravel layer that would not permit the drilling rods to be recovered if additional mud-rotary drilling was completed through this layer. The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations, and standpipe piezometers were installed in two boreholes (Boreholes 12-02 and 12-08). The piezometers consist of 50 mm diameter PVC pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled to the ground surface with bentonite pellets. The piezometer installation details and water level readings are indicated on the borehole records contained in Appendix A. Although the groundwater conditions in Borehole 12-08 were not artesian during drilling, the water level was observed in late May 2012 to be above the ground surface at this borehole location (with a very slight trickle of clean water from the piezometer); Golder is arranging to have this borehole decommissioned in accordance with Ontario Regulation 903 (as amended), and details of this decommissioning will be provided in the final Foundation Investigation Report. All remaining boreholes were backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member of Golder’s staff who observed the drilling, sampling and in situ testing operations, and logged the subsurface conditions encountered in the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder’s laboratory in Barrie for further examination. Laboratory testing was completed in Golder’s London office. Index and classification tests consisting of water content determinations, Atterberg limits and grain size distribution analyses were carried out on selected soil samples.

The borehole locations were measured in the field relative to site features and survey staking, and the ground surface elevations were obtained from existing topographical drawings. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized below and are shown on Drawings 1 and 2.

Borehole No.	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
12-01	4,921,041.5	292,822.3	246.8	18.8
12-02	4,921,022.8	292,796.3	237.3	14.0
12-03	4,921,008.7	292,814.8	235.1	6.6
12-04	4,920,996.4	292,832.8	237.2	15.7
12-05	4,920,973.3	292,840.8	237.0	6.6
12-06	4,920,947.6	292,845.8	236.3	24.8
12-07	4,920,922.0	292,859.9	234.0	8.1
12-08	4,920,906.8	292,853.8	233.5	26.4
12-09	4,920,896.1	292,893.8	241.8	34.0
09-F-6	4,920,849.8	292,830.3	232.0	12.7 + DCPT
09-F-10	4,921,092.7	292,763.1	248.0	21.7

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 400 is located within the physiographic region known as the Simcoe Uplands, according to *The Physiography of Southern Ontario* (Chapman and Putnam, 1984).

The general topography within the Simcoe Uplands consists of sloping till or moraine plains (Ontario Geological Society, 1991). The surficial soils in this region consist of sandy silt to sand and gravel, representing shoreline deposits of a former glacial lake that once flooded the area, overlying a glacial till deposit. Surficial deposits of clayey silt to silty clay are also present adjacent to current and former streams.

4.2 Overview of Subsurface Conditions

A summary of the subsurface conditions at the Crown Hill overpass site is provided below. Appendix A provides borehole records that show the detailed subsurface soil and groundwater conditions encountered in each borehole and the results of in situ and laboratory testing. Appendix B provides a summary of the geotechnical laboratory test reports.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic sections and profiles on Drawings 1 and 2 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils encountered in the boreholes consist of surficial topsoil and fill, which are in turn underlain by native deposits that are predominantly cohesionless (granular) in nature. These granular deposits are interlayered with variable tills and soft to hard (but predominantly firm to stiff) clayey silt to silty clay deposits,. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

Approximately 100 mm to 200 mm of topsoil was encountered immediately below the existing ground surface in Boreholes 12-02, 12-03, 12-07, 12-08, and 09-F-6.

4.2.2 Fill

Each of the boreholes, excluding Boreholes 12-02, 12-03 and 09-F-6, encountered granular embankment fill. As the boreholes were advanced both at the Highway 400 NBL embankment level and at the Highway 11 level, the elevations of the surface of the fill materials are highly variable. The granular fill was encountered for a maximum thickness of 7.6 m (Elevation 240.4 m) in Borehole 09-F-10, which was advanced through the Highway 400 NBL embankment.

Borehole No.	Fill Surface Depth (m)	Fill Surface Elevation (m)	Fill Thickness (m)	Base of Fill Elevation (m)
12-01	0.0	246.8	5.7	241.1
12-02	Not encountered			
12-03	Not encountered			
12-04	0.0	237.2	2.9	234.3
12-05	0.0	237.0	1.4	235.6
12-06	0.0	236.3	2.1	234.2
12-07	0.2	233.8	1.2	232.6
12-08	0.2	233.3	1.9	231.4
12-09	0.0	241.8	5.6	236.2
09-F-6	Not encountered			
09-F-10	0.0	248.0	7.6	240.4

The fill materials vary in composition from sand containing trace silt and trace to some gravel, to silty sand containing trace to some clay and gravel. The results of grain size distribution tests completed on five selected samples of the fill are shown on Figure B1 in Appendix B.

The measured Standard Penetration Test (SPT) "N" values within the fill range from 7 to 44 blows per 0.3 m of penetration, indicative of a loose to dense relative density.

4.2.3 Sand to Silt

An extensive cohesionless deposit of sand to sand and silt, although predominantly sand to silty sand, was encountered in all of the boreholes. The deposit was encountered below the fill materials in Boreholes 12-01, 12-04, 12-05, 12-06, 12-07, 12-08, 12-09, and 09-F-10; and below the topsoil in Boreholes 12-02, 12-03, and 09-F-6. The granular deposits are generally extensive and are interlayered with the till and clayey silt to silty clay deposits, as described in the following sections. Each of the boreholes, excluding Boreholes 12-07 and 12-09, were terminated in the cohesionless deposit. The elevations of the surface and base of this deposit and the deposit thickness as encountered in the boreholes are summarized below.

A deposit of silt was encountered within the extensive sand to sand and silt deposit in Boreholes 12-06, 12-07, and 09-F-6. The silt deposit was encountered immediately below sand to silty sand in Boreholes 12-06 and 12-07, and below a clayey silt layer in Borehole 09-F-6. The silt was penetrated in each of the boreholes and has a maximum thickness of 5.9 m in Borehole 12-06. The elevations of the surface and base of this deposit and the deposit thickness as encountered in the boreholes are summarized below.

Borehole No.	Sand to Silty Sand Surface Depth (m)	Sand to Silty Sand Surface Elevation (m)	Sand to Silty Sand Thickness (m)	Sand to Silt Base Elevation (m)
12-01	5.7	241.1	3.1	238.0
	10.3	236.5	> 8.4	Below 228.1
12-02	0.1	237.2	2.0	235.2
	4.0	233.3	> 10.0	Below 223.3
12-03	0.1	235.0	> 6.4	Below 228.6
12-04	2.9	234.3	> 12.8	Below 221.5
12-05	1.4	235.6	> 5.1	Below 230.5
12-06	2.1	234.2	12.6	221.6
	17.8	218.5	> 7.0	Below 211.5
12-07	1.4	232.6	5.7	226.9
12-08	2.1	231.4	0.8	230.6
	5.6	227.9	2.3	225.6
	8.6	224.9	4.6	220.3
	17.8	215.7	3.0	212.7
	22.3	211.2	> 4.1	Below 207.1
12-09	5.6	236.2	7.6	228.6
	14.7	227.1	9.1	218.0
09-F-6	0.1	231.9	0.6	231.3
	2.9	229.1	5.8	223.3
	10.1	221.9	> 2.6	Below 219.3
09-F-10	7.6	240.4	3.2	237.2
	16.8	231.2	> 4.9	Below 226.3

The deposit typically is comprised of sand containing trace to some silt, trace to some gravel, and trace to some clay, to silty sand containing variable amounts gravel and clay, to sand and silt, to silt containing some sand and trace to some clay. Lenses of clayey silt were encountered in the sand deposit in several of the boreholes. In addition, organics (such as topsoil, rootlets or wood fragments) were noted in several of the boreholes on the northern half of the site. Auger grinding, indicative of boulders and cobbles, was noted within the deposit at various depths; additional details in regard to the above noted items are shown on the borehole in Appendix A.

The results of grain size distribution tests carried out on eighteen selected samples of the sand to silt deposit are shown on Figures B2 to B4 in Appendix B. Laboratory testing of two samples containing visible organic matter from Borehole 12-03 indicated that the tested samples had organic contents of approximately 1 per cent and 3 per cent expressed as a percentage of the dry weight of the soil.

The measured SPT "N" values in the sand to silt deposit range from 1 blow per 0.3 m of penetration to 172 blows per 0.23 m of penetration, although the values of less than about 5 blows per 0.3 m of penetration are considered to have been affected by sample disturbance due to groundwater inflow to the borehole. These results indicate this deposit has a loose to very dense relative density, but that it is typically compact to very dense. The SPT "N" values generally increase with depth.

4.2.4 Gravelly Sand to Sand and Gravel

A deposit of gravelly sand to sand and gravel was encountered in Boreholes 12-02, 12-08, 12-09, 09-F-6, and 09-F-10. Borehole 12-09 was terminated in the deposit at a depth of 34.0 m (Elevation 207.8 m). The elevations of the surface and base of this deposit and the deposit thickness as encountered in the boreholes are summarized below.

Borehole No.	Gravelly Sand to Sand and Gravel Surface Depth (m)	Gravelly Sand to Sand and Gravel Surface Elevation (m)	Gravelly Sand to Sand and Gravel Thickness (m)	Gravelly Sand to Sand and Gravel Base Elevation (m)
12-02	2.1	235.2	0.8	234.4
12-08	20.8	212.7	1.5	211.2
12-09	26.9	214.9	3.0	211.9
	32.2	209.6	> 1.8	Below 207.8
09-F-6	8.7	223.3	1.4	221.9
09-F-10	10.9	237.2	2.8	234.3

The gravelly sand to sand and gravel deposit contains trace to some silt and clay. The results of a grain size distribution test carried out on one selected sample of the deposit are shown on Figure B5 in Appendix B.

The measured SPT “N” values within the gravelly sand to sand and gravel range from weight of hammer to 125 blows per 0.15 m of penetration, indicative of a very loose to very dense relative density. However, the lower SPT “N” values are considered to be affected by sample disturbance due to groundwater inflow into the borehole, and the deposit is therefore considered to have a compact to very dense relative density.

4.2.5 Clayey Silt Till to Silty Sand Till

A till deposit was encountered in Boreholes 12-01, 12-02, and 12-09. The till was encountered underlying the sand deposit in Boreholes 12-01 and 12-02, and below the gravelly sand in Borehole 12-09. The till deposits are relatively thin, with a maximum thickness of 2.2 m.

The till deposit is variable in composition and ranges from clayey silt with sand and trace to some gravel, to silty sand containing some gravel, and trace to some clay. The results of grain size distribution tests carried out on two selected samples of the till are shown on Figure B6 in Appendix B.

Atterberg limits testing carried out on two selected samples of the till deposit measured plastic limits of 9 per cent and 11 per cent, liquid limits of 15 per cent and 19 per cent and plasticity indices of 6 per cent and 8 per cent. These results, which are plotted on the plasticity chart on Figure B7 in Appendix B, confirm that the cohesive till deposit consists of clayey silt of low plasticity.

One SPT “N” value of 42 blows per 0.3 m of penetration was measured within the silty sand till, indicative of a dense relative density. The measured SPT “N” values within the clayey silt till deposit range from 14 blows to 22 blows per 0.3 m of penetration, suggestive of a stiff to very stiff consistency.

4.2.6 Clayey Silt to Silty Clay

A clayey silt to silty clay deposit was encountered in Boreholes 12-06, 12-08, 12-09, and 09-F-6 underlying the silty sand to sand deposit; in Borehole 12-07 underlying the silt deposit; and in Borehole 09-F-10 underlying the sand and gravel deposit. Borehole 12-07 was terminated in the clayey silt to silty clay deposit at a depth of 8.1 m (Elevation 225.9 m). The elevations of the surface and base of the clayey silt to silty clay deposits and the deposit thickness encountered at the borehole locations are summarized below.

Borehole No.	Clayey Silt to Silty Clay Surface Depth (m)	Clayey Silt to Silty Clay Surface Elevation (m)	Clayey Silt to Silty Clay Thickness (m)	Clayey Silt to Silty Clay Base Elevation (m)
12-06	14.7	221.6	3.0	218.6
12-07	7.1	226.9	> 1.0	Below 225.9
12-08	2.9	230.6	2.7	227.9
	7.9	225.6	0.7	224.9
	13.2	220.3	4.5	215.8
12-09	13.2	228.6	1.5	227.1
	23.9	217.9	3.0	214.9
09-F-6	0.7	231.3	2.2	229.1

Borehole No.	Clayey Silt to Silty Clay Surface Depth (m)	Clayey Silt to Silty Clay Surface Elevation (m)	Clayey Silt to Silty Clay Thickness (m)	Clayey Silt to Silty Clay Base Elevation (m)
09-F-10	13.7	234.3	3.1	231.2

The deposit is comprised of clayey silt to silty clay containing some sand and gravel in Borehole 12-06, and typically containing sand seams or layers. Cobbles were noted within the deposit in Borehole 09-F-10. The results of grain size distribution tests completed on eight selected samples of the clayey silt to silty clay deposits are shown on Figures B8 and B9 in Appendix B.

Atterberg limits testing was carried out on four selected samples of the deposit and measured plastic limits between 12 per cent and 20 per cent, liquid limits between 22 per cent and 45 per cent, and plasticity indices between 9 per cent and 25 per cent. These results, which are plotted on the plasticity chart on Figure B10, confirm that the deposit consists of clayey silt of low plasticity to silty clay of intermediate plasticity.

The natural water contents measured on samples of the clayey silt to silty clay were between 13 per cent and 42 per cent.

The measured SPT "N" values within the clayey silt to silty clay deposits were variable and ranged from 2 blows to 48 blows per 0.3 m of penetration, suggestive of a soft to hard consistency. In situ vane shear strength testing was carried out in Borehole 09-F-6 and measured an undrained shear strength of approximately 80 kPa.

4.3 Groundwater Conditions

The observed water levels in the each of the open boreholes following completion of drilling are indicated on the borehole records in Appendix A. The water levels measured in the open boreholes and in the two standpipe piezometers are summarized below.

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement	Notes
North Approach Embankment	09-F-10	248.0	237.9	August 10, 2010	Open Borehole
North Abutment	12-01	246.8	236.1	March 28, 2012	Open Borehole
	12-02	237.3	232.7 236.7	April 17, 2012 May 28, 2012	Open Borehole Piezometer
Piers	12-04	237.2	231.1	April 11, 2012	Open Borehole
	12-06	236.3	231.7	April 12, 2012	Open Borehole
South Abutment	12-08	233.5	228.9 > 234.5	April 18, 2012 May 28, 2012	Open Borehole Piezometer
	12-09	241.8	231.8	March 29, 2012	Open Borehole

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement	Notes
South Approach Embankment	09-F-6	232.0	230.2	August 5, 2012	Open Borehole

It is noted that artesian groundwater levels were observed in Borehole 12-08 in the vicinity of the proposed south abutment, associated with the deeper portion of the cohesionless soil deposit (below some confining layers of clayey silt to silty clay). The artesian conditions were not observed during drilling, but were measured on a subsequent site visit once the water level had had time to equilibrate. Based on observations during drilling, sample moisture conditions, the colour change from brown to grey, and the water levels measured in the piezometers, it is anticipated that the "shallow" groundwater level associated with the near-surface cohesionless soil deposits will vary from about Elevation 232 m near the south abutment and south pier, to about Elevation 236.5 m near the north abutment.

The water levels at the abutment and pier sites are expected to fluctuate seasonally in response to changes in precipitation and snow melt, and are expected to be higher during the spring season.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Nick La Posta, P.Eng., and reviewed by Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Fin Heffernan, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.

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PART B

FOUNDATION DESIGN REPORT
CROWN HILL OVERPASS REPLACEMENT
HIGHWAY 400 NBL REHABILITATION
HIGHWAY 400-HIGHWAY 11 INTERCHANGE
SIMCOE COUNTY
G.W.P. 2179-10-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical/foundation recommendations for the detail design of the proposed Highway 400 NBL Crown Hill overpass replacement structure. The recommendations are based on interpretation of the factual data obtained from the present investigation and a past investigation:

- "Foundation Investigation Report, High Fill Embankments, Highway 400 NBL Rehabilitation, Simcoe County, G.W.P. 2039-06-00", prepared by Golder Associates Ltd., dated March 2011.

Where comments are made on construction, they are provided to highlight those aspects that could affect the detail 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.

Based on the General Arrangement (GA) Drawings provided by Morrison Hershfield (MH), the new overpass structure is planned to be a 133 m long, three-span structure (two end spans of 33 m and a central span of 55 m) incorporating two piers. The new overpass will be located west of the existing Highway 400 NBL Crown Hill overpass. In conjunction with the new overpass construction, approach embankment widening/construction for the full height of the overpass will be required both north and south of the proposed three-span structure, where it will tie into the existing Highway 400 NBL embankment.

Boreholes advanced in the vicinity of the proposed Highway 400 NBL Crown Hill overpass encountered an extensive granular deposit consisting of loose to very dense silty sand to sand, as well as loose to dense gravelly sand to sand and gravel, and loose silt; the relative density of the granular deposit generally increases with depth. These granular soils are interlayered with deposits of dense silty sand till, stiff to very stiff clayey silt till, and soft to hard clayey silt to silty clay.

6.2 Foundation Options

Shallow foundations could be adopted at the north abutment and north pier locations. However, based on the presence of firm to stiff clayey silt to silty clay layers and very loose to compact silt with a high groundwater level in the vicinity of the south abutment and south pier, shallow foundations are not considered to be an appropriate option for these foundation elements, and therefore have not been considered further in this report. Only deep foundation options have been considered for support of the abutments and piers for the Highway 400 NBL Crown Hill overpass replacement.

A summary of the advantages and disadvantages associated with each deep foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report. For all three deep foundation options, the artesian groundwater pressures that were observed in the piezometer installed in Borehole 12-08 within the very dense ("100-blow") granular soil at depth will have an impact on the construction and will necessitate the use of a granular drainage blanket below the pile cap for the south abutment and south and north pier locations.

- **Driven steel H-piles:** Driven steel H-piles are suitable and feasible for support of the abutments, piers, and associated wing walls, and would permit integral abutment design. The abutment pile caps could be perched within the approach embankment fill, minimizing excavation through the existing overpass embankment and minimizing dewatering requirements.
- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments and piers, as well as associated wing walls at this site. However, pipe piles are considered to have a slightly higher risk than H-piles for "hanging up" or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders, which were encountered during the subsurface investigation in both the granular deposits and the till deposits.
- **Caissons:** Caissons are feasible for this site but would require the use of temporary or permanent liners given the high risk of "running soil" associated with the water-bearing deposits through which the caissons would be constructed. In addition, artesian groundwater conditions are present at the south abutment location, associated with the lower "100-blow" soil in which the caissons would be founded. Due to the moderate to high potential for disturbance of the founding soils during caisson construction, driven piles are preferred over caissons as a deep foundation option.

The following sections provide recommendations for deep foundations to support the proposed overpass structure. From a foundations perspective, particularly given the artesian groundwater conditions and the resulting potential for disturbance to caissons founded in the very dense aquifer at depth, the preferred option from a geotechnical/foundations perspective is to support the abutments and piers for the replacement structure on driven pile foundations. Deep foundations will minimize the depth of excavation through the existing Highway 400 NBL embankment for the new abutment locations, and will also minimize the total and differential settlement associated with the soft to stiff cohesive deposits and very loose to compact cohesionless soils that underlie the site.

6.3 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

6.3.1 Founding Elevations

The abutments, piers, and associated wing walls may be supported on steel H-piles or steel tube (pipe) piles driven to found within the "100-blow" granular deposits, in either a closed or perched configuration. The following pile tip elevations may be used for design purposes, assuming about 2 m of penetration into the "100-blow" granular deposits. It is noted that "100-blow" material was not encountered at Borehole 12-09; as such the pile tip elevation provided below for the south abutment has been based on the subsurface information from Borehole 12-08 (which is also much closer to the foundation unit for the south abutment).

Foundation Element	Borehole Nos.	Estimated Design Pile Tip Elevation
North Abutment	12-01, 12-02	230 m
North Pier	12-04	223 m
South Pier	12-06	213 m
South Abutment	12-08	211 m

The pile caps should be constructed at a minimum depth of 1.5 m for frost protection purposes, per OPSD 3090.101 *(Foundation Frost Depths for Southern Ontario).* It is noted that the groundwater at in the boreholes was observed to be near the ground surface, between about Elevation 232 m and 236 m. As such, to minimize the requirement for dewatering of the foundation excavations, it is recommended to perch the pile caps for the abutments in the approach embankment fill.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and/or boulders within the soil deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe 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. The piles should be reinforced at the tip with driving shoes (such as Titus Standard "H" Bearing Pile Points) to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 *(Deep Foundations);* this requirement should be noted on the Contract Drawings.

Artesian groundwater pressures have been noted in the piezometer installed in Borehole 12-08 near the south abutment, associated with the deep granular deposit into which piles would be driven. Therefore, specialized construction techniques are recommended at the south abutment, south pier and north pier to mitigate the possible upward flow of water and fine soil particles along the pile shafts. It is recommended that a granular drainage blanket be placed beneath the pile cap to minimize the migration of fines that may be transported along the piles during and after construction. The drainage blanket should consist of a minimum 0.5 m thick layer of concrete fine aggregate, meeting the gradation requirements of OPSS 1002 (Aggregates Concrete). Appropriate drainage should be provided for the granular blanket.

Groundwater control will be required during excavation and construction for the pier footings; further discussion is provided in Section 6.8.

6.3.2 Axial Geotechnical Resistance/Reaction

For HP 310x110 piles driven to the estimated tip elevations provided in Section 6.3.1, the factored axial resistance at Ultimate Limit States (ULS) and the axial geotechnical reaction at Serviceability Limit States (SLS, for 25 mm of settlement) may be taken as follows:

Foundation Element	Borehole Nos.	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
North Abutment	12-01, 12-02	1,600 kN	1,300 kN
North Pier	12-04	1,600 kN	1,300 kN
South Pier	12-06	1,600 kN	1,300 kN
South Abutment	12-08	1,600 kN	1,300 kN

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

Pile installation should be in accordance with OPSS 903 *(Deep Foundations).* The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO Standard Structural Drawing SS-103-11) during the final stages of driving to achieve the appropriate ultimate capacity. Based on MTO experience with the Hiley formula in Southern Ontario, a resistance factor equal to 0.5 may be used on the ultimate resistance to verify the factored ULS design values. The following note from MTO's Structural Manual should be shown on the Contract Drawing, assuming that a resistance factor of 0.5 is applied to the use of the Hiley formula:

- *Piles to be driven in accordance with Standard SS103-11 using an ultimate geotechnical resistance of 3,200 kN per pile.*

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 shown above and at 0.5 m intervals of depth until the ultimate axial resistance 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 given prior to driving the pile below the design pile tip elevation.

6.3.3 Downdrag Load (Negative Skin Friction)

Assuming the use of conventional earth or granular fill, the widened embankment loading will cause consolidation settlement of the firm to stiff clayey silt to silty clay layers at the south abutment (as discussed further in Section 6.6). Negative skin friction or downdrag loads will need to be taken into account in the design of the piles supporting the new south abutment, unless measures to eliminate downdrag loads are adopted as discussed at the end of this section. Downdrag loads do not need to be taken into account at the other foundation elements as firm to stiff clayey soils are not present (north abutment and north pier), and/or the Highway 11 embankment loading will remain similar to existing (north pier and south pier).

In calculating the magnitude of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] were considered. Considering the larger predicted settlement of the clayey silt to silty clay deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the underside of the upper layer of clayey silt to silty clay, around approximately Elevation 227 m to 228 m.

Based on the above, the unfactored downdrag load acting on a single HP 310 x 110 pile over the length of pile within the native soils is estimated to be 250 kN. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the CHBDC.

Downdrag loads could be eliminated with the use of EPS fill as backfill behind the abutments, although this is not considered to be necessary given the relatively small proportion of the downdrag load relative to the axial geotechnical resistance. Consideration could also be given to the use of bitumen coating on the piles to eliminate the downdrag loads; however, the use of bitumen coating increases the pile costs by approximately 20 to 45 per cent depending on the size of the job; for this widening project, it is estimated that the cost increase would be closer to the upper limit.

Due to the consolidation of the firm to stiff clayey silt to silty clay layers under the widened south approach embankment loading, lateral spreading in these deposits has also been considered. Lateral spreading is not considered a significant issue for the design of the piles

6.3.4 Resistance to Lateral Loading

Resistance to lateral loading can 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 (as noted in CHBDC C6.8.7.1):

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where

k_h is the coefficient of horizontal subgrade reaction (MPa/m);
n_h is the constant of subgrade reaction (MPa/m);
z is the depth (m); and
B is the pile diameter (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

where

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
s_u is the undrained shear strength of the soil (kPa); and
B is the pile diameter (m).

The following ranges for the value of n_h and s_u may be assumed in the structural analyses. The soil stratigraphy has been generalized and the values reflect the variability in the subsurface conditions within each foundation element footprint, however, the deposit boundaries vary slightly at the abutments and reference can be made to the borehole records and to the interpreted stratigraphic sections for each foundation element on Drawing 2 to assess the variation.

Foundation Element	Soil Unit	Elevation Interval (m)	n_h (MPa/m)	s_u (kPa)
North Abutment	Loose to dense sand fill/ New embankment fill	Above 241	7	-
	Compact to very dense sand and silt to gravelly sand	241 – 236.5	12	-
	Compact to very dense silty sand to sand and gravel	236.5 – 232	8	-
	Very dense sand and silt to gravelly sand	Below 232	20	-
North Pier	Compact sand fill and compact to dense silty sand to sandy silt	Above 232	6	-
	Compact to dense silty sand to sandy silt	232 – 225	7	-
	Very dense sand	Below 225	20	-
South Pier	Dense sand fill and compact to dense silty sand	Above 232	12	-
	Loose to compact silt to sand	232 – 221.5	4	-
	Stiff to very stiff clayey silt	221.5 – 218.5	-	100
	Very dense silty sand to sandy silt	Below 218.5	20	-
South Abutment	Compact to dense sand to silty sand fill and loose to compact sand to sand and silt	Above 232	6	-
	Loose to dense sand to sand and silt	232 – 230.5	3	-
	Firm to stiff clayey silt	230.5 – 228	-	75
	Loose to compact sand	228 – 226	3	-
	Stiff clayey silt	226 – 225	-	75
	Loose to compact silty sand to sand	225 – 220.5	4	-
	Firm to stiff clayey silt to silty clay	220.5 – 215	-	50
	Very dense silty sand to gravelly sand	Below 215	20	-

For design of a single vertical HP310x110 pile driven to the design pile tip elevation in very dense sand to silty sand as given in Section 6.3.1 above, a maximum factored lateral geotechnical resistance at ULS of 110 kN and a lateral geotechnical reaction at SLS of 40 kN (for 10 mm of lateral displacement at the pile cap level) may be used with reference to Clause C6.8.7.1, Table C6.4, of the Commentary on *CHBDC*.

The upper zone of soil (down to a depth below the pile cap equal to about 1.5 x D (after Broms, 1964, where D = pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during driving.

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 (R)
8D	1.00
6D	0.70
4D	0.40
3D	0.25

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

Due to the consolidation of the firm to stiff clayey silt to silty clay layers under the widened south approach embankment loading (as discussed in Section 6.3.3 and in Section 6.7), lateral spreading in these deposits has also been considered. Lateral spreading is not considered a significant issue for the design of the piles at the south abutment, even if the piles are driven relatively soon after placement of the fill for the south approach embankment widening up to the pile cap underside. However, to minimize the effects of any lateral spreading and lateral load transfer to the piles, consideration could be given to the placement of a 50 mm to 100 mm thick layer of EPS (expanded polystyrene) on the rear side of the south abutment wall.

6.4 Caissons

Caissons socketted into the “100-blow” sand to silty sand deposit could be considered for support of the abutments and piers. However, temporary or permanent liners would be required to support the soils during construction, to minimize disturbance and loss of ground in the water-bearing cohesionless soil zones.

In addition, due to the artesian groundwater conditions observed in the piezometer installed in Borehole 12-08 near the south limit of the proposed structure, there is a high risk of disturbance of the founding soils during caisson construction. Specialized construction techniques would be required, such as the use of drilling mud to minimize disturbance to the soils at the base of the caisson, and the use of tremie techniques for placing concrete. If caisson foundations are adopted, 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.

In addition, as discussed in Section 6.3.1 for driven pile foundations, it is recommended that a granular drainage blanket be placed beneath the pile caps at the south abutment, south pier and north pier, to minimize the migration of fines that may be transported along the caissons during and after construction. The drainage blanket should consist of a minimum 0.5 m thick layer of concrete fine aggregate, meeting the gradation requirements of OPSS 1002 (Aggregates Concrete). Appropriate drainage should be provided for the granular blanket.

6.4.1 Founding Elevations

If caisson foundations are adopted, they should be founded at the design elevations provided in Section 6.3.1 for driven pile foundations, assuming approximately 2 m of penetration into soils having SPT "N" values of greater than 100 blows per 0.3 m of penetration.

6.4.2 Axial Geotechnical Resistance/Reaction

The following factored geotechnical resistances at ULS and geotechnical reactions at SLS (for 25 mm of settlement) may be used for the detail design of caissons founded at the design elevations as given above.

Caisson Diameter	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
1.2 m	4,500 kN	3,500 kN
1.5 m	6,500 kN	5,500 kN

The performance of caissons will depend upon the final cleaning and verification of the subgrade quality at the base of the caissons. Each caisson excavation should be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. The Ontario Occupational Health and Safety Act 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.

6.4.3 Resistance to Lateral Loading

The recommendations for resistance to lateral loading for driven piles as provided in Section 6.3.4, above, may be used for the design of caisson foundations.

6.5 Liquefaction Assessment

The peak zonal acceleration ratio is 0.05g for the City of Barrie, Ontario (CHBDC Table A3.1.1). The Site Coefficient (S) may be taken as 1.2, consistent with Soil Profile Type II in accordance with Section 4.4.6 and Table 4.4 of the CHBDC (2006).

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary*, which correlates the cyclic resistance ratio (CRR) of the soils with their normalised penetration resistance and fines content.

Based on this assessment and assuming a ground acceleration of 0.06g, the subsoils are not considered liquefiable under design earthquake loads.

6.6 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

The following recommendations are made concerning the design of the stems/wing walls. It should be noted that these design recommendations and parameters assume a 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 meeting the specifications of Special Provision (SP) 110S13 (Aggregates) 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. This fill should be compacted in accordance with SP 105S10 (Compaction). 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 OPSD 3101.150 *(Abutment Walls – Backfill)* and 3121.150 *(Retaining Walls – 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 MTO’s Special Provision 105S10 *(Compacting).* Other surcharge loadings should be accounted for in the design, as required.

- 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 (see Case A on Figure C6.20 (a) of the Commentary to the CHBDC), or 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 on Figure C6.20(b) of the Commentary to the CHBDC).

- For Case A, the pressures are based on the proposed embankment fill materials and the existing native soils and the following parameters (unfactored) may be used assuming the use of granular earth fill such as SP 110S13 (Aggregates) Select Subgrade Material (SSM) for embankment construction:

Unfactored Parameters		**New Earth Fill**
Soil unit weight:		21 kN/m^3
Coefficients of static lateral earth pressure:	At rest, K_o	0.47
	Active, K_a	0.31

- For Case B, where the pressures are based on SP110S13 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II fill behind the wall, the following parameters (unfactored) may be assumed:

Unfactored Parameters		Granular A	Granular B Type II
Soil unit weight:		22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:	At rest, K_o	0.43	0.43
	Active, K_a	0.27	0.27

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, 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 structure where the rotational and/or horizontal movement is not sufficient to mobilize the active pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

6.6.1 Seismic Loading Conditions

Seismic loading must be taken into account in accordance with Section 4.6.4 of *CHBDC*, as it can result in increased lateral earth pressures acting on the abutment stem and any associated wing walls/retaining walls.

The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure. The earthquake-induced dynamic pressure distribution is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$P = K\,\gamma'\,d + (K_{AE} - K)\,\gamma'\,(H-d)$$

where
- K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
- K_{AE} is the seismic active earth pressure coefficient;
- γ' is the effective unit weight of the soil (kN/m^3)
  - taken as soil unit weights given above for fill materials
  - taken as 20 kN/m^3 for the native materials
- d is the depth below the top of the wall (m); and
- H is the height of the wall above the toe (m).

According to Table C4.2 of the *Commentary* to the *CHBDC*, this site is located in Seismic Zone 1, and the site-specific zonal acceleration ratio (A) for the Barrie area is 0.05. The site-specific peak ground acceleration (PGA) is 0.031g based on the NRC website; however, the more conservative *CHBDC* value has been used in the assessments presented below. The Site Coefficient (S) may be taken as 1.2, consistent with Soil Profile Type II in accordance with Section 4.4.6 and Table 4.4 of *CHBDC* (2006). Based on the subsurface conditions at the site, a 20 per cent amplification of the ground motion is recommended for design, resulting in an increase in the ground surface acceleration to approximately 0.06g.

The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.06. These coefficients have been determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary.*

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case A	Case B	
	Earth Fill	Granular A	Granular B Type II
Yielding Wall	0.32	0.26	0.26
Non-Yielding Wall	0.37	0.30	0.30

Notes:

1. These seismic K_{AE} values include the effect of wall friction, and assume that the back of the wall is vertical and the ground surface behind the wall is flat.
2. The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.06. This corresponds to displacements of up to approximately 15 mm at this site.

It is noted that for the very low zonal acceleration ratio for this site, the seismic K_{AE} values are similar to or less than the static values of K_a and K_o reported above.

6.7 Approach Embankments

The new Crown Hill overpass structure will be replaced approximately 35 m (centreline to centreline) west of the existing structure. The new Highway 400 NBL ramp embankment will be about 15 m wide at the top, and constructed partially on top of the west side slope of the existing Highway 400 NBL embankment. Based on the GA Drawing provided by MH, it is anticipated that the approach embankments will be a maximum of about 9 m high at the south approach, and a local maximum of about 10 m high (but more typically 5 m to 6 m high) at the north approach.

6.7.1 Subgrade Preparation and Embankment Construction

To improve the performance of the widened portion relative to the existing embankments, it is recommended that all topsoil/organic material and loose surficial fill materials be stripped from the footprint of the proposed approach embankment widening areas. Based on the boreholes results, it is recommended that the upper 200 mm be stripped below the south and north approach embankment areas. The exposed soils should be proof-rolled to identify any additional softened or loosened existing fill materials. Such softened/loosened materials should be removed and replaced with compacted granular fill or select subgrade material.

The fill for construction of the approach embankment widening should consist of granular fill or clean earth fill, in order to minimize settlement of the fill itself. Outside of the granular backfill, the materials used for the approach embankment widening should meet the requirements of OPSS 212 *(Borrow Material),* placed in accordance with OPSS 206 *(Grading)* and OPSS 501 *(Compacting).* OPSS 212 stipulates the field moisture content as an acceptance criteria for the use of earth borrow with more than 50 per cent of the particles smaller than 75 μm as determined by LS-702. If it is impractical to meet the field moisture content acceptance criteria in wet weather conditions, earthwork operations may have to be suspended, or alternatively, the use of coarser-grained cohesionless OPSS 212 material may be necessary.

The new embankment fill materials should be benched into the existing embankment in accordance with OPSD 208.010 *(Benching)*. Transition treatments between the granular and earth fill should be provided in accordance with OPSD 205-040 *(Transition Treatment – Earth Fill to Granular Fill).*

The fill for the widened embankment should be placed and compacted in accordance with MTO's *SP105S10*, with inspection and field density testing by qualified personnel during placement operations to confirm that appropriate materials are used and that adequate levels of compaction are achieved.

In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where the embankment side slopes are equal to or greater than 8 m in height such that the uninterrupted slope height does not exceed 8 m. To reduce 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 must be in accordance with OPSS 572. If this protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting, is recommended to reduce the potential for remedial works being required on the side slopes in the spring prior to topsoil and seeding.

6.7.2 Approach Embankment Stability

Static Stability Analysis

Slope stability analyses have been performed for the proposed embankment widening using the commercially available program SLIDE, produced by Rocscience Inc., to check that a minimum factor of safety of 1.3 is achieved for the proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for approach embankments on this project, considering the design requirements and the available field and laboratory testing data.

The stability analyses were completed for an approximately 9 m high south approach embankment, and a maximum 10 m high north approach embankment, based on the subsurface conditions as encountered in the boreholes across the site. The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations:

Material Type	Undrained Analysis			Drained Analysis		
	ϕ (degrees)	c (kPa)	□ (kN/m^3)	ϕ' (degrees)	c' (kPa)	□ (kN/m^3)
New embankment fill (local earth fill)	31	0	19	31	0	19
Existing loose to dense fill	31-32	0	20	31-32	0	20
Loose to very dense sand and silt to silty sand	29-35	0	19-20	29-35	0	19-20
Soft to hard clayey silt to silty clay	0	25-200	19-20	28-30	3	19-20
Loose to very dense sand	29-35	0	19-20	29-35	0	19-20
Loose to very dense sand and gravel to gravelly sand	30-35	0	20	30-35	0	20
Dense silty sand till	32	0	21	32	0	21
Stiff to very stiff clayey silt till	0	75-125	21	30	3	21

The results of the stability analyses for both undrained and drained conditions at the south and north approaches are shown on Figures 1 to 4. These results indicate that a factor of safety of greater than 1.3 is achieved for an approximately 9 m high south approach embankment and maximum 10 m high north approach embankment with side slopes oriented no steeper than 2H:1V, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials.

Seismic Stability Analysis

Under earthquake conditions, the stability of slopes is assessed using conventional pseudo-static methods of slope stability analysis under the earthquake-induced peak ground acceleration. A calculated factor of safety of 1.0 is considered appropriate for global stability under seismic conditions. A seismic global stability analysis has been performed for the widened approach embankment side slopes for the replacement structure, using the parameters summarized in the preceding section.

The pseudo-static seismic slope stability analyses for a 2H:1V side slope configuration indicates that the slope will have a factor of safety of greater than 1.0 against deep-seated slope instability, using a peak ground acceleration of 0.06g. The results of the pseudo-static seismic stability analyses do indicate that some shallow sloughing could occur on the slopes during seismic events. This sloughing would not, however, impair the use of the highway, and would mainly be a maintenance issue. The potential for sloughing following seismic events could be reduced by providing well-vegetated slopes, as recommended in Section 6.6.1.

6.7.3 Approach Embankment Settlement

Settlement analyses below the new/widened approach embankments were carried out using a commercially available computer program, Settle-3D from Rocscience, using estimated elastic deformation moduli and consolidation parameters based on correlations with the SPT "N" values, Atterberg limits and field vane shear strengths (Bowles, 1984 and CHBDC, 2006), and engineering judgement from experience with similar soils in this region of Ontario.

The results of the analyses indicate a total settlement of up to about 30 mm to 40 mm would occur below the new/widened north approach embankment. The maximum settlement would occur under the centreline and west edge of the new embankment, reducing to essentially zero settlement where the new embankment ties into the existing embankment. This settlement will be elastic and immediate, occurring during and immediately following placement of the embankment fill.

At the south approach embankment, total settlement of between about 150 mm and 200 mm is predicted under the maximum 9 m high new/widened approach embankment. Although the majority of this settlement (associated with the loose to compact portions of the extensive cohesionless soil deposit) is expected to occur relatively rapidly during and immediately following the construction of the south approach embankment, it is expected that approximately 80 mm to 90 mm of this settlement will represent longer-term, post-construction consolidation settlement in the clayey silt to silty clay soil layers. As the firm to stiff cohesive soil layers are relatively thin, and because they are interlayered with permeable sand layers that will provide drainage to the consolidating clayey deposits, it is predicted that the majority of the consolidation settlement will be completed within approximately three to four months after placement of the fill materials.

To minimize the post-construction settlement at the south approach embankment, it is recommended that the south approach embankment be preloaded, by placing the engineered fill for the embankment widening up to the pavement subgrade and placing the pavement Granular B subbase, then allowing the foundation soils to settle for a period of at least four months prior to final paving and approach slab construction. It is understood that based on the current construction schedule and staging plan, the Crown Hill overpass abutments are to be constructed by mid-September 2013, followed by approach fill placement that will be completed by late October 2013. The approach fills will then be allowed to sit for approximately one year prior to final paving, which is sufficient time for settlement of the subsurface soils to be essentially complete.

Therefore, other means of accelerating settlement, such as surcharging or the use of light-weight fill materials, are not likely to be warranted. However, should the construction schedule be accelerated and a shorter preloading period be available, consideration could be given to the use of a 1 m to 2 m surcharge on top of the pavement subgrade level within 30 m of the abutments. Under a 2 m surcharge loading, the time to complete the majority of the consolidation settlement, and reduce the post-construction consolidation settlement to less than 10 mm to 25 mm within the limits of the approach embankments, is estimated to be on the order of ten to twelve weeks.

A settlement instrumentation plan is not warranted since the subsurface soils below the embankment footprint will be preloaded for approximately one year prior to commencing paving operations. However, an operational constraint should be included in the Contract Documents to ensure that placement of the approach embankment fill and Granular B sub-base is completed by the end of October 2013.

6.8 Design and Construction Considerations

The following subsections identify construction issues that should be considered at this stage as they may impact the detail design. Where applicable, Non-Standard Special Provisions (NSSP) will be developed for incorporation in the Contract Documents.

6.8.1 Open-Cut Excavations

The excavations for the north and south pier pile caps (and potentially the abutment pile caps as well) will extend through existing fill materials and into the predominantly cohesionless deposits. If 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 and Regulations for Construction Projects. The existing fill materials and the loose to compact cohesionless soils are classified as Type 3 soil and the dense cohesionless soils are classified as Type 2 soil (provided that all cohesionless materials are dewatered prior to construction) according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period, recommended as less than one month) should be made with side slopes no steeper than 1 horizontal to 1 vertical.

6.8.2 Excavation and Temporary Protection Systems

Given the proximity of the new Highway 400 NBL Crown Hill overpass structure to the existing Highway 400 NBL overpass embankments, excavations into portions of the existing west embankment side slope will be needed to permit the construction of the new overpass structure foundations. Temporary protection systems are expected to be required on and adjacent to Highway 11 to facilitate construction of the new pier pile caps, and protection systems may be required on the west side of the existing Highway 400 NBL to facilitate construction of the new abutment pile caps and associated wing walls. These temporary excavation support systems should be designed and constructed in accordance with OPSS 539 *(Construction Specification for Temporary Protection Systems)*. The lateral movement of temporary shoring systems on Highway 400 should meet Performance Level 2 as specified in OPSS 539.

It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the subsurface soil and groundwater conditions.

6.8.3 Groundwater Control

Based on the groundwater levels measured upon completion of the drilling and water levels in the piezometers on May 28, 2012, the groundwater levels are relatively high across the site. In particular, groundwater was near the original ground surface in Borehole No. 12-02 and artesian groundwater conditions were noted in Borehole 12-08 (associated with the deep granular deposit). Based on observations during drilling, it is anticipated that the “shallow” groundwater level associated with the near-surface cohesionless soil deposits varies from about Elevation 232 m near the south abutment and south pier, to about Elevation 236.5 m near the north abutment.

Provided that the pile caps for the north and south abutments are perched within the approach embankment fill, dewatering for these foundation elements should be relatively minor. However, the north and south pier pile caps are proposed to be founded at approximately Elevation 232.9 m and 231.6 m, respectively. The groundwater level is anticipated to be at about Elevation 236.5 m near the north abutment, declining to about Elevation 232 m at the south abutment and south pier. The pier excavations will extend into or near the groundwater level, and dewatering will be required during pile driving and pile cap construction to maintain the groundwater level below the pile cap founding level. Alternatively, an interlocking steel sheetpile system may be used at the piers, driven to a suitable distance below the pile cap level, to minimize the dewatering requirements.

A non-standard special provision (NSSP) has been developed to address dewatering requirements for the foundation elements. The dewatering system should be designed and constructed by a specialist dewatering firm. If dewatering at the site results in removal of greater than 50,000 L per day of groundwater, a Permit to Take Water (PTTW) from the Ministry of Environment will be required.

6.8.4 Artesian Groundwater Conditions and Granular Blankets at Pile Caps

Artesian groundwater pressures have been noted in the piezometer installed in Borehole 12-08 near the south abutment, associated with the deep granular deposit into which piles would be driven or caissons installed. Therefore, specialized construction techniques are recommended at the south abutment, south pier and north pier to mitigate the possible upward flow of water and fine soil particles along the pile or caisson shafts. It is recommended that a granular drainage blanket be placed beneath the pile caps to minimize the migration of fine soil particles that may be transported along the piles during and after construction. The granular drainage blanket should consist of a minimum 0.5 m thick layer of concrete fine aggregate, meeting the gradation requirements of OPSS 1002 *(Aggregates – Concrete)*.

6.8.5 Vibration Monitoring During Pile Installation

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities (such as pile driving) will reach this threshold level and, therefore, vibration monitoring for the existing structure is not expected to be required during construction at this site.

7.0 CLOSURE

This Foundation Design Report was prepared by Nick La Posta, P.Eng., and reviewed by Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder, with technical input from Murty Devata, P.Eng., a specialist foundations consultant with Golder. Fin Heffernan, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
22Oct12
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NLP/LCC/FJH/sm

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Ontario Provincial Standard Specifications (OPSS)

OPSS 206	Construction Specification for Grading
OPSS 212	Construction Specification for Borrow Material
OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 572	Construction Specification for Seed and Cover
OPSS 903	Construction Specification for Deep Foundations
OPSS 1002	Material Specification for Aggregates - Concrete
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 205.040	Transition Treatment – Earth Fill to Granular Fill
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation Frost 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 105S10	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
SP 206S03	Excavation and Grading; Excavation for Pavement Widening

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-Piles driven to found in the “100-blow” sand	• Feasible for support of new abutments and piers, and associated wing walls	• Abutment pile caps could be maintained within embankment fill to minimize dewatering requirements • Allows for integral abutment construction • Would minimize differential settlement between foundation elements	• Potential for encountering obstructions (cobbles and/or boulders) during pile driving; this could result in piles “hanging up” and lower geotechnical resistances • Potential for noise and/or vibration impacts on nearby buildings	• Conventional construction methods for H-pile foundations	• Lower relative cost compared with caisson option • Estimated unit cost is approximately \$250/linear metre for pile installation and \$600/m^3 for pile cap construction
Steel pipe (tube) piles, driven to found in the “100-blow” sand	• Feasible for support of new abutments and piers, and associated wing walls	• Abutment pile caps could be maintained within embankment fill to minimize dewatering requirements • Allows for semi-integral abutment construction • Would minimize differential settlement between foundation elements	• Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in piles “hanging up” and lower geotechnical resistances • Potential for noise and/or vibration impacts on nearby buildings	• Conventional construction methods	• Costs for steel pipe (tube) piles slightly higher than for steel H-piles
Caissons founded in the hard (100-blow) sand	• Feasible but not recommended for support of abutments and piers, and associated wing walls	• Abutment pile caps could be maintained within embankment fill to minimize dewatering requirements • Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles	• Potential for loss of ground in the water-bearing cohesionless deposits • Temporary or permanent liners would be required; likely not possible to inspect caisson base • Precludes use of integral abutments	• Conventional construction methods	• Higher cost compared with shallow foundations or steel H-piles

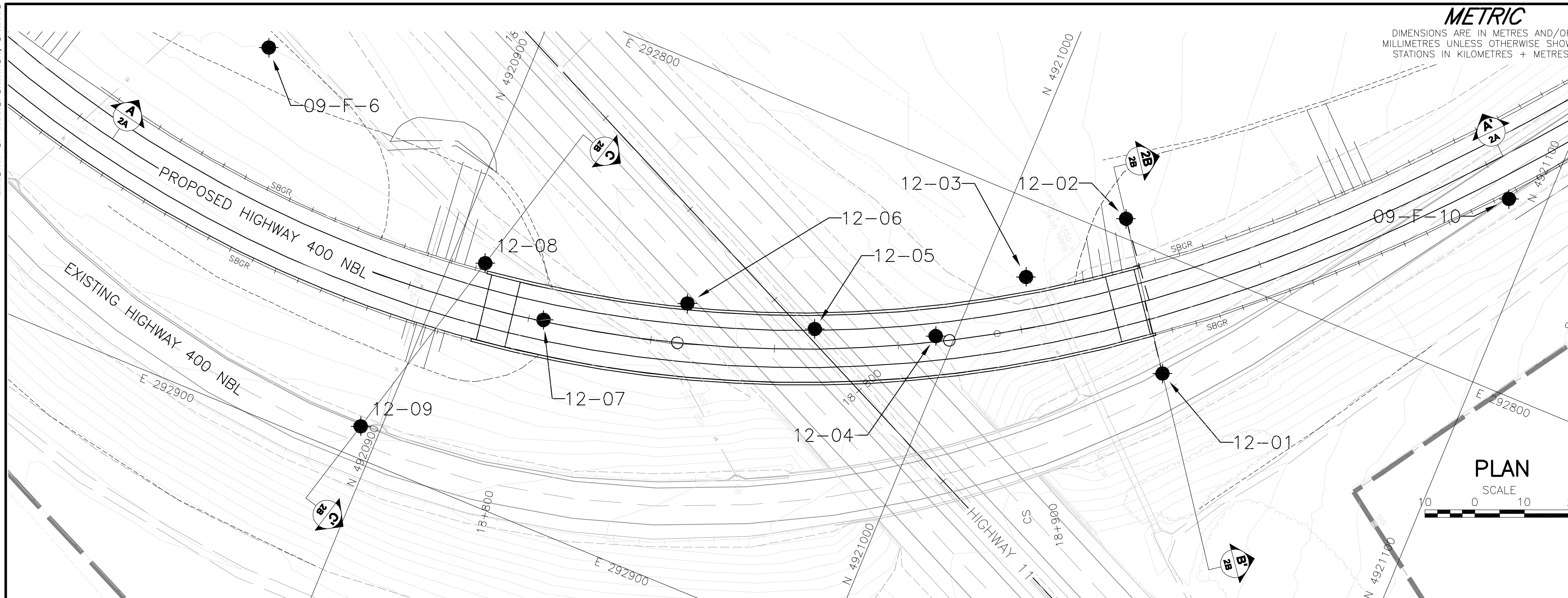
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

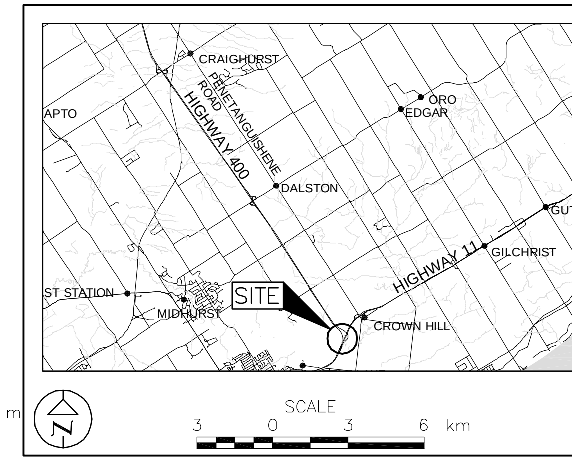
CONT No. 2012–2034
GWP No. 2179–10–00

HIGHWAY 400 NBL CROWN HILL OVERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



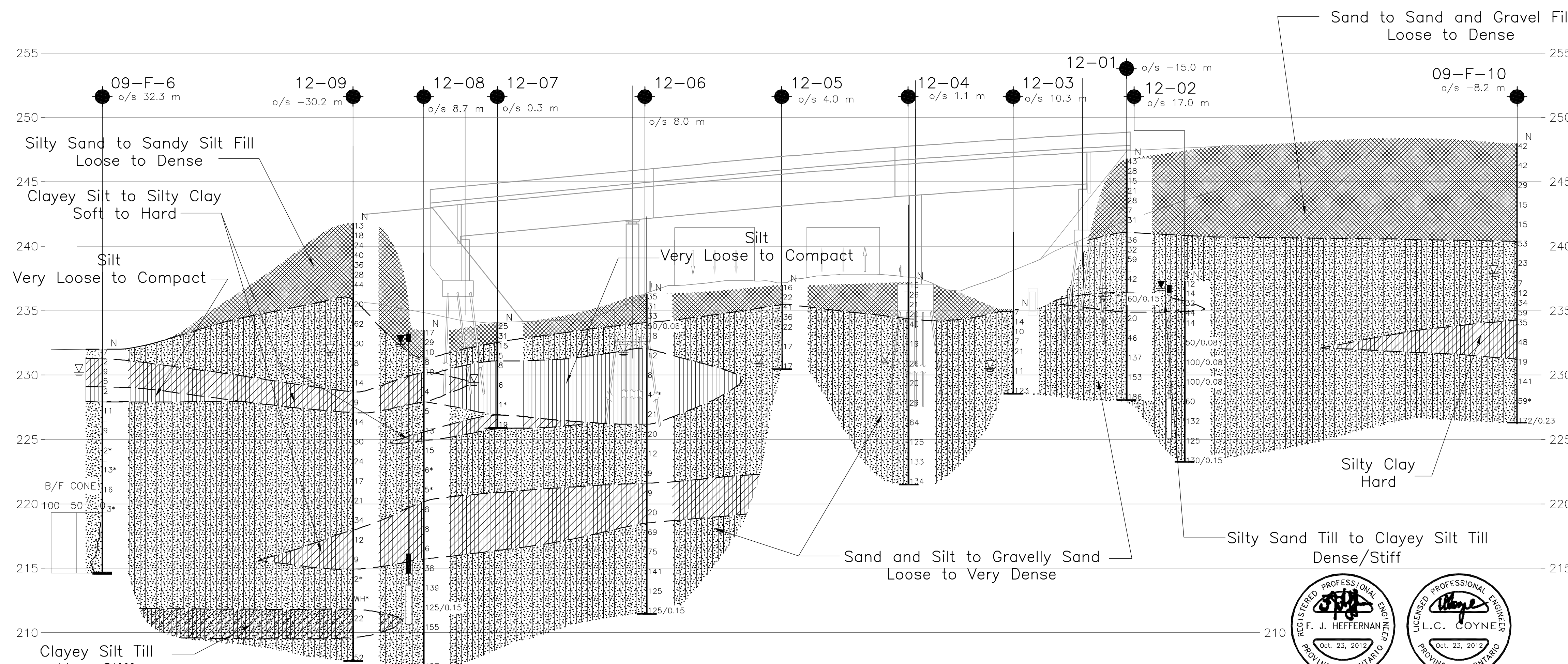


LEGEND

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

BOREHOLE CO–ORDINATES

No.	ELEVATION	NORTHING	EASTING
09–F–6	232.0	4920849.8	292830.3
09–F–10	248.0	4921092.7	292763.1
12–01	246.8	4921041.5	292822.3
12–02	237.3	4921022.8	292796.3
12–03	235.1	4921008.7	292814.8
12–04	237.2	4920996.4	292832.8
12–05	237.0	4920973.3	292840.8
12–06	236.3	4920947.6	292845.8
12–07	234.0	4920922.0	292859.9
12–08	233.5	4920906.8	292853.8
12–09	241.8	4920896.1	292893.8



A–A' 2A CENTERLINE PROFILE
HORIZONTAL SCALE
10 0 10 20 m
4 0 4 8 m
VERTICAL SCALE

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.

REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN Oct. 23, 2012 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Oct. 23, 2012 PROVINCE OF ONTARIO

REFERENCE

Base plans provided in digital format by MH, drawing files x84117Align.dwg, x84117Base.dwg and x84117design.dwg received May 24, 2012 and X094197Contours.dwg, received July 18, 2011.

NO.	DATE	BY	REVISION

Geocres No. 31D–552			
HWY. 400		PROJECT NO. 09–1111–0022	DIST.
SUBM'D. NLP	CHKD. NLP	DATE: 10/23/2012	SITE:
DRAWN: JFC	CHKD. NLP	APPD. LCC	DWG. 1

PLOT DATE: October 23, 2012
FILENAME: T:\Projects\2009\09–1111–0022 MH, Vespra)\–Hd– (Crown Hill Structure) (with stamps)\091111002HC001.dwg

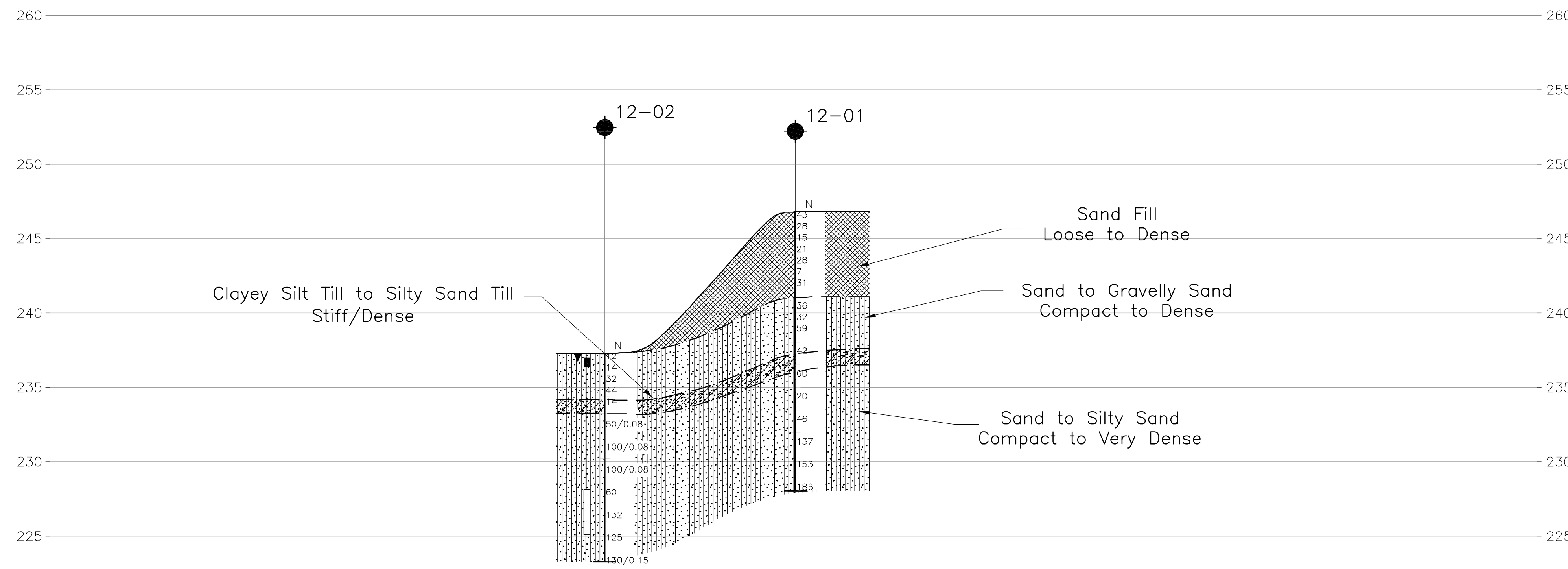
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2012–2034
GWP No. 2179–10–00

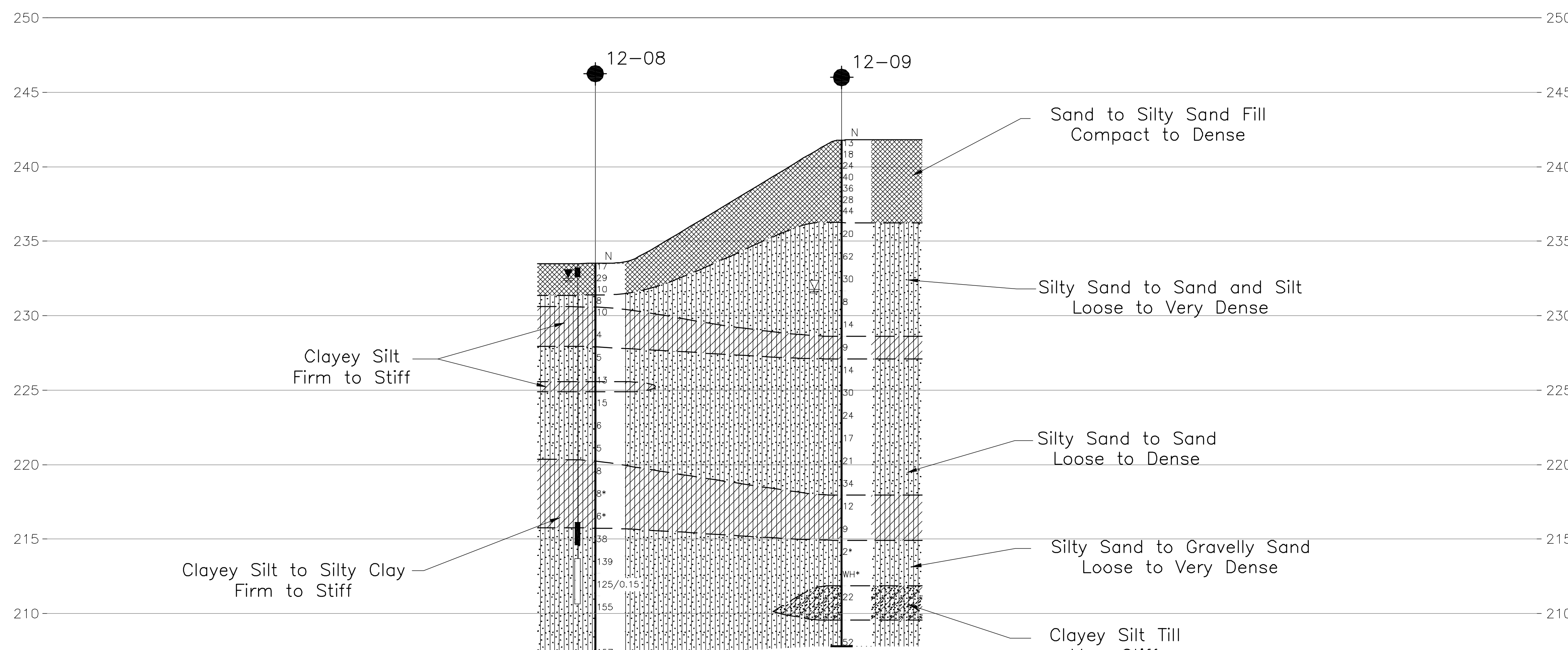
HIGHWAY 400 NBL CROWN HILL OVERPASS
SOIL STRATA

SHEET

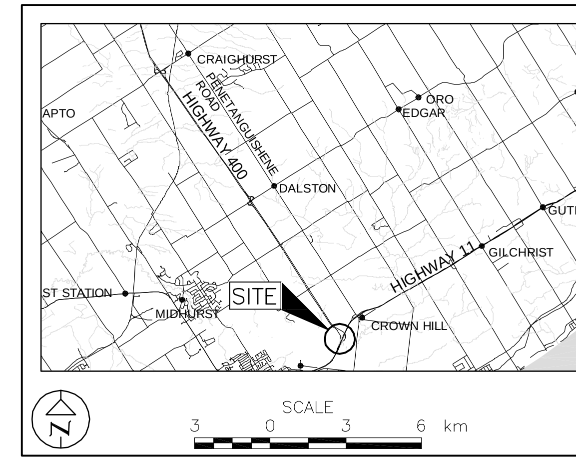
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



B-B' 2A NORTH ABUTMENT
HORIZONTAL SCALE 10 0 10 20 m
VERTICAL SCALE 4 0 4 8 m



C-C' 2A SOUTH ABUTMENT
HORIZONTAL SCALE 10 0 10 20 m
VERTICAL SCALE 4 0 4 8 m



LEGEND

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

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
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12–02	237.3	4921022.8	292796.3
12–08	233.5	4920906.8	292853.8
12–09	241.8	4920896.1	292893.8

NOTES

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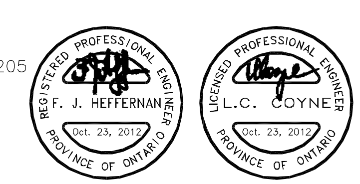


NO.	DATE	BY	REVISION

Geocres No. 31D–552

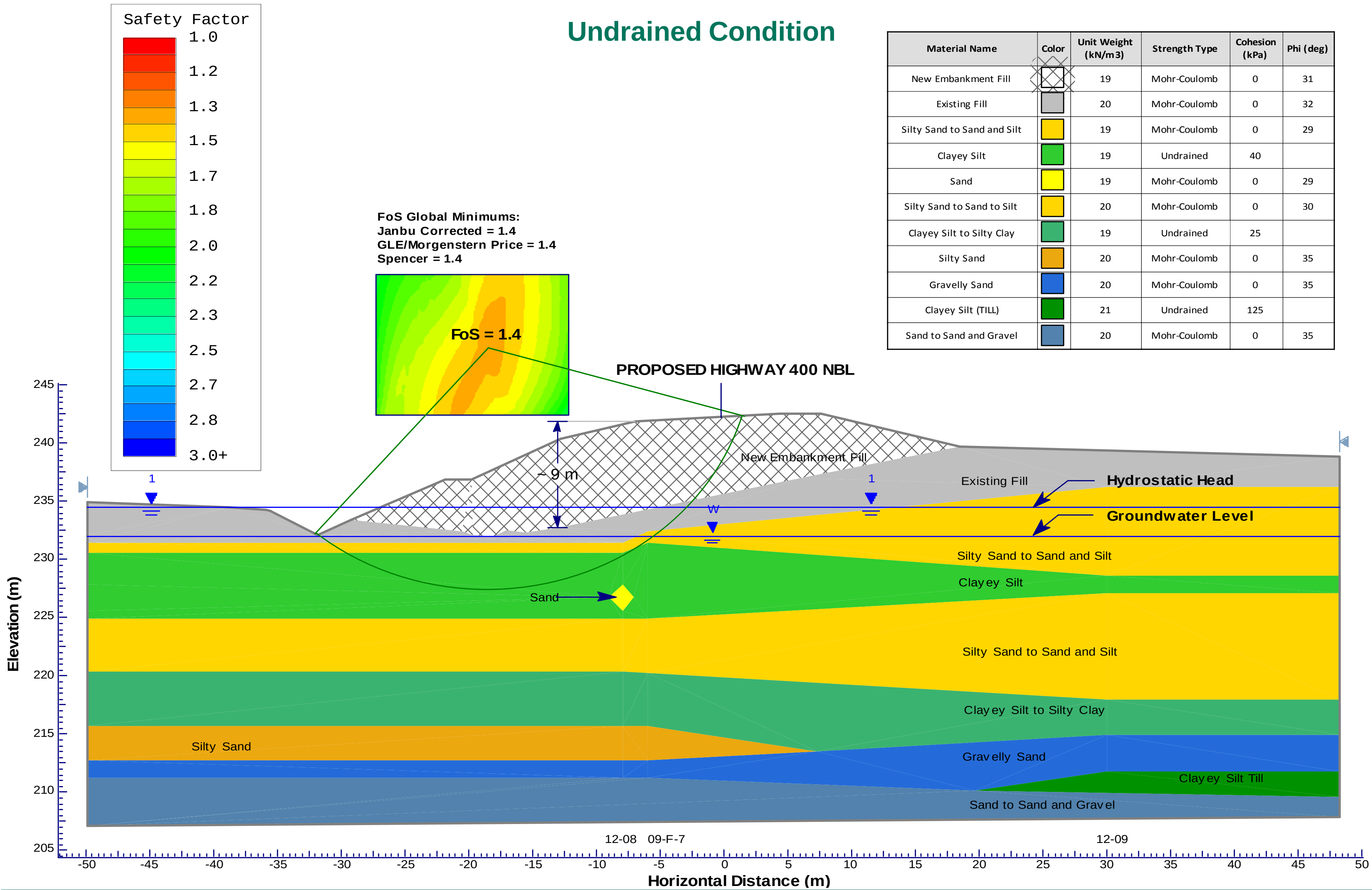
HWY. 400		PROJECT NO. 09–1111–0022	DIST.
SUBM'D. NLP	CHKD. NLP	DATE: 10/23/2012	SITE:
DRAWN: JFC	CHKD. NLP	APPD. LCC	DWG. 2

Crown Hill Overpass – Hwy 400 NBL, Station 18+775
South Approach Embankment Widening – Static Global Stability

Figure 1

Undrained Condition

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Embankment Fill		19	Mohr-Coulomb	0	31
Existing Fill		20	Mohr-Coulomb	0	32
Silty Sand to Sand and Silt		19	Mohr-Coulomb	0	29
Clayey Silt		19	Undrained	40	
Sand		19	Mohr-Coulomb	0	29
Silty Sand to Sand to Silt		20	Mohr-Coulomb	0	30
Clayey Silt to Silty Clay		19	Undrained	25	
Silty Sand		20	Mohr-Coulomb	0	35
Gravelly Sand		20	Mohr-Coulomb	0	35
Clayey Silt (TILL)		21	Undrained	125	
Sand to Sand and Gravel		20	Mohr-Coulomb	0	35



Date: July 2012

Project No: 09-1111-0022-7

Analysis By: TVA **Reviewed By:** RA

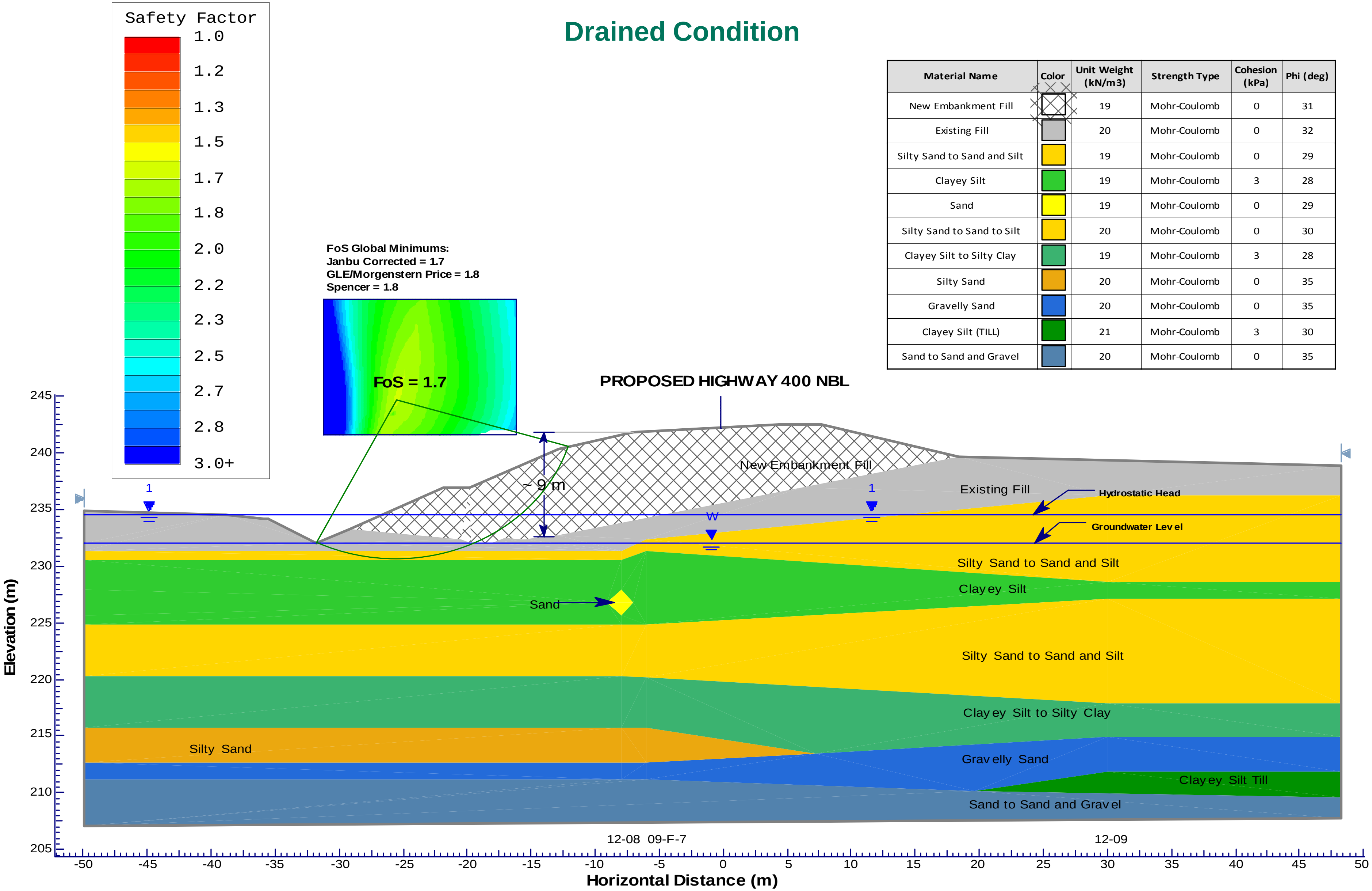
Golder Associates

Crown Hill Overpass – Hwy 400 NBL, Station 18+775
South Approach Embankment Widening – Static Global Stability

Figure 2

Drained Condition

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Embankment Fill		19	Mohr-Coulomb	0	31
Existing Fill		20	Mohr-Coulomb	0	32
Silty Sand to Sand and Silt		19	Mohr-Coulomb	0	29
Clayey Silt		19	Mohr-Coulomb	3	28
Sand		19	Mohr-Coulomb	0	29
Silty Sand to Sand to Silt		20	Mohr-Coulomb	0	30
Clayey Silt to Silty Clay		19	Mohr-Coulomb	3	28
Silty Sand		20	Mohr-Coulomb	0	35
Gravelly Sand		20	Mohr-Coulomb	0	35
Clayey Silt (TILL)		21	Mohr-Coulomb	3	30
Sand to Sand and Gravel		20	Mohr-Coulomb	0	35

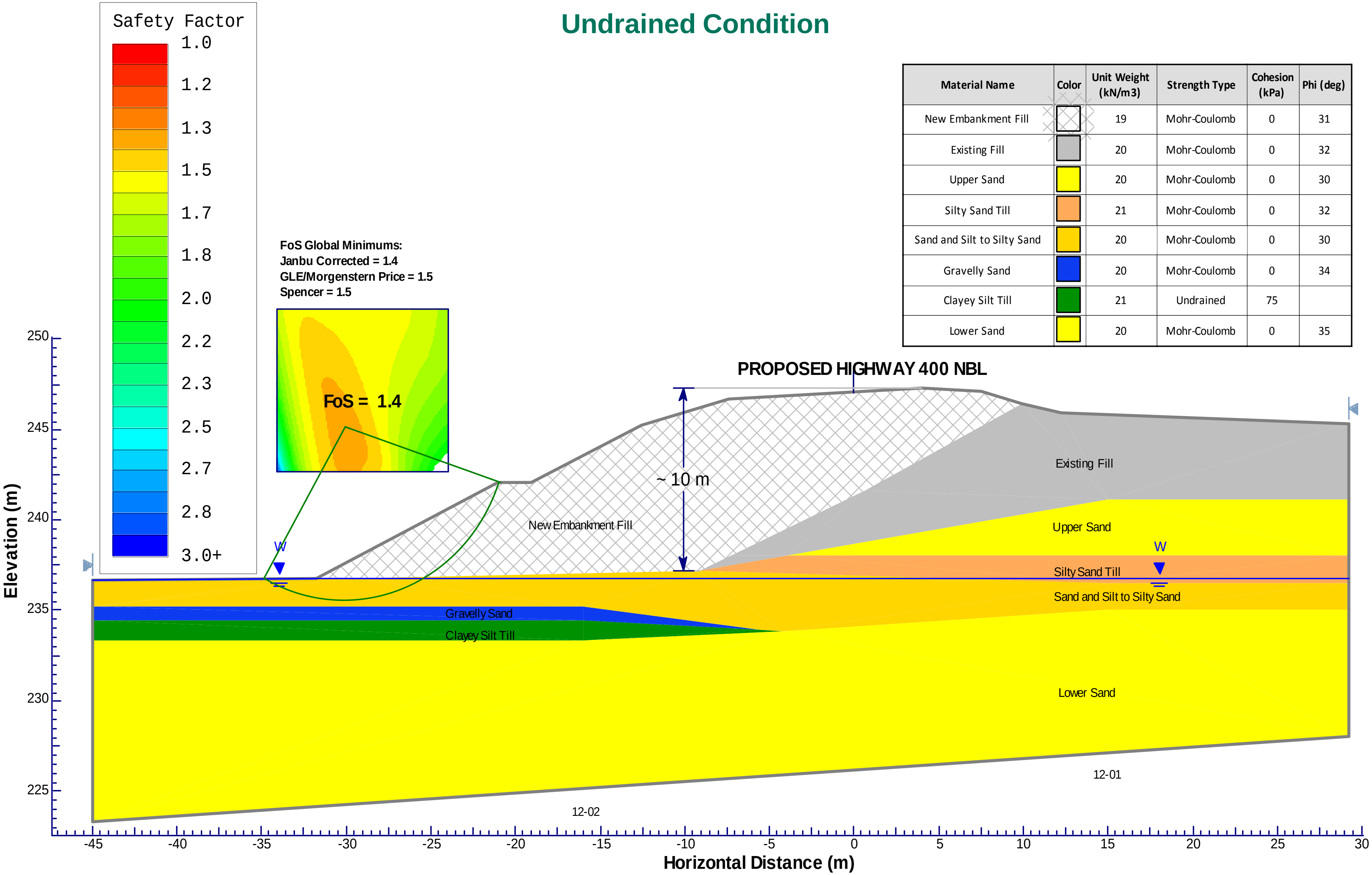


Date: July 2012
Project No: 09-1111-0022-7

Analysis By: TVA **Reviewed By:** RA

Golder Associates

Undrained Condition



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Embankment Fill		19	Mohr-Coulomb	0	31
Existing Fill		20	Mohr-Coulomb	0	32
Upper Sand		20	Mohr-Coulomb	0	30
Silty Sand Till		21	Mohr-Coulomb	0	32
Sand and Silt to Silty Sand		20	Mohr-Coulomb	0	30
Gravelly Sand		20	Mohr-Coulomb	0	34
Clayey Silt Till		21	Undrained	75	
Lower Sand		20	Mohr-Coulomb	0	35

Date: July 2012

Project No: 09-1111-0022-7

Analysis By: TVA **Reviewed By:** RA

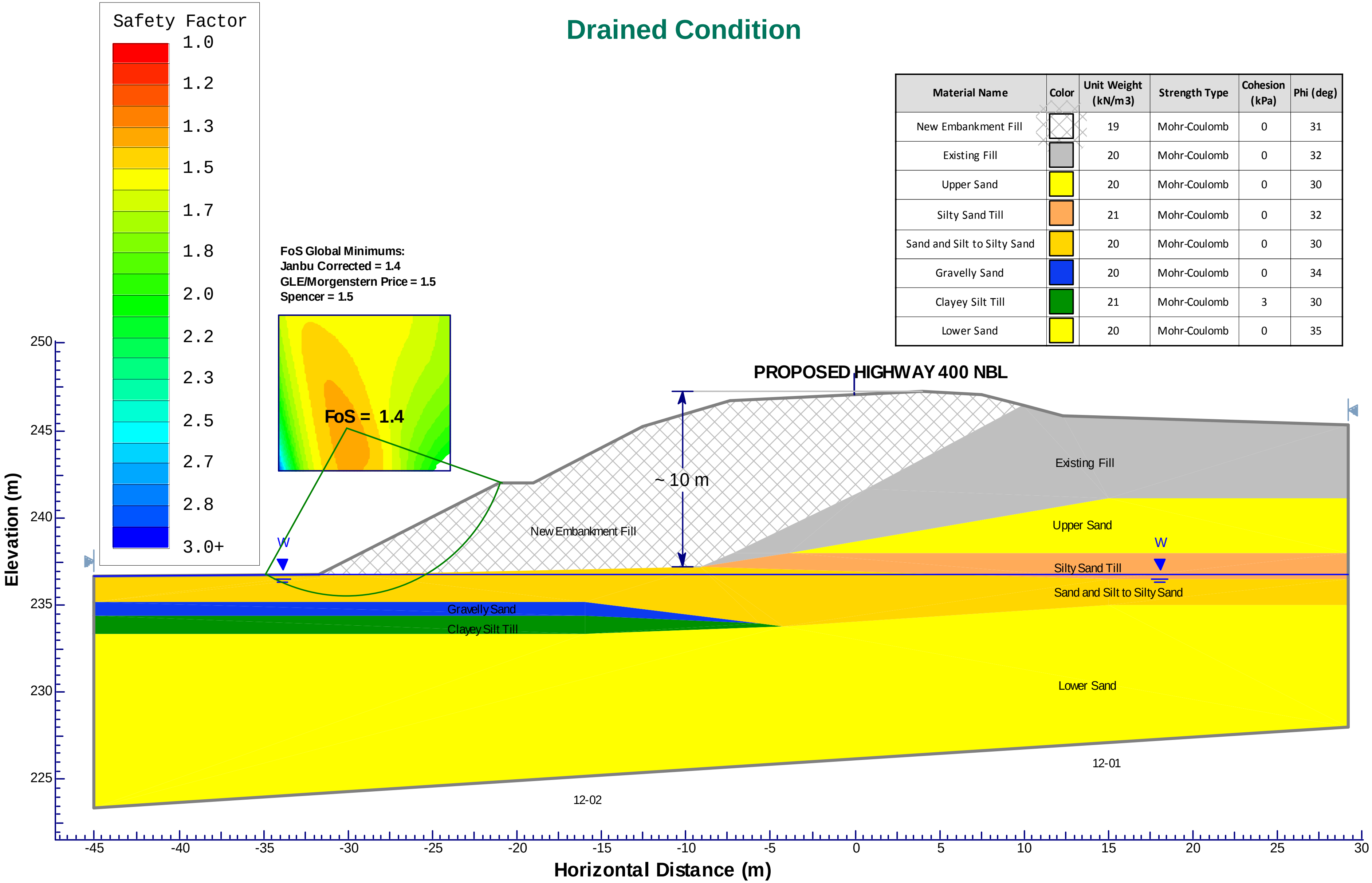
Golder Associates

Crown Hill Overpass – Hwy 400 NBL, Station 18+925
North Approach Embankment Widening – Static Global Stability

Figure 4

Drained Condition

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
New Embankment Fill		19	Mohr-Coulomb	0	31
Existing Fill		20	Mohr-Coulomb	0	32
Upper Sand		20	Mohr-Coulomb	0	30
Silty Sand Till		21	Mohr-Coulomb	0	32
Sand and Silt to Silty Sand		20	Mohr-Coulomb	0	30
Gravelly Sand		20	Mohr-Coulomb	0	34
Clayey Silt Till		21	Mohr-Coulomb	3	30
Lower Sand		20	Mohr-Coulomb	0	35



Date: July 2012

Project No: 09-1111-0022-7

Analysis By: TVA **Reviewed By:** RA

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APPENDIX A

Borehole Records

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, 60º cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

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

Piezo-Cone Penetration Test (CPT)
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 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 Relative Density	N 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	c_u, s_u 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[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	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

Golder Associates

LIST OF SYMBOLS

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

I. GENERAL

π	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

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	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
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
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_α	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
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

RECORD OF BOREHOLE No 09-F-6

SHEET 1 OF 2 **METRIC**

PROJECT 09-1111-0022

G.W.P. 2179-10-00 LOCATION N 4920849.8 ;E 292830.3 ORIGINATED BY AB

DIST Central HWY 400 BOREHOLE TYPE CME 55 Track-Mount, 108 mm Diameter Hollow Stem Auger COMPILED BY MS/NK

DATUM Geodetic DATE August 5, 2010 CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
232.0	GROUND SURFACE								
0.0	TOPSOIL								
0.1	SAND, some silt Loose Brown Moist	1	SS	7					
231.3 0.7	CLAYEY SILT, trace to some sand, containing silty sand seams and layers Firm to stiff Brown Moist to wet	2	SS	2		231			0 22 63 15
		3	SS	9	∇	230	Field vane 3.2		
		4	SS	5					
229.1 2.9	SILT, some sand Very loose Grey Wet	5	SS	2		229			
228.0 4.0	Sandy SILT, trace to some clay, containing silt seams Compact Brown Wet	6	SS	11		228 227			
226.4 5.6	SAND, trace silt, clay and gravel, containing clayey silt layers Very loose to loose Brown Wet	7	SS	9		226 225			
		8	SS	2*		224			6 79 8 7
223.3 8.7	SAND and GRAVEL, trace silt, containing clayey silt layers Compact Grey Wet	9	SS	13*		223			
221.9 10.1	SAND, some silt, trace clay, containing clayey silt layers Compact Brown Wet	10	SS	16		222 221			0 82 14 4
		11	SS	3*		220			
219.3 12.7	END OF BOREHOLE Dynamic Cone Penetration Test					219 218			

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 09-F-6

SHEET 2 OF 2 **METRIC**

PROJECT 09-1111-0022

G.W.P. 2179-10-00 LOCATION N 4920849.8 ;E 292830.3 ORIGINATED BY AB

DIST Central HWY 400 BOREHOLE TYPE CME 55 Track-Mount, 108 mm Diameter Hollow Stem Auger COMPILED BY MS/NK

DATUM Geodetic DATE August 5, 2010 CHECKED BY LCC

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE Dynamic Cone Penetration Test						216 215				
214.6 17.4	END OF DCPT Notes: *SPT "N" value considered to be affected by sample disturbance due to groundwater inflow to borehole. 1. Water level in open borehole at a depth of 1.8 m (Elevation 230.2 m) on completion of drilling.										

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 09-F-10

METRIC

PROJECT: 09-1111-0022			
G.W.P. 2179-10-00	LOCATION: N 4921092.7 ;E 292763.1		ORIGINATED BY: AB
DIST: Central	HWY: 400	BOREHOLE TYPE: CME 75 Truck-Mounted, 200 mm Diameter Hollow Stem Augers	COMPILED BY: NK
DATUM: Geodetic		DATE: August 10, 2010	CHECKED BY: LCC

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION	STRAT PLOT
248.0	GROUND SURFACE	
0.0	Sand and gravel, trace to some silt (FILL) Dense Brown Moist	Fill
247.4 / 0.6	Sand, some silt, some gravel (FILL) Dense to compact Brown Moist	Fill
241.9 / 6.1	Sand and gravel, some silt (FILL) Compact Brown Moist	Fill
240.4 / 7.6	SAND, some silt, trace to some gravel, containing clayey silt lenses Very dense to compact Brown Moist to wet	Sand
237.1 / 11.0	Gravelly SAND, trace to some silt, trace clay, containing clayey silt lenses Loose to very dense Brown Moist to wet	Gravelly sand
234.3 / 13.7	SILTY CLAY, trace sand and gravel, containing cobbles Hard Grey Moist	Silty clay

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	42
2	SS	42
3	SS	29
4	SS	15
5	SS	15
6	SS	53
7	SS	23
8	SS	7
9	SS	12
10	SS	34
11	SS	59
12	SS	35

GROUND WATER CONDITIONS: water level symbol at approximately elevation 238.

ELEVATION SCALE: 247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235, 234

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

Sample 12: plastic limit–liquid limit range plotted, liquid limit value 45.

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Sample	GR	SA	SI	CL
6	11	66	19	4
9	24	47	20	9

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-F-10

SHEET 2 OF 2 **METRIC**

PROJECT 09-1111-0022
G.W.P. 2179-10-00
LOCATION N 4921092.7 ;E 292763.1
ORIGINATED BY AB
DIST Central HWY 400
BOREHOLE TYPE CME 75 Truck-Mounted, 200 mm Diameter Hollow Stem Augers
COMPILED BY NK
DATUM Geodetic
DATE August 10, 2010
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY, trace sand and gravel, containing cobbles Hard Grey Moist	13	SS	48	232	
231.2 16.8	SAND, trace to some silt, trace gravel, trace clay Compact to very dense Brown Wet	14	SS	19	231	0 78 20 2
		15	SS	141	230	
		16	SS	59*	229, 228	
226.3		17	SS	172/0.23	227	3 89 6 2
21.7	END OF BOREHOLE					

Notes:

*SPT "N" values considered to be affected by sample disturbance due to groundwater inflow to borehole.

1. Water level in open borehole at a depth of 10.1 m (Elevation 237.9 m) on completion of drilling.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 12-01

SHEET 1 OF 2 **METRIC**

PROJECT 09-1111-0022
G.W.P. 2179-10-00 LOCATION N 4921041.5 ;E 292822.3 ORIGINATED BY DD
DIST Central HWY 400 BOREHOLE TYPE Truck Mount Power Auger COMPILED BY NLP
DATUM Geodetic DATE March 28, 2012 CHECKED BY LCC

ELEV DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
246.8	GROUND SURFACE					
0.0	Sand, trace to some silt, trace clay, trace gravel (FILL) Loose to dense Brown Moist	1	SS	43		
		2	SS	28		
		3	SS	15		
		4	SS	21		
		5	SS	28		
		6	SS	7		2 85 7 6
		7	SS	31		
241.1 / 5.7	SAND, trace to some gravel, some silt, containing interlayers of silty sand at a depth of 7.6 m Dense to very dense Brown Moist	8A	SS	36		
		8B	SS	32		
		9	SS	59		8 70 15 7
238.0 / 8.8	Silty SAND, some gravel, trace to some clay (TILL) Dense Brown Moist	10	SS	42		
236.5 / 10.3	Silty SAND, some gravel, containing organics and wood fragments Very dense Grey Moist to wet Auger grinding noted from 10.7 m to 12.2 m	11	SS	60/0.15	▽	
235.0 / 11.8	SAND, trace to some silt, some gravel, trace clay, containing seams or lenses of clayey silt in Sample 13 Compact to dense Grey Wet Auger grinding noted from 12.2 m to 15.2 m	12	SS	20		15 65 15 5
		13	SS	46		
231.9						

Elevation scale: 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235, 234, 233, 232

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 12-01

SHEET 2 OF 2 **METRIC**

PROJECT 09-1111-0022
G.W.P. 2179-10-00
LOCATION N 4921041.5 ;E 292822.3
ORIGINATED BY DD
DIST Central HWY 400
BOREHOLE TYPE Truck Mount Power Auger
COMPILED BY NLP
DATUM Geodetic
DATE March 28, 2012
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
14.9	SAND, trace to some silt, trace clay Very dense Brown Wet	14	SS	137			~18 (o)		0	91	7	2
						231						
						230						
		15	SS	153								
						229						
228.0		16	SS	186			~16 (o)					
18.8	END OF BOREHOLE					228						

END OF BOREHOLE

NOTE:

1. Water level in open borehole at a depth of 10.7 m (Elev. 236.1 m) upon completion of drilling.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity   ○$^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 12-02

METRIC

PROJECT: 09-1111-0022			
G.W.P.: 2179-10-00	LOCATION: N 4921022.8 ;E 292796.3	ORIGINATED BY: DD
DIST: Central	HWY: 400	BOREHOLE TYPE: Track Mount Power Auger	COMPILED BY: NLP
DATUM: Geodetic	DATE: April 17, 2012	CHECKED BY: LCC	

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
237.3	GROUND SURFACE
0.0	TOPSOIL
0.1	SAND and SILT , some gravel, trace clay, containing clayey silt seams Compact to dense Grey Moist
235.2 2.1	Gravelly SAND, trace silt Dense Brown Moist
234.4 2.9	CLAYEY SILT with sand, trace to some gravel (TILL) Stiff Brown Moist
233.3 4.0	SAND, some gravel, trace to some silt, trace clay Very dense Brown Moist to wet Spoon bouncing on possible boulder at 4.7 m and 6.2 m, auger grinding noted from 4.6 m to 7.6 m
230.2 7.1	SAND, trace to some silt Very dense Brown Moist Spoon bouncing at 7.7 m
223.3 14.0	

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	12
2	SS	14
3	SS	32
4	SS	44
5	SS	14
6	SS	50/0.08
7	SS	100/0.08
8	SS	100/0.08
9	SS	60
10	SS	132
11	SS	125
12	SS	130/0.15

ELEVATION SCALE: 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Sample	GR	SA	SI	CL
3	15	47	31	7
5	10	48	24	18
7	13	72	9	6

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 09-1111-0022.GPJ GAL-MISS.GDT 7/10/12 DD/SAC

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RECORD OF BOREHOLE No 12-02

METRIC

PROJECT	09-1111-0022		
G.W.P.	2179-10-00	LOCATION	N 4921022.8 ;E 292796.3
DIST	Central	HWY	400
BOREHOLE TYPE	Track Mount Power Auger		
DATUM	Geodetic	DATE	April 17, 2012
ORIGINATED BY	DD	COMPILED BY	NLP
CHECKED BY	LCC		

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) / SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10, 20, 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE NOTES: 1. Water level in open borehole at a depth of 4.6 m (Elev. 232.7 m) upon completion of drilling 2. Water level in piezometer at a depth of 0.6 m (Elev. 236.7 m) on May 28, 2012										

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

Foundation Design

Golder Associates

RECORD OF BOREHOLE No 12-03

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0022

G.W.P. 2179-10-00 LOCATION N 4921008.7 ;E 292814.8 ORIGINATED BY DD

DIST Central HWY 400 BOREHOLE TYPE Track Mount Power Auger COMPILED BY NLP

DATUM Geodetic DATE April 17, 2012 CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
235.1	GROUND SURFACE										
0.0 / 0.1	TOPSOIL										
	SAND, trace to some gravel, trace to some silt, trace clay, containing organics below 1.5 m Loose to compact Brown to grey Moist	1	SS	7							
		2	SS	14							
		3	SS	10		w ≈ 19	O.C. = 3.6%	10	75	10	5
233.0 / 2.1	Silty SAND TO SAND, trace to some silt, trace gravel, containing trace organics at 4.6 m Loose to very dense Grey to brown Moist	4	SS	7							
		5	SS	21		w ≈ 10					
					▽ (Elev. 230.5 m)						
		6	SS	11		w ≈ 22	O.C. = 0.9%				
		7	SS	123		w ≈ 17					
228.5 / 6.6	END OF BOREHOLE										

NOTE:

1. Water level in open borehole at a depth of 4.6 m (Elev. 230.5 m) upon completion of drilling

Elevation scale: 235, 234, 233, 232, 231, 230, 229

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 09-1111-0022.GPJ GAL-MISS.GDT 7/10/12 DD/SAC

Golder Associates

RECORD OF BOREHOLE No 12-04

 METRIC

PROJECT 09-1111-0022

G.W.P. 2179-10-00 LOCATION N 4920996.4 ;E 292832.8 ORIGINATED BY DD

DIST Central HWY 400 BOREHOLE TYPE Track Mount Power Auger COMPILED BY NLP

DATUM Geodetic DATE April 11, 2012 CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	GR	SA	SI	CL
237.2	GROUND SURFACE									
0.0	Sand, some gravel, trace silt (FILL) Compact Brown Moist	1	SS	15						
236.5 / 0.7	Sand, some silt to sandy silt, trace to some gravel, trace to some clay (FILL) Compact Grey Moist	2	SS	26		o				
		3	SS	21						
		4	SS	20		o	11	59	18	12
234.3 / 2.9	Sandy SILT to silty SAND, trace to some gravel, trace to some clay, containing organics (wood fragments) at 4.6 m Dense to compact Brown to grey Moist to wet	5	SS	40						
		6	SS	19		o	10	60	20	10
		7	SS	26	▽ (approx. El. 231.2)					
230.1 / 7.1	SAND, trace to some silt, trace clay, containing some gravel in Sample 8 Compact to very dense Brown to grey Moist	8	SS	20		o				
		9	SS	29						
		10	SS	64		o	0	89	8	3
		11	SS	125						
		12	SS	133		o				

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10, 20, 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%). ELEVATION SCALE: 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224, 223. STRAT PLOT.

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 12-04

SHEET 2 OF 2 **METRIC**

PROJECT 09-1111-0022

G.W.P. 2179-10-00 LOCATION N 4920996.4 ;E 292832.8 ORIGINATED BY DD

DIST Central HWY 400 BOREHOLE TYPE Track Mount Power Auger COMPILED BY NLP

DATUM Geodetic DATE April 11, 2012 CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
221.5			13	SS	134		222				
15.7	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 6.1 m (Elev. 231.1 m) upon completion of drilling										

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 12-05

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0022
G.W.P. 2179-10-00
DIST Central HWY 400
DATUM Geodetic
LOCATION N 4920973.3 ;E 292840.8
BOREHOLE TYPE Track Mount Power Auger
DATE April 11, 2012
ORIGINATED BY DD
COMPILED BY NLP
CHECKED BY LCC

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
237.0	GROUND SURFACE						
0.0	Sand, trace gravel, trace silt (FILL) Compact Brown Moist	1	SS	16			
		2	SS	22			
235.6 / 1.4	Silty SAND, trace to some gravel, trace to some clay, containing clayey silt seams Dense Brown Moist	3	SS	41		~8	5 61 24 10
		4	SS	36			
234.1 / 2.9	Sandy SILT to silty SAND, trace to some gravel, trace clay, containing organics (wood fragments) in Sample 6 at 4.6 m Compact Brown to dark grey Moist to wet	5	SS	22		~10	
		6	SS	17			
					▽ (Elev. 230.9)		
		7	SS	17		~17	
230.4 / 6.6	END OF BOREHOLE						

NOTE:

1. Water level in open borehole at a depth of 6.1 m (Elev. 230.9 m) upon completion of drilling

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³
ELEVATION SCALE: 236, 235, 234, 233, 232, 231

+ ³, × ³: Numbers refer to Sensitivity ○ ³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 12-06

SHEET 1 OF 2 **METRIC**

PROJECT	09-1111-0022		
G.W.P.	2179-10-00	LOCATION	N 4920947.6 ;E 292845.8
DIST	Central HWY 400	BOREHOLE TYPE	Track Mount Power Auger
DATUM	Geodetic	DATE	April 12, 2012
ORIGINATED BY	DD	COMPILED BY	NLP
CHECKED BY	LCC		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
236.3	GROUND SURFACE
0.0	Sand, some gravel, trace silt and clay (FILL) Dense Brown Moist
235.6 / 0.7	Silty sand, trace to some gravel, trace to some silt (FILL) Dense Brown Moist
234.2 / 2.1	Silty SAND, trace gravel, trace clay Compact to very dense Brown Moist
232.1 / 4.2	SILT, some sand, trace gravel, trace to some clay, containing clayey silt to silty clay lenses or seams Loose to compact Brown to grey Moist to wet
226.2 / 10.1	SAND, trace silt to silty SAND, trace clay, containing clayey silt to silty clay lenses or seams Compact Brown Wet
221.6 / 14.7	

SAMPLES

NUMBER	TYPE	"N" VALUES	Elevation (approx.)	REMARKS GR	SA	SI	CL
1	SS	35	236				
2	SS	31					
3	SS	33		5	61	24	10
4	SS	50/0.08					
5	SS	18	233				
6	SS	12					
7	SS	8	230	0	12	77	11
8	SS	4*					
9	SS	21					
10	SS	20					
11	SS	12					
12	SS	9					

GROUND WATER CONDITIONS: ∇ at approx. elevation 231.8

ELEVATION SCALE: 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224, 223, 222

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 12-06

SHEET 2 OF 2 **METRIC**

PROJECT 09-1111-0022
G.W.P. 2179-10-00 LOCATION N 4920947.6 ;E 292845.8 ORIGINATED BY DD
DIST Central HWY 400 BOREHOLE TYPE Track Mount Power Auger COMPILED BY NLP
DATUM Geodetic DATE April 12, 2012 CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	CLAYEY SILT with sand, trace to some gravel Loose to compact Grey Moist to wet	13	SS	9	221	o	8	47	33	12
					220					
		14	SS	20	219	o	7	45	30	18
218.5 / 17.8	Silty SAND to sandy SILT, trace gravel, trace clay Very dense Grey Moist to wet				218					
		15	SS	69	217	o				
		16	SS	75	216					
		17	SS	141	215	o	1	29	67	3
		18	SS	125	214, 213					
		19	SS	125/0.15	212	o				
211.5 / 24.8	END OF BOREHOLE									

NOTE:

1. Water level in open borehole at a depth of 4.6 m (Elev. 231.7 m) upon completion of drilling

* SPT "N" values considered to have been affected by sample disturbance due to groundwater inflow to borehole

Chart legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 12-07

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0022
G.W.P. 2179-10-00
LOCATION N 4920922.0 ;E 292859.9
ORIGINATED BY DD
DIST Central HWY 400
BOREHOLE TYPE Track Mounted Power Auger
COMPILED BY NLP
DATUM Geodetic
DATE April 18, 2012
CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
234.0	GROUND SURFACE						
0.0	TOPSOIL						
0.2	Sand, some silt, trace gravel, trace clay (FILL) Compact to dense Brown Moist	1	SS	25			
		2	SS	31			
232.6 / 1.4	Silty SAND, trace clay, trace gravel Loose to compact Brown Moist	3	SS	15		○	2 73 21 4
		4	SS	5			
231.1 / 2.9	SILT, trace sand to SAND and SILT, trace clay, trace gravel, containing clayey silt to silty clay lenses or seams Loose Grey Moist to wet	5	SS	8		○	
		6	SS	6	▽ (Elev. 229.4)		
		7	SS	1*		○	
226.9 / 7.1	CLAYEY SILT, trace sand, containing silty sand seams Very stiff Grey Moist	8	SS	19			
225.9 / 8.1	END OF BOREHOLE						

Elevation scale: 233, 232, 231, 230, 229, 228, 227, 226

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

NOTE:

1. Water level in open borehole at a depth of 4.6 m, (Elev. 229.4 m) upon completion of drilling

* SPT "N" values considered to have been affected by sample disturbance due to groundwater inflow to borehole

+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 12-08

SHEET 1 OF 2 **METRIC**

PROJECT 09-1111-0022

G.W.P. 2179-10-00 LOCATION N 4920906.8 ;E 292853.8 ORIGINATED BY DD

DIST Central HWY 400 BOREHOLE TYPE Track Mount Power Auger COMPILED BY NLP

DATUM Geodetic DATE April 18, 2012 CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
233.5	GROUND SURFACE						
0.0	TOPSOIL						
0.2	Sand, trace to some silt to silty sand, trace to some clay, (FILL) Compact Brown to grey Moist	1	SS	17	233		
		2	SS	29	232		
		3	SS	10		○	0 61 29 10
231.4 / 2.1	Silty SAND, trace clay Loose Grey Moist	4	SS	8	231		
230.6 / 2.9	CLAYEY SILT, with to some sand, containing sand layers Firm to stiff Grey Moist to wet	5	SS	10	230	○	0 22 63 15
		6	SS	4	229	○	0 10 65 25
227.9 / 5.6	SAND, trace to some silt, trace to some clay, containing clayey silt seams Loose to compact Grey Moist to wet	7	SS	5	228, 227	○	0 75 14 11
225.6 / 7.9	CLAYEY SILT with sand, containing sand layers Stiff Grey Moist to wet	8	SS	13	226		
224.9 / 8.6	SAND to SILT to SAND, trace to some silt, trace clay, trace gravel Loose to compact Brown Moist to wet	9	SS	15	225, 224	○	1 87 7 5
		10	SS	6*	223		
		11	SS	5*	222, 221	○	0 59 30 11
220.3 / 13.2	CLAYEY SILT to SILTY CLAY, trace to some sand, trace gravel Firm to stiff Grey Moist to wet	12	SS	8	220, 219	w_P ├──┤ ○	0 6 52 42

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 09-1111-0022.GPJ GAL-MISS.GDT 7/10/12 DD/SAC

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RECORD OF BOREHOLE No 12-08

SHEET 2 OF 2 **METRIC**

PROJECT 09-1111-0022
G.W.P. 2179-10-00
LOCATION N 4920906.8 ;E 292853.8
ORIGINATED BY DD
DIST Central HWY 400
BOREHOLE TYPE Track Mount Power Auger
COMPILED BY NLP
DATUM Geodetic
DATE April 18, 2012
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	CLAYEY SILT to SILTY CLAY, trace to some sand, trace gravel; Firm to stiff; Grey; Moist to wet	13	SS	8	218	45 / 42.4	0	9	22	69
		14	SS	6	217 / 216					
215.7 / 17.8	Silty SAND, trace gravel, trace clay; Dense to very dense; Brown to grey; Moist to wet	15	SS	38	215					
		16	SS	139	214 / 213		1	64	28	7
212.7 / 20.8	Gravelly SAND, trace silt, trace clay; Very dense; Grey; Wet	17	SS	125/0.15	212					
211.2 / 22.3	SAND, trace silt to silty SAND, trace clay; Very dense; Grey; Wet	18	SS	155	211 / 210					
		19	SS	157	209 / 208					
207.1 / 26.4	END OF BOREHOLE									

NOTES:

1. Water level in open borehole at a depth of 4.6 m, (Elev. 228.9 m) upon completion of drilling

2. Water level in piezometer measured at 1.0 m above ground surface (Elev. 234.5 m) on May 28, 2012

* SPT "N" values considered to have been affected by sample disturbance due to groundwater inflow to borehole

+3, ×3: Numbers refer to Sensitivity   ○3% STRAIN AT FAILURE

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 12-09

SHEET 1 OF 3 **METRIC**

PROJECT 09-1111-0022

G.W.P. 2179-10-00 LOCATION N 4920896.1 ;E 292893.8 ORIGINATED BY DD

DIST Central HWY 400 BOREHOLE TYPE Track Mount Power Auger COMPILED BY NLP

DATUM Geodetic DATE March 29, 2012 CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.8	GROUND SURFACE				
0.0	Sand, some gravel, trace silt (FILL) Compact Brown Moist	1	SS	13	
241.1 / 0.7	Silty sand to sand, trace gravel, trace to some silt, trace clay (FILL) Compact to dense Brown Moist	2	SS	18	
		3	SS	24	
		4	SS	40	0 87 7 6
		5	SS	36	
		6	SS	28	
		7	SS	44	
236.2 / 5.6	SAND and SILT to SAND, trace to some silt, trace clay, trace to some gravel, containing clayey silt layers Compact to very dense Brown to grey Moist	8	SS	20	
		9	SS	62	
		10	SS	30	
231.7 / 10.1	SAND and SILT, trace gravel, trace to some clay, containing clayey silt layers Loose to compact Grey Moist to wet	11	SS	8	
		12	SS	14	2 48 40 12
228.6 / 13.2	CLAYEY SILT, trace sand Stiff Grey Moist to wet	13	SS	9	0 1 71 28
227.1 / 14.7					

GROUND WATER CONDITIONS: water level symbol at approx. elev. 231.8

ELEVATION SCALE: 241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 09-1111-0022.GPJ GAL-MISS.GDT 7/10/12 DD/SAC

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RECORD OF BOREHOLE No 12-09

METRIC

PROJECT 09-1111-0022

G.W.P. 2179-10-00

LOCATION N 4920896.1 ;E 292893.8

ORIGINATED BY DD

DIST Central HWY 400

BOREHOLE TYPE Track Mount Power Auger

COMPILED BY NLP

DATUM Geodetic

DATE March 29, 2012

CHECKED BY LCC

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L | WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	Silty SAND to SAND, trace to some silt, trace clay, trace gravel, containing silty clay layers from 15.2 m to 15.7 m Compact to dense Brown to grey Moist to wet	14	SS	14				
		15	SS	30				
		16	SS	24	0	83	10	7
		17	SS	17				
		18	SS	21				
		19	SS	34				
217.9 23.9	CLAYEY SILT with to some sand Stiff Grey Moist to wet	20	SS	12	0	26	42	32
		21	SS	9				
214.9 26.9	Gravelly SAND, some silt, trace clay Loose Grey Wet	22	SS	2*				
		23	SS	WH*				
211.8								

Elevation scale: 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212

Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 12-09

SHEET 3 OF 3 **METRIC**

PROJECT 09-1111-0022
G.W.P. 2179-10-00
LOCATION N 4920896.1 ;E 292893.8
ORIGINATED BY DD
DIST Central HWY 400
BOREHOLE TYPE Track Mount Power Auger
COMPILED BY NLP
DATUM Geodetic
DATE March 29, 2012
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
30.0	CLAYEY SILT with sand, trace to some gravel, (TILL) Very stiff Grey Moist	24	SS	22	211		8 44 25 23
					210		
209.6 / 32.2	SAND and GRAVEL, trace to some silt, trace clay Very dense Grey Wet	25	SS	52	209 / 208		31 56 9 4
207.8 / 34.0	END OF BOREHOLE						

NOTE:

1. Water level in open borehole at a depth of 10.0 m, (Elev. 231.8 m) upon completion of drilling

* SPT "N" values considered to have been affected by sample disturbance due to groundwater inflow to borehole

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; STRAT PLOT

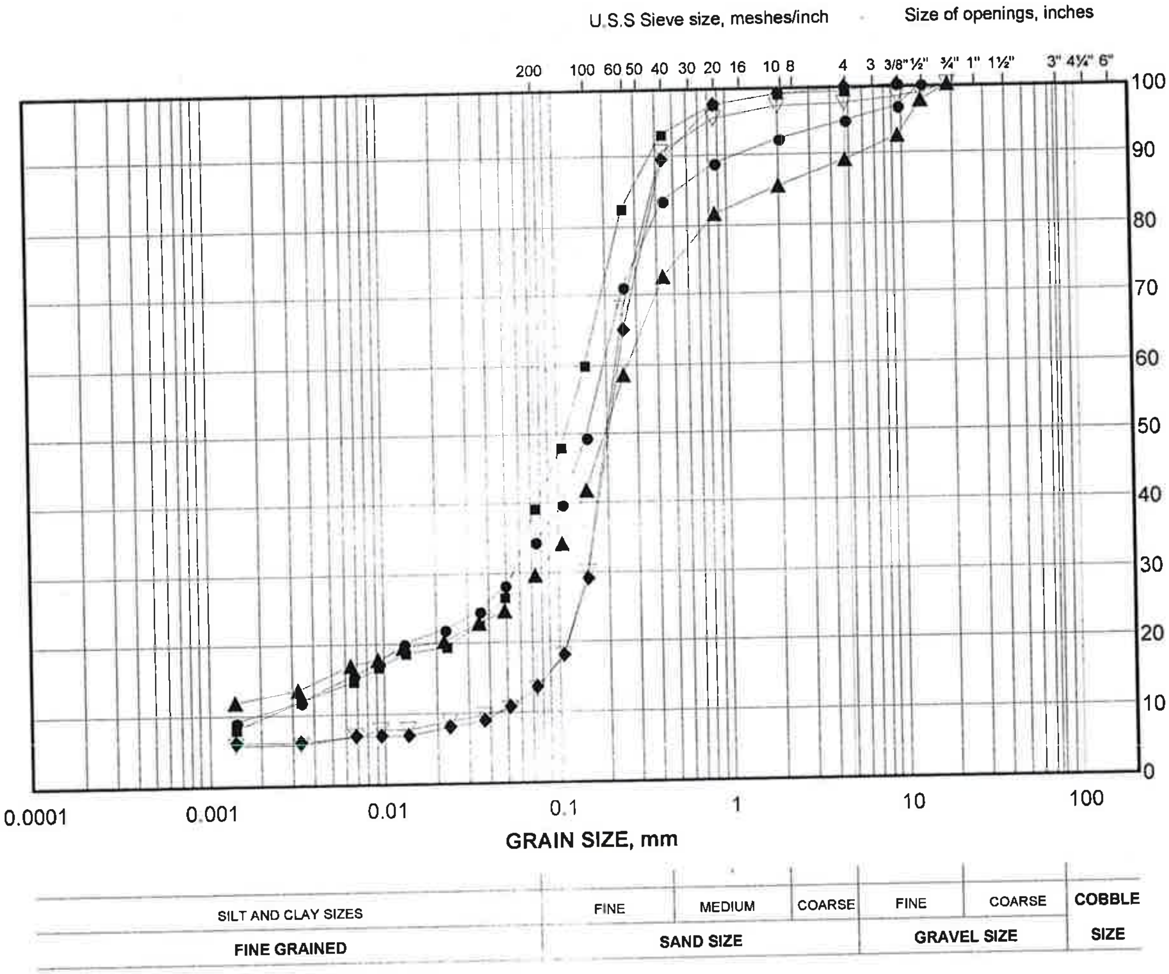
+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION
Silty Sand to Sand (Fill)

FIGURE B1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-06	3	234.8
■	12-08	3	232
◆	12-09	4	239.5
▲	12-04	4	234.9
▽	12-01	6	243

Project Number: 09-1111-0022

Checked By:

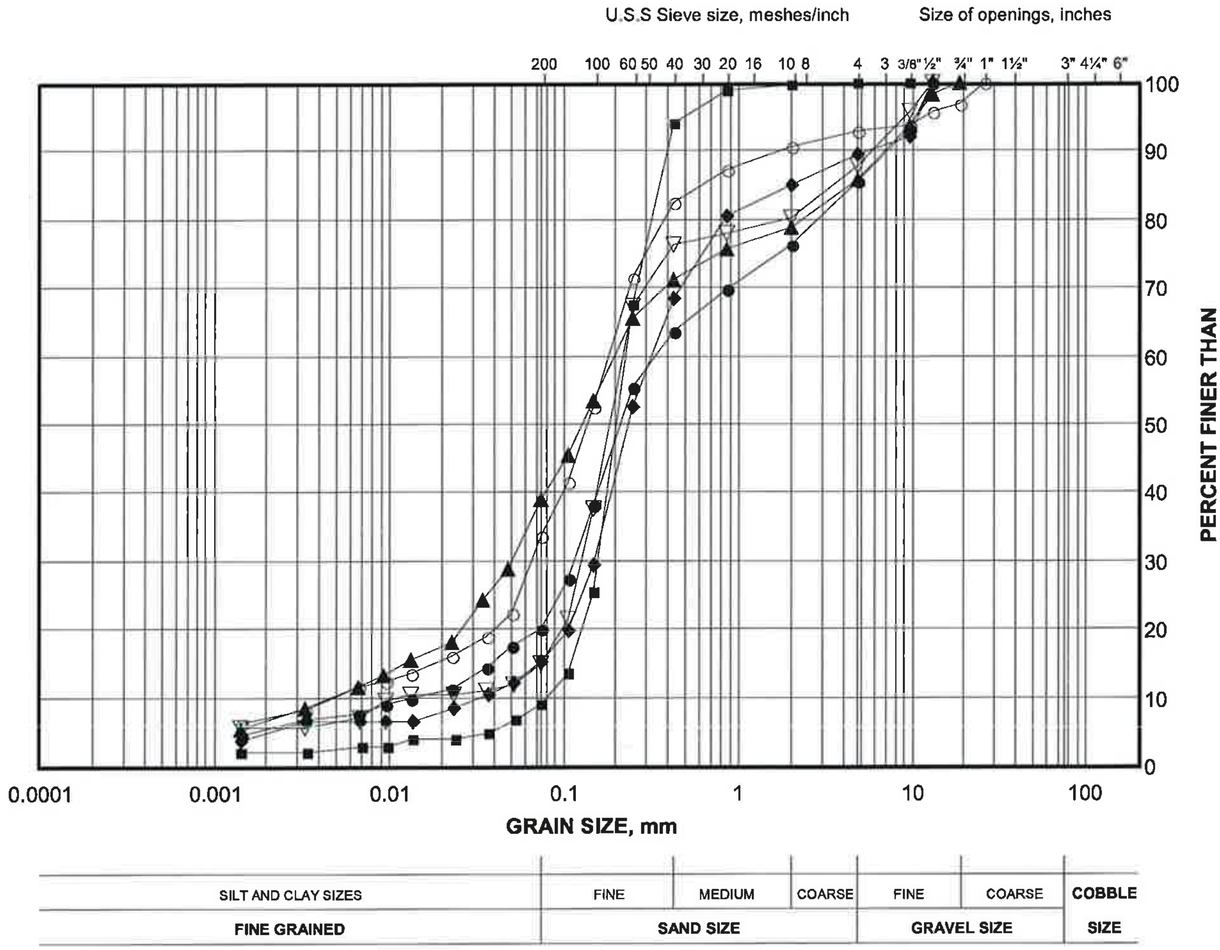
Golder Associates

Date: 12-Jun-12

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand

FIGURE B2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-01	12	234.6
■	12-01	14	231.6
◆	12-03	3	233.6
▲	12-02	3	235.8
▽	12-02	7	231.2
○	12-01	9	239.2

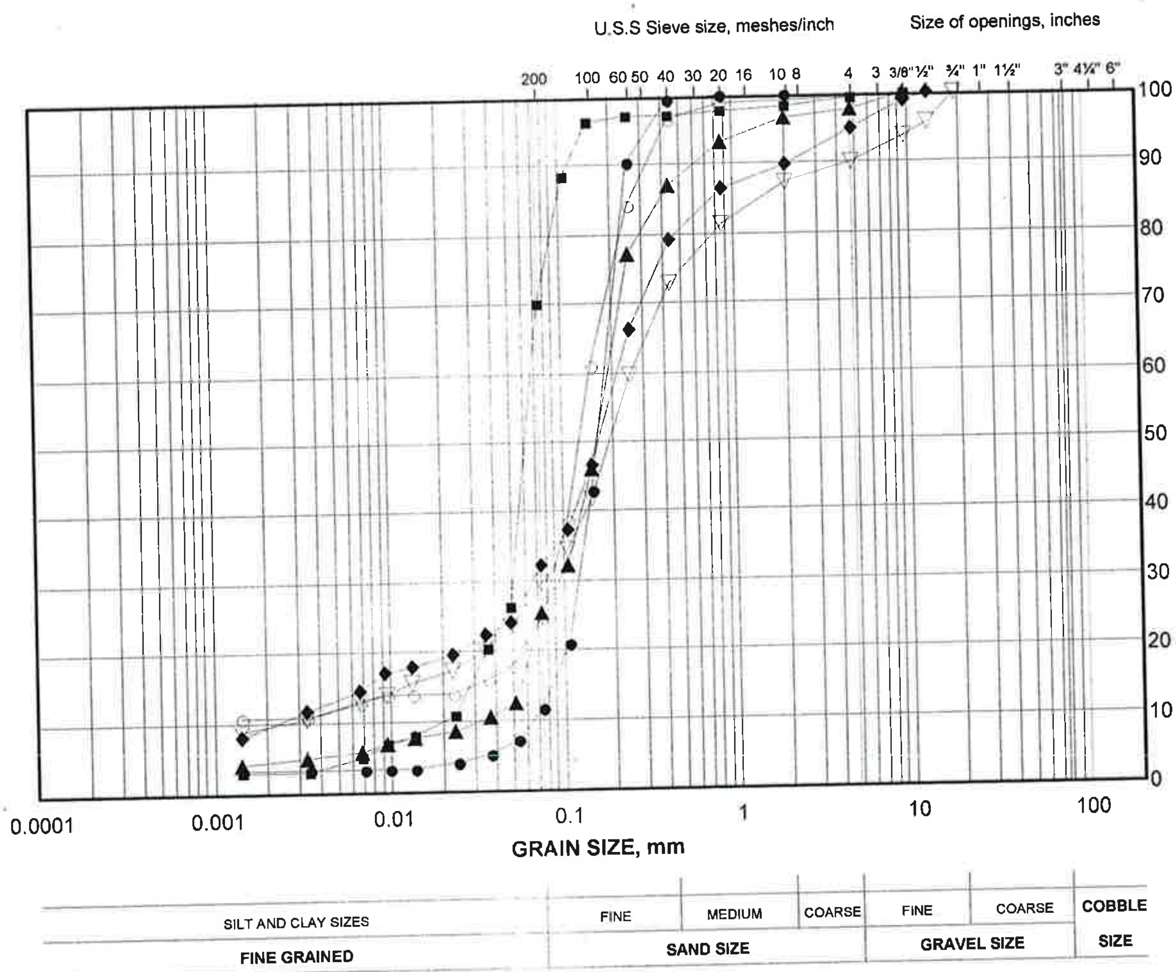
Project Number: 09-1111-0022

Checked By:

Golder Associates

Date: 29-Jun-12

GRAIN SIZE DISTRIBUTION Silty Sand to Sandy Silt	FIGURE B3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-04	10	226.5
■	12-06	17	215
◆	12-05	3	233.9
▲	12-07	3	232.5
▽	12-04	6	232.6
○	12-08	7	227.4

Project Number: 09-1111-0022

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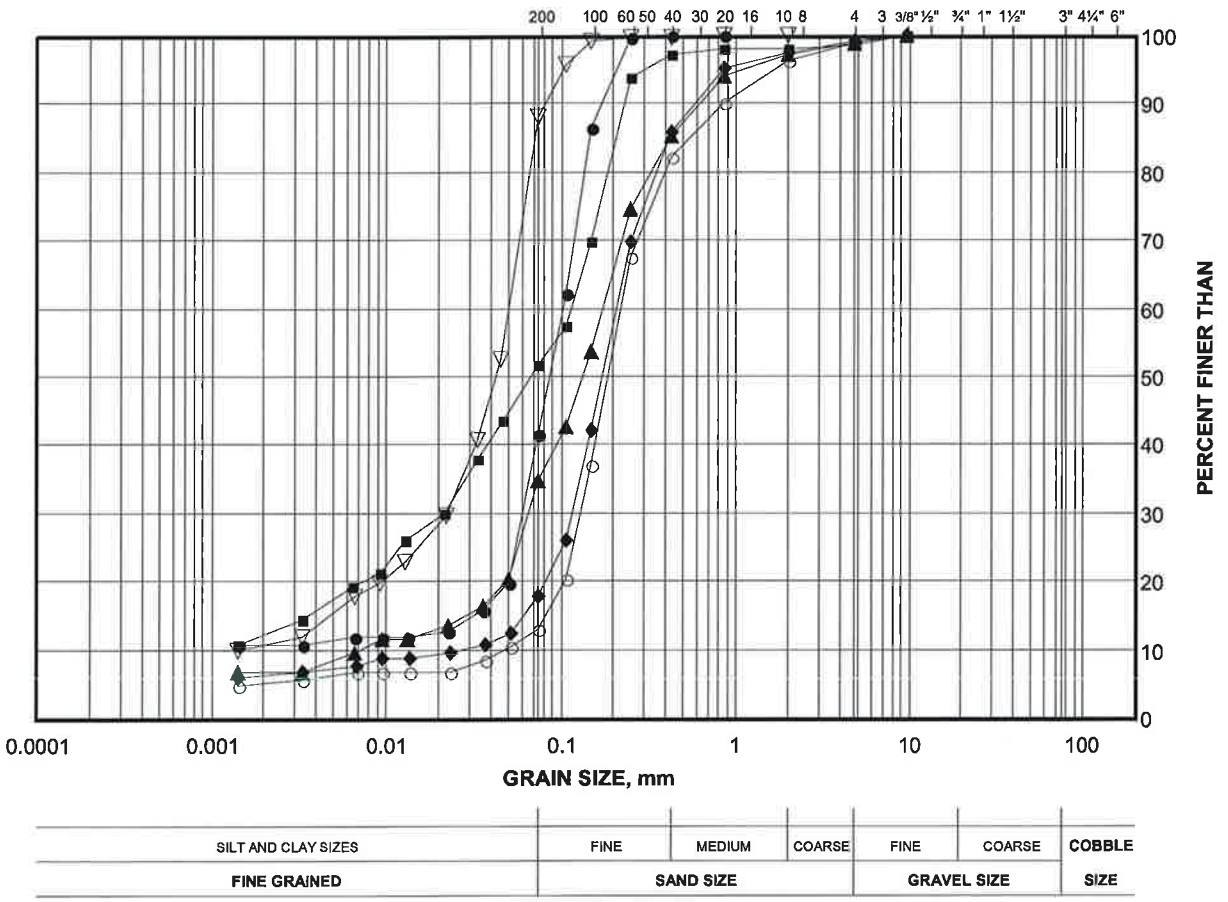
Date: 12-Jun-12

GRAIN SIZE DISTRIBUTION

Silty Sand to Silt

FIGURE B4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-08	11	221.3
■	12-09	12	229.6
◆	12-09	16	223.5
▲	12-08	16	213.7
▽	12-06	7	230.2
○	12-08	9	224.4

Project Number: 09-1111-0022

Checked By: [signature]

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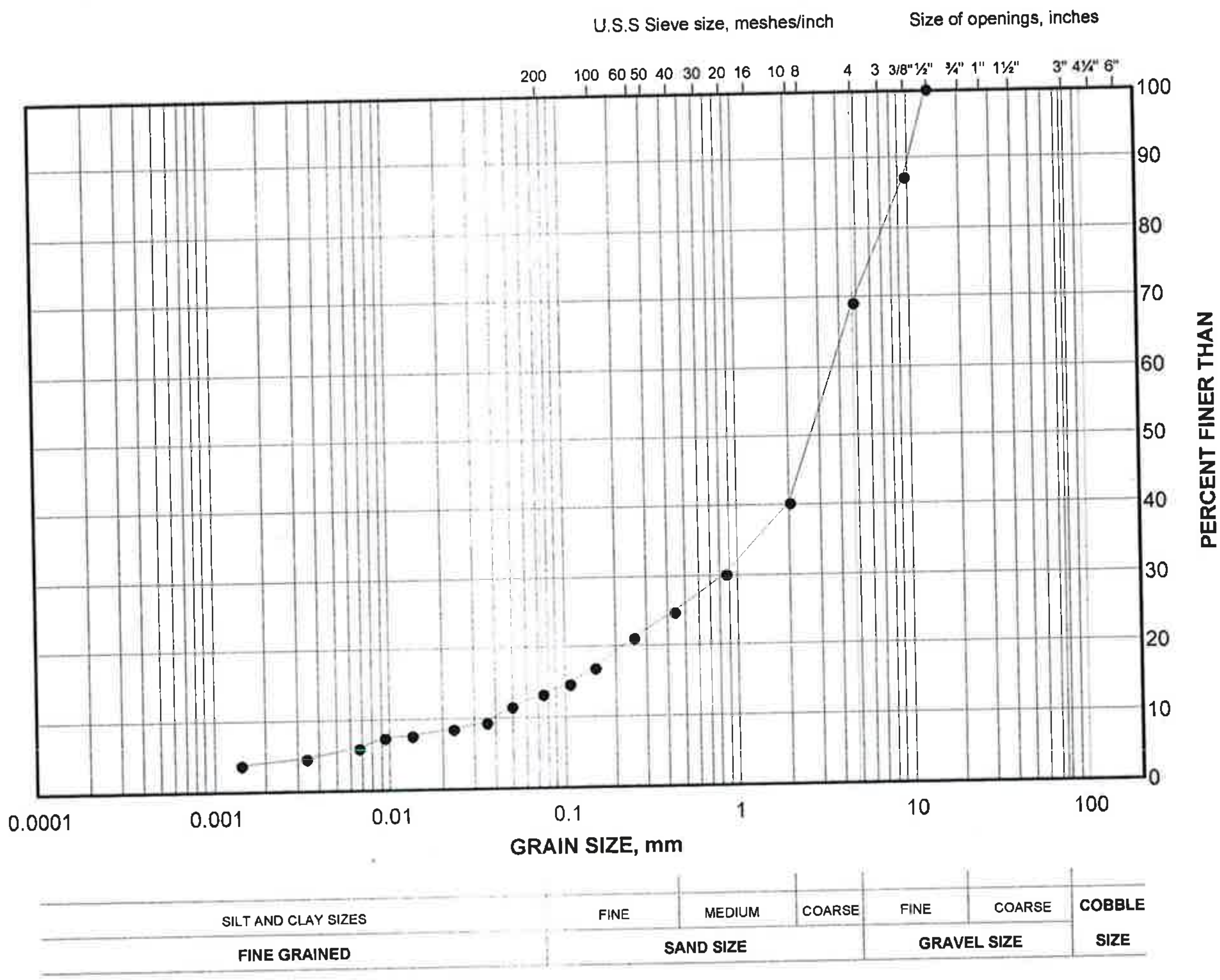
Date: 29-Jun-12

GRAIN SIZE DISTRIBUTION

Sand and Gravel

FIGURE B5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-09	25	208.3

Project Number: 09-1111-0022

Checked By:

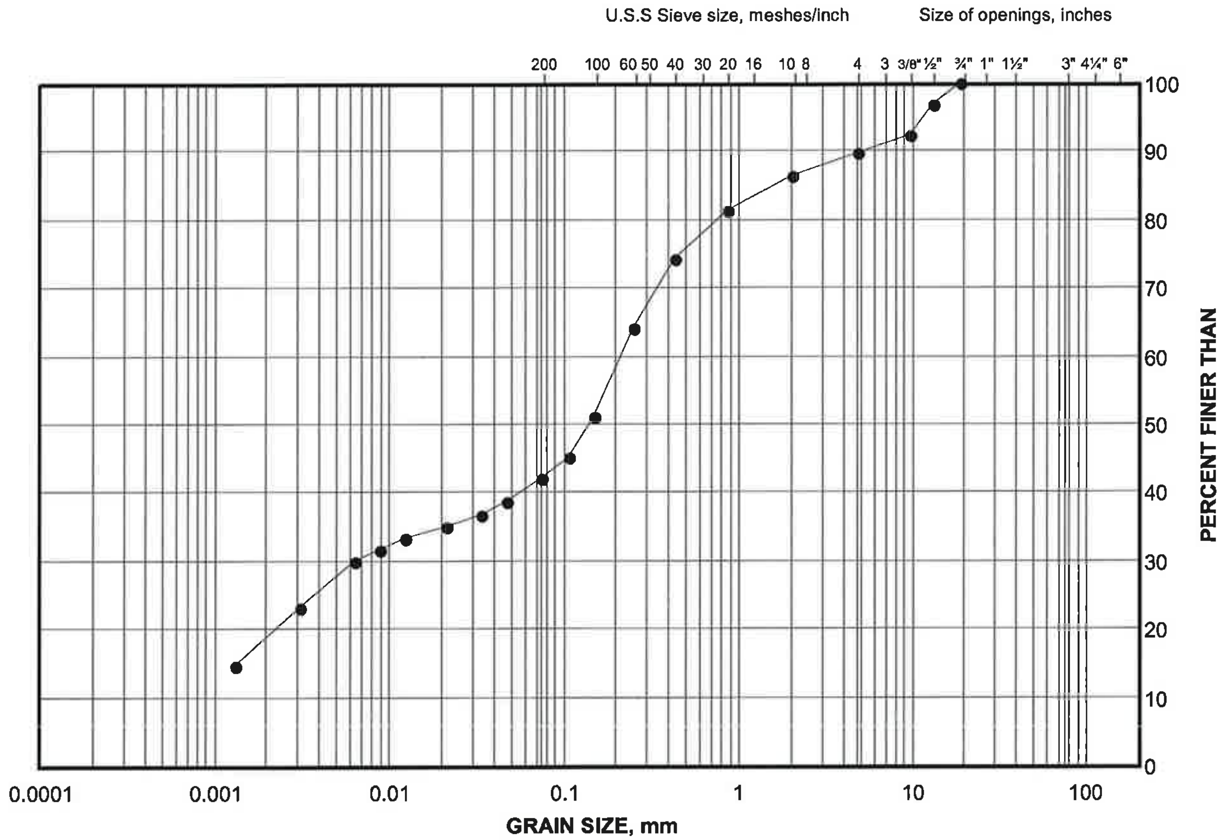
Golder Associates

Date: 12-Jun-12

GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE B6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-02	5	234.2

Project Number: 09-1111-0022

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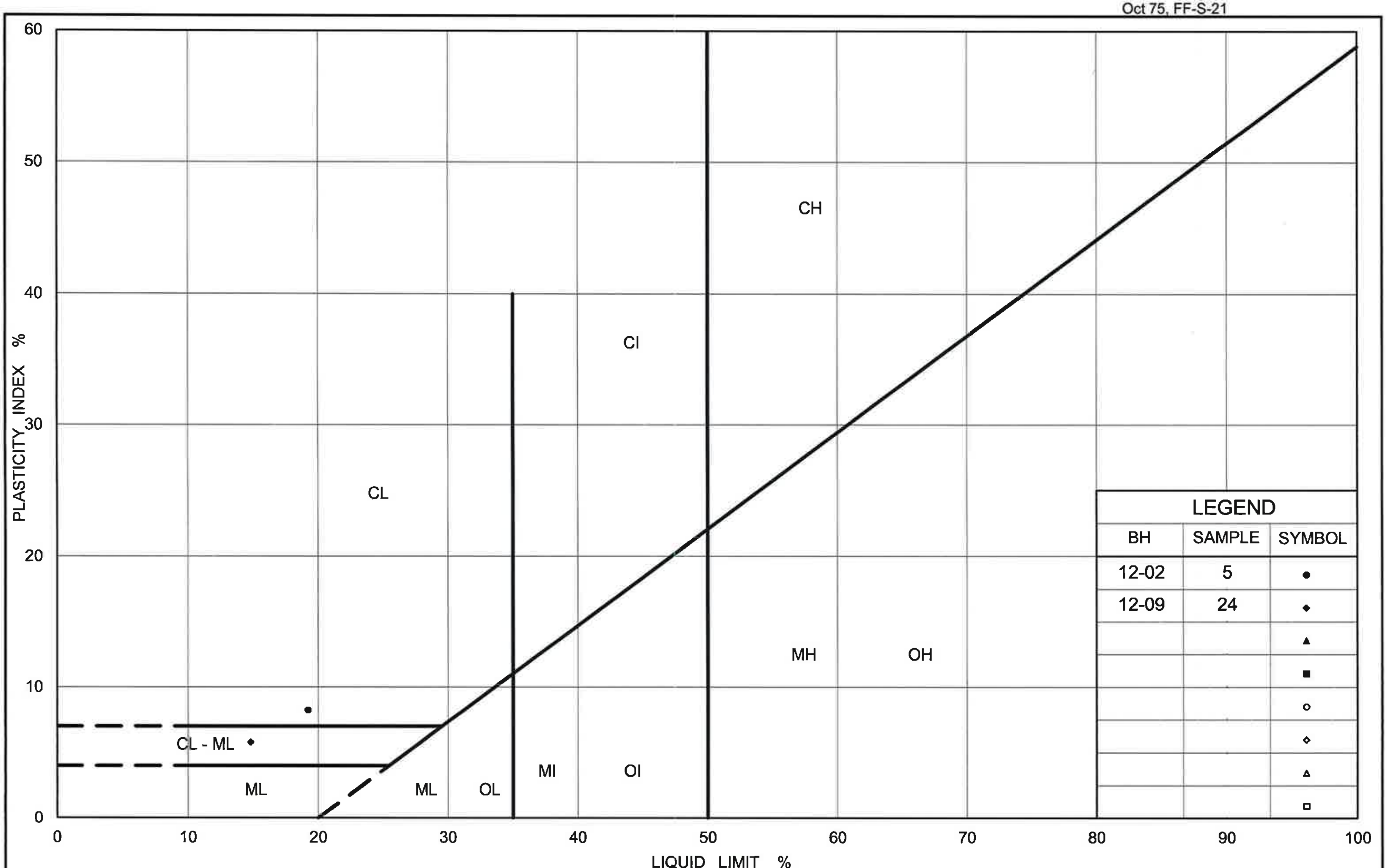
Date: 29-Jun-12

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt Till

Figure No. B7

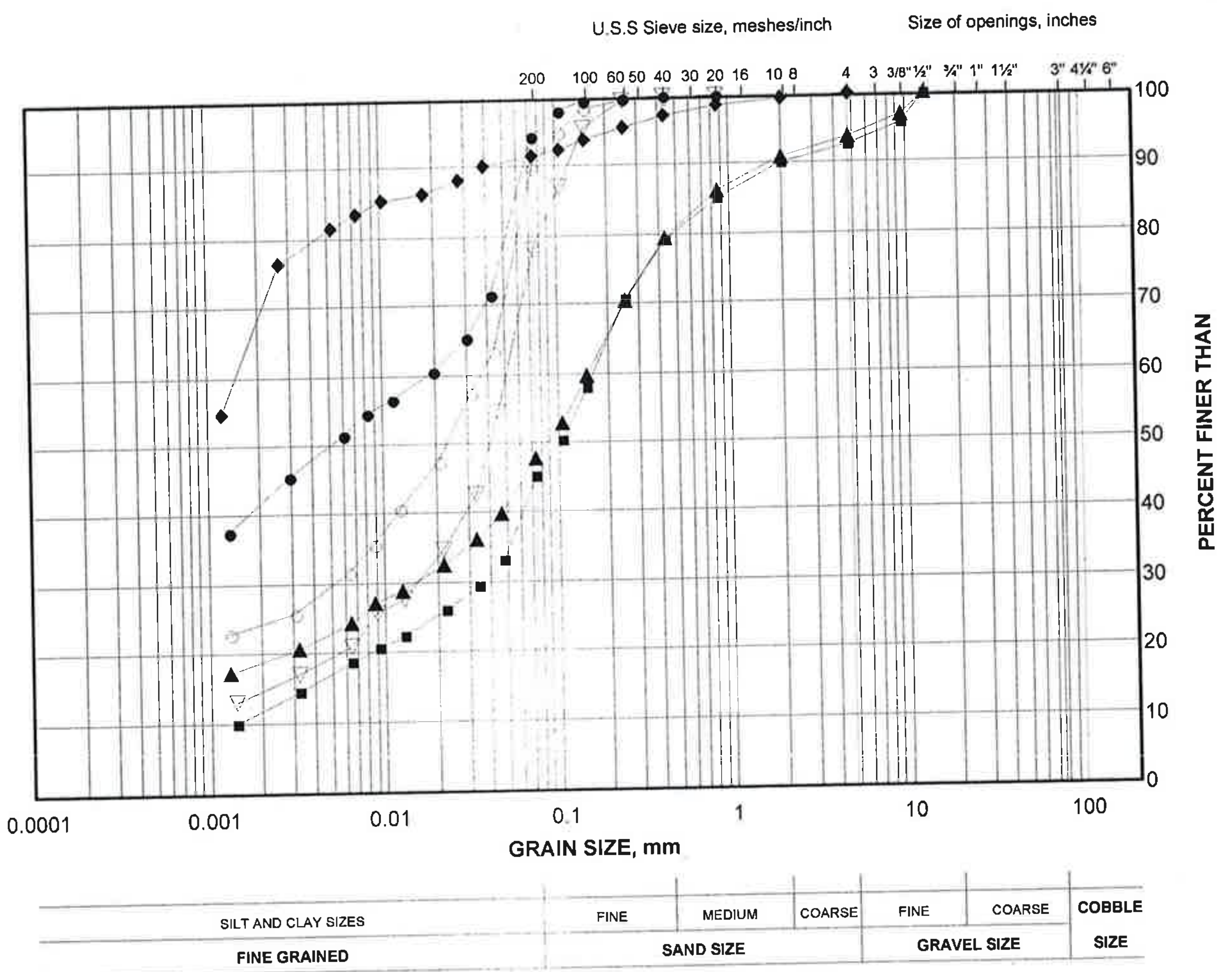
Project No. 09-1111-0022

Checked By: [signature]

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay

FIGURE B8



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-08	12	219.8
■	12-06	13	221.1
◆	12-08	13	218.3
▲	12-06	14	219.5
▽	12-08	5	230.4
○	12-08	6	228.9

Project Number: 09-1111-0022

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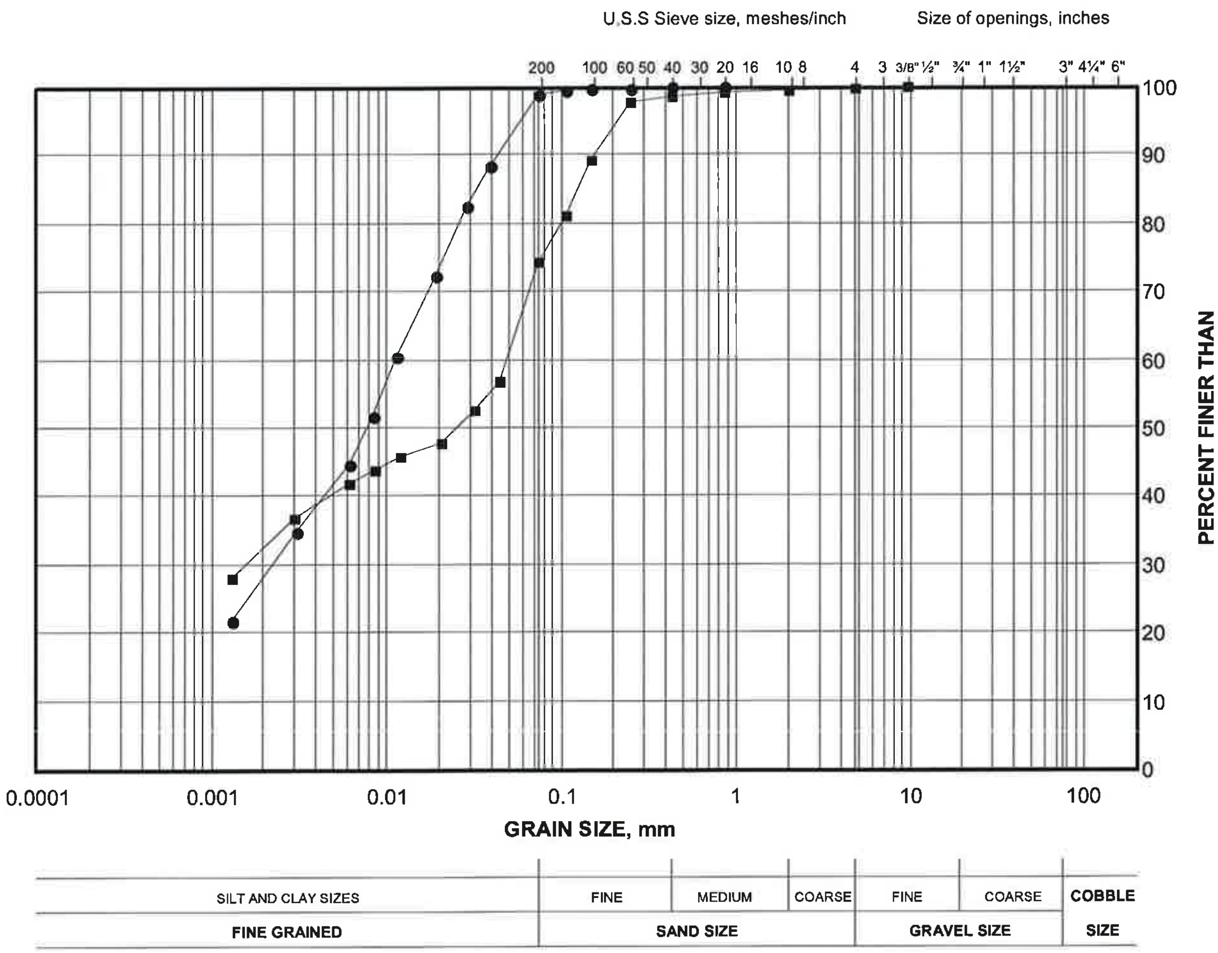
Date: 12-Jun-12

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay

FIGURE B9



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	12-09	13	228.1
■	12-09	20	217.4

Project Number: 09-1111-0022

Checked By: [signature]

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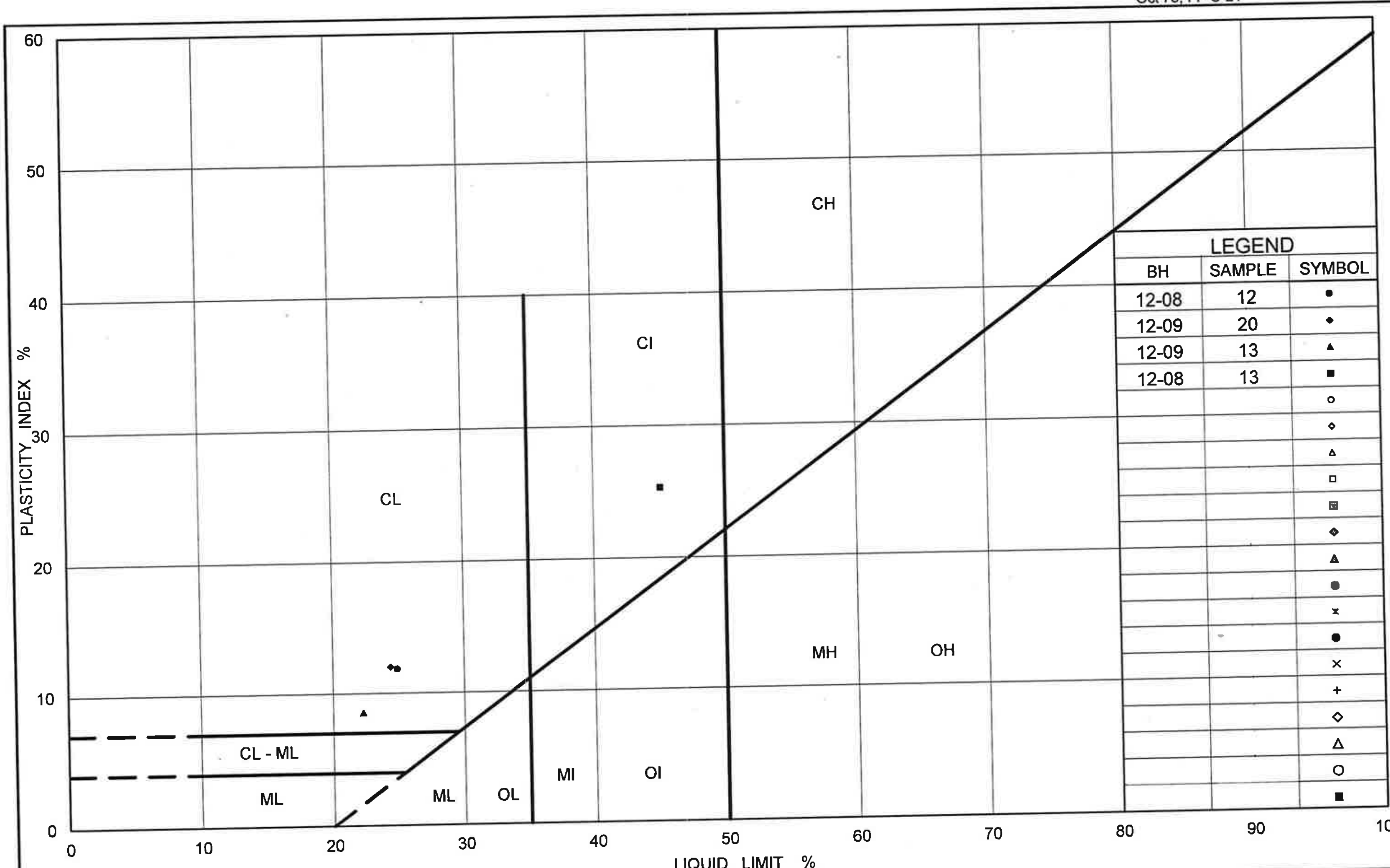
Date: 29-Jun-12

Oct 75, FF-S-21



Ministry of Transportation

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PLASTICITY CHART
Clayey Silt to Silty Clay

Figure No. B10

Project No. 09-1111-0022

Checked By: CMS

APPENDIX C

Non-Standard Special Provisions

DEWATERING – Item No.

Special Provision

SCOPE

The work under this item includes the design, installation, operation, maintenance and removal of temporary dewatering systems to facilitate the construction of the pier foundations at the Crown Hill overpass site.

Construction of the pier foundations will require excavation to near or below the groundwater level in the sand and silt to gravelly sand deposit. Cohesionless soils below the groundwater table will be subjected to conditions of unbalanced hydrostatic head and can slough, boil and cave in during temporary excavation work.

REFERENCES

OPSS 518 Construction Specification for Control of Water from Dewatering Operations

SUBMISSION AND DESIGN REQUIREMENTS

Written details for the proposed dewatering system shall be submitted to the Contract Administrator for information purposes a minimum of ten business days prior to commencing dewatering operations. The Contractor shall reference borehole logs included in the Contract Documents as a guide in determining requirements.

CONSTRUCTION

Dewatering System

The Contractor is responsible for the design, installation, operation and maintenance of an adequate dewatering system to lower the groundwater level to at least 0.3 m below the founding level for the piers, to allow excavation, subgrade preparation and foundation construction in dry conditions.

Operation

A continuous dewatering operation shall be provided to facilitate the pier foundation construction at all times during the work. All components of the dewatering system shall be maintained in an effective, functioning and stable condition at all times during the work. Notwithstanding the above, the work shall be completed in accordance with the environmental and operational constraints specified elsewhere in the contract.

Restoration

All equipment and materials placed shall be removed from the right-of-way upon the completion of the work and all areas disturbed as part of this work shall be restored to their preconstruction conditions, unless specified otherwise.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work.

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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