November 2012

FOUNDATION INVESTIGATION AND DESIGN REPORT

HIGHWAY 9 UNDERPASS
HIGHWAY 400 WIDENING FROM NORTH OF KING ROAD TO SOUTH CANAL ROAD
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 2835-02-00

Submitted to:
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REPORT

Golder Associates

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PART A

FOUNDATION INVESTIGATION REPORT
HIGHWAY 9 UNDERPASS
HIGHWAY 400 WIDENING FROM NORTH OF KING ROAD TO SOUTH CANAL ROAD
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the proposed widening and replacement of Highway 400 from north of King Road to South Canal Road in the Regional Municipality of York, Ontario.

This report addresses the investigation carried out for the Highway 9 underpass structure and the associated approach embankments. The purpose of this investigation is to establish the subsurface conditions at the location of the proposed structure, including the associated approach embankments, by borehole drilling and laboratory testing on selected samples.

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

2.0 SITE DESCRIPTION

The Highway 9 underpass is located at the intersection of Highway 400 and Highway 9, approximately 3 km north of the Lloydtown-Aurora Road interchange, in the Regional Municipality of York, Ontario. The existing structure consists of an approximately 34 m long by 28 m wide two-span twinned bridge with the abutments and piers supported on spread footings.

In general, the topography throughout the project limits consists of rolling terrain covered by agricultural fields and densely treed areas, with commercial facilities located along the Highway 400 corridor.

The natural ground surface at the site varies from approximately Elevation 244.5 m to 247.5 m; to the north of Highway 9, the ground surface slopes downward to the Holland Marsh and Schomberg River. Highway 400 and the interchange ramps have been constructed in a cut, with the existing Highway 400 grade in the general area of the underpass varying between about Elevation 239.5 m and 240.3 m. The existing Highway 9 pavement varies between about Elevation 248.0 m and 248.5 m.

The existing Highway 400 cut slopes, which are up to about 8 m high, are oriented at approximately 2 horizontal to 1 vertical (2H:1V).

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigation by Others

During the preliminary foundation investigation for the twinning of the Highway 9 underpass structure, a total of six boreholes, designated as Boreholes 97-1 to 97-6, were advanced at this site by Thurber Engineering Ltd. (Thurber). The results of the Thurber investigation are contained in their report titled "Foundation Investigation Report for Highway 400-Highway 9 Underpass Widening and Slope Stability Assessment for Re-alignment of Highway 9 E to Highway 400 S Ramp", Report No. 15-64-2, dated May 8, 1997. The locations of the boreholes advanced by Thurber are shown on Drawing 1 and the borehole records used to supplement the current investigation are presented in Appendix B.

The Thurber boreholes are located approximately 15 m to 25 m away from the proposed foundation elements, and thus additional boreholes were advanced at each proposed foundation element as part of the current investigation program. However, the Thurber investigation results show that refusal (soil having Standard Penetration Test 'N'-values of greater than 100 blows per 0.3 m of penetration) was encountered at depths between about 8 m and 9 m (approximately Elevation 232 m), and reference to the subsurface conditions at these borehole location is made where appropriate in the following sections of the report.

3.2 Current Investigation

The field work for the detail foundation investigation of the Highway 9 underpass site was carried out between October 15 and November 19, 2010, during which time a total of eight sampled boreholes were advanced at the bridge site: two boreholes were drilled in the vicinity of the proposed west abutment; two boreholes were drilled in the vicinity of the proposed centre pier; two boreholes were drilled in the vicinity of the proposed east abutment; and one (1) borehole was advanced in the vicinity of both the west and east approach embankments. The boreholes, designated as Boreholes HN1 to HN8, were advanced at the locations shown on Drawing 1.

The field investigation was carried out using track-mounted D-50 and D-90 drill rigs, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced using 108 mm inside diameter hollow stem augers or 108 mm outside diameter solid stem augers. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08a)[1]. All the boreholes advanced at the proposed foundation elements were advanced into a stratum of equivalent SPT "N"-values equal to or greater than 100 blows per 0.3 m of penetration when corrected for the higher energy automatic hammer used during this investigation. The depths of the boreholes range from about 9.8 m to 20.4 m below existing ground surface.

The groundwater conditions in the open boreholes were observed during the drilling operations and one piezometer was installed in Borehole HN7 to permit monitoring of the water level at this location. The installed piezometer consists of a 50 mm diameter PVC pipe, with a 3 m slotted screen sealed within a filter sand pack at a depth of 19.5 m below ground surface. The borehole and annulus surrounding the piezometer pipe above the filter sand pack was backfilled to the ground surface with bentonite pellets/cement grout. Piezometer installation details and water level readings are described on the borehole records presented following the text of the report. All boreholes in which standpipe piezometers were not installed were backfilled to ground surface with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, monitored the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga

[1] ASTM D1586-08a – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the soil.

geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples.

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

Borehole	Location (MTM NAD 83)		Ground Surface Elevation (m)	Depth Drilled (m)
	Northing (m)	Easting (m)		
HN1	4,876,591.2	297,170.0	248.4	12.8
HN2	4,876,612.1	297,196.1	248.2	20.4
HN3	4,876,590.9	297,204.2	248.2	18.9
HN4	4,876,628.8	297,241.2	239.5	9.8
HN5	4,876,595.3	297,248.8	240.3	14.1
HN6	4,876,638.0	297,280.8	247.9	17.4
HN7	4,876,615.6	297,284.9	248.0	20.2
HN8	4,876,634.4	297,317.5	248.0	11.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The 23 km section of Highway 400 included in this project traverses, in a south–north direction, the physiographic regions known as South Slope, Oak Ridges Moraine and Simcoe Lowlands, according to *The Physiography of Southern Ontario (Chapman and Putman, 1984)*[2]. Along Highway 400, the South Slope is present south of King Road; the Oak Ridges Moraine extends from north of King Road to south of Highway 9; and the Simcoe Lowlands occupy a 4 km wide strip extending from south of Highway 9 to Holland River. The Highway 9 underpass site is located within the Simcoe Lowlands physiographic region.

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

The Holland River valley, which crosses Highway 400 just north of Highway 9 and South Canal Road, is located within the Simcoe Lowlands region. This valley extends to the southwest from Cook Bay at the south end of Lake Simcoe, and was once a shallow extension of the lake. The floor of the valley consists of peat, soft clays

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

and loose sands. It is understood that during initial construction of Highway 400 through this area, a layer of peat about 2 m to 3 m thick was removed in order to construct the road upon the underlying sand and clay.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced for the detail foundation investigation, together with results of the laboratory tests carried out on selected soil samples, are provided on the Record of Borehole sheets presented in Appendix A. The borehole records and results of the laboratory testing from the Thurber investigation are provided in Appendix B, following the text of this report. The stratigraphic boundaries shown on the borehole records are inferred from non continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The interpreted stratigraphy in profile along Highway 9 and in cross section at the abutment and pier location is shown on Drawings 1 and 2, and is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, the subsurface conditions at the Highway 9 underpass site consist of asphalt/topsoil and fill on the highway alignments. The fill is underlain by upper till deposits that vary in composition from clayey silt to sand and silt to sand, which in turn are underlain by a deposit of clayey silt. The clayey silt (where present) is underlain by a lower till deposit that varies in composition from clayey silt to sand and silt. The soil deposits contain occasional pockets and interlayers of clayey silt, sand and silt, silty sand and sand.

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

4.2.1 Asphalt

A layer of asphalt about 0.1 m to 0.3 m thick was encountered immediately below the ground surface in all the boreholes advanced as part of the current investigation. The records for Boreholes 97-1, 97-6 and 97-7 advanced by Thurber note that topsoil was encountered immediately below the ground surface at the time of the investigation; these boreholes were drilled before the former northward widening of Highway 9 and northward twinning of the existing bridge structure.

4.2.2 Fill

Fill was encountered underlying the asphalt in all the boreholes drilled at this site. The thickness of the fill deposit is variable across the site. In Boreholes HN2 and HN6, which were drilled through the existing Highway 9 embankment on the westbound shoulder near the west and east abutments, the fill extends to depths of about 3.7 m and 2.2 m (Elevation 244.5 m and 245.7 m), respectively. At all the other borehole locations, the fill extend to depths of about 0.6 m and 1.9 m below ground surface (between Elevation 247.4 m and 246.6 m).

The fill material is variable in composition. In general, cohesionless fill was encountered below the asphalt layer in all the boreholes, and it varies from sand containing trace to some silt; to sand and silt containing trace to some gravel; to gravelly sand containing some silt; to sand and gravel containing trace silt, all containing trace to some clay. Cohesive fill was encountered between the cohesionless fill in Borehole HN2, and it consists of clayey silt with sand, containing trace gravel.

The SPT "N"-values measured within the cohesionless portions of the fill generally range from 11 blows to 55 blows per 0.3 m of penetration, indicating a compact to very dense relative density. SPT "N"-values of 64 blows per 0.3 m of penetration and 79 blows per 0.23 m of penetration were encountered within the cohesionless fill in Boreholes HN2 and HN3; these high values are attributed to the high percentage of gravel within the fill material in the vicinity of these samples. The SPT "N"-value measured within the cohesive fill was 13 blows per 0.3 m of penetration, suggesting a stiff consistency.

The grain size distribution test results for three samples of the sand to gravelly sand fill are shown on Figure 1 in Appendix A. The natural water content measured on samples of the cohesionless fill ranges from 2 per cent to 8 per cent.

4.2.3 Clayey Silt Till (Upper Deposit)

A cohesive upper till deposit was encountered below the fill in Boreholes HN1 and HN3 advanced near the proposed west abutment and in Borehole HN7 advanced near the east abutment. The top of the till deposit was encountered between about Elevation 247.4 m and 246.6 m. The base of this deposit extends to depths of between about 2.2 m and 3.9 m below ground surface, corresponding to about Elevation 245.8 m and 243.4 m.

The SPT "N"-values measured within the till deposit range from 9 blows to 82 blows per 0.3 m of penetration, and an "N"-value of 110 blows per 0.3 m of penetration was recorded at the interface of the clayey silt till with the underlying sand and silt till, generally suggesting a stiff to hard consistency.

This till deposit consists of clayey silt with sand to trace sand, and trace gravel. During drilling, grinding of the augers was noted in Borehole HN3 at a depth of about 1.4 m (about Elevation 246.8 m), and this has been inferred to represent the presence of cobbles and/or boulders within the till. Atterberg limits tests were carried out on two samples of the cohesive till deposit. The liquid limits were about 16 per cent and 19 per cent, the plastic limits were about 10 per cent and 11 per cent, and the plasticity indices were about 6 per cent and 8 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure 2 in Appendix A and indicate that the material is clayey silt till of low plasticity.

The natural water content measured on samples of the clayey silt till ranges from 8 per cent to 9 per cent.

4.2.4 Sand and Silt Till and Sand Till (Upper Deposit)

Underlying the upper clayey silt till in Borehole HN1, a deposit of cohesionless till was encountered to a depth of about 6.4 m (Elevation 242.0 m). Underlying the fill in Borehole HN6, a layer of cohesionless till was encountered to a depth of about 3.5 m (Elevation 244.4 m).

The SPT "N"-values measured within the cohesionless till deposit range from 48 blows to 110 blows per 0.3 m of penetration, indicating a dense to very dense relative density.

This portion of the upper till consists of sand and silt to silt, some sand, containing trace gravel and trace to some clay. Grain size distribution tests were carried out on two samples of this deposit and the results are shown on Figure 3 in Appendix A.

The natural water content measured on samples of the sand and silt till and sand till ranges from 3 per cent to 7 per cent.

4.2.5 Clayey Silt

A deposit of brown to grey clayey silt was encountered below the fill and/or upper till deposit in Boreholes HN1, HN2, HN3, HN7 and HN8, which were advanced near the proposed west and east abutments and approach embankments, above the Highway 400 cut grade. The top of this deposit was encountered at depths between about 3.7 m and 6.4 m in boreholes drilled near the proposed west abutment (between about Elevations 242.0 m and 244.5 m). In Boreholes HN7 and HN8 advanced in vicinity of the proposed east abutment, the surface of the clayey silt deposit was encountered at shallower depths of about 2.2 m and 1.5 m, corresponding to Elevation 245.8 m and 246.6 m, respectively. In general, the clayey silt deposit is thicker at the west abutment than at the east abutment; it is up to about 2.2 m at the east abutment and between about 4.5 m and 5.0 m at the west abutment.

The clayey silt deposit contains trace to some sand as well as seams, interlayers or lenses of silty sand to sandy silt and silty clay. In Borehole HN2, advanced through the Highway 9 westbound shoulder near the proposed west abutment, an approximately 0.5 m thick layer of silty sand, trace gravel and trace clay containing silt seams was encountered within the clayey silt deposit.

The SPT “N”-values measured within the clayey silt deposit range from 19 blows to 116 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency. Typically higher SPT “N”-values were measured at boreholes advanced near the west abutment compared to the east abutment.

Atterberg limits tests were carried out on eight samples of this deposit. The liquid limits range from about 22 per cent to 27 per cent, the plastic limits range from about 11 per cent to 14 per cent and the plasticity indices range from about 9 per cent to 13 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure 4 in Appendix A and indicate that this material is a clayey silt of low plasticity.

Grain size distribution tests were carried out on four samples of the clayey silt deposit and the results are shown on Figure 5 in Appendix A.

The natural water content measured on samples of this deposit ranges from about 11 per cent to 20 per cent.

4.2.6 Clayey Silt Till to Sand and Silt Till (Lower Deposit)

A predominantly cohesive lower till deposit was encountered underlying the clayey silt stratum in Boreholes HN1, HN2, HN3, HN5, HN7 and HN8, below the upper sand till in Borehole HN6, and below the fill in Borehole HN4. The top of the lower till deposit ranges from about Elevation 238.3 m to 239.5 m at the proposed west approach, west abutment and centre pier; at the east abutment and east approach, the surface of the deposit was encountered between about Elevation 244.4 m and 243.5 m. Within the cohesive till deposit, a 0.9 m to 2.0 m thick granular till zone was encountered in Boreholes HN6, HN7 and HN8 at a depth of about 8.7 m (between about Elevation 239.2 m and 239.4 m). All boreholes were terminated within this lower till deposit between about Elevation 236.7 m and 226.2 m.

The lower till deposit consists predominantly of clayey silt containing some sand and trace gravel, as well as seams or lenses of sand to silt and silty clay. The cohesionless portion of the lower till varies in composition from sand and silt to silt, trace sand, containing trace gravel and trace clay. Within the lower till deposit, interlayers of clayey silt and sand to silty sand were encountered at varying elevations throughout the deposit. Grinding of the augers occurred between about Elevation 236.0 m and 238.1 m in Borehole HN3, and this is inferred to represent the presence of cobbles and/or boulders within the lower till deposit.

The SPT "N"-values measured within the lower clayey silt till deposit range from 10 blows to 144 blows per 0.3 m of penetration, and are typically greater than 30 blows per 0.3 m of penetration, suggesting a firm to hard (and typically hard) consistency. The lower SPT "N"-values were mostly recorded near the interface of the clayey silt till and its interlayers, while the higher SPT "N"-values were recorded within the lower portion of the clayey silt till deposit. Also, SPT "N"-values of 50 blows per 0.05 m, 70 blows per 0.1 m and 100 blows per 0.25 m were recorded within the clayey silt till deposit prior to the termination of the borehole. In the sand and silt to silt till zone, the measured SPT "N"-values range between about 41 blows and 79 blows per 0.3 m of penetration, indicating that this portion of the lower till deposit has a dense to very dense relative density.

Atterberg limits tests were carried out on twenty samples of the lower clayey silt till deposit. The liquid limits range from about 15 per cent to 23 per cent, the plastic limits range from about 10 per cent to 12 per cent and the plasticity indices range from about 5 per cent to 12 per cent. The results of the Atterberg limits tests are shown on plasticity charts on Figures 6A to 6C in Appendix A and indicate that this material is a clayey silt till of low plasticity.

The grain size distribution test results for eleven samples of the lower clayey silt till deposit are shown on Figures 7A and 7B in Appendix A, and the results for two samples of the sand and silt to silt till deposit are shown on Figure 8 in Appendix A.

The natural water content measured on samples of the clayey silt till ranges from 7 per cent to 16 per cent and the water content measured on samples of the sand and silt to silt till ranges from 3 per cent to 7 per cent.

4.2.7 Clayey Silt and Sand to Silty Sand Interlayers

Within the lower clayey silt till deposit, 0.9 m to 1.5 m thick interlayers of clayey silt were encountered at varying depths/elevations in Boreholes HN3, HN4 and HN5. Sand to silty sand interlayers, about 0.2 m to 0.5 m thick, were also encountered at varying intervals within the lower clayey silt till deposit in Boreholes HN2, HN4 and HN7.

The SPT "N"-values measured within the clayey silt interlayers range from 7 blows to 133 blows per 0.3 m of penetration, suggesting a firm to hard consistency. The SPT "N"-values measured within the sand to silty sand interlayers range from 19 blows to 116 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

Atterberg limits tests were carried out on four samples of the clayey silt interlayers and measured liquid limits between about 23 per cent and 34 per cent, plastic limits of about 13 per cent to 16 per cent and plasticity indices between about 10 per cent and 18 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure 9 in Appendix A and indicate that this material is a clayey silt of low plasticity.

The grain size distributions of three samples of the clayey silt interlayer are shown on Figure 10 in Appendix A.

The natural water content measured on samples of the clayey silt layer ranges from 14 per cent to 19 per cent.

4.3 Groundwater Conditions

The water level in the boreholes was observed during and upon completion of drilling operations between about Elevation 231.4 m and 242.8 m. A standpipe piezometer was installed in the lower till deposit in Borehole HN7 to permit monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the record for Borehole HN7, and the groundwater levels measured in the piezometer are summarised below.

Borehole No.	Ground Surface Elevation (m)	Stratum Sealed Into	Piezometer Tip Elevation (m)	Groundwater Elevation (m)	Date of Measurement
HN7	248.0	Lower Clayey Silt Till	228.5	246.1 239.8	November 25, 2010 December 2, 2010

Based on observations of moisture content and colour changes from brown to grey, it is considered that the water level at the site typically varies between about Elevation 242 m and 243 m, which is above the Highway 400 cut grade at this site. The groundwater level in this area will be subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. T. Veronica Ayetan, P.Eng., a geotechnical engineer, and reviewed by Ms. Lisa C. Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
HIGHWAY 9 UNDERPASS
HIGHWAY 400 WIDENING FROM NORTH OF KING ROAD TO SOUTH CANAL ROAD
GWP 2835-02-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATION

6.1 General

This section of the report provides foundations engineering recommendations for the detail design of the Highway 9 underpass replacement as part of Highway 400 widening from north of King Road northerly to South Canal Road. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the structure foundations and approach embankments. Where comments are made on construction, they are provided to highlight those aspects which could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing Highway 9 underpass consists of a two-span twinned bridge with span lengths of 17 m each. The existing abutments and piers are supported on spread footings (shallow foundations). It is understood that the new underpass structure is proposed to consist of a two-span pre-cast girder bridge with each span about 40.5 m long as a result of the proposed widening of Highway 400. In addition, the replacement structure is to be widened to the south by about 6.0 m.

Based on the General Arrangement (GA) drawing provided by URS on November 17, 2010, the proposed Highway 9 bridge deck varies between about Elevation 248.8 m and 248.6 m. The existing Highway 9 pavement surface near the proposed west and east abutments is at about Elevation 248.2 m and 248.0 m; therefore the proposed grade of Highway 9 will be raised slightly by approximately 0.8 m). The Highway 400 pavement beneath the existing Highway 9 structure is at about Elevation 240 m, and the ground surface at the toe of the existing west and east embankments is at about Elevation 239 m. The existing and proposed Highway 400 cut slopes are typically approximately 7 m to 8 m high, and the existing and proposed Highway 9 embankment is typically about 1 m to 2 m high.

6.2 Foundation Options

The new underpass structure will consist of two spans with approximate lengths of 40.5 m each. Within the vicinity of the foundation elements, the subsurface soil conditions consist of surficial fill material underlain by upper and lower till deposits (comprised of very stiff to hard clayey silt/clayey silt till and dense to very dense sand to silt till), which are separated by a very stiff to hard clayey silt layer.

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

- **Strip or spread footings founded within the very stiff to hard clayey silt/clayey silt till or very dense silt till in a "closed-end" structure configuration:** Spread footings are considered feasible and suitable to support the new abutments and central pier given the competency of the native soils at this site and the relative cost of construction; this option would also allow for the use of semi-integral abutments. However, for a “closed-end” bridge structure configuration, abutment spread footings founded on the native soil at lower elevation would require excavation of approximately 6 m to 9 m to reach the Highway 400 cut grade, which may not be practical or economical. Depending on construction staging, temporary roadway protection would likely be required at the abutments and at the centre pier.

- **Strip or spread footing “perched” above the Highway 400 cut grade on the very stiff to hard or dense to very dense native soils:** For a longer “open” structure configuration with 2H:1V abutment foreslopes, the abutment spread footings may be founded within the very stiff to hard or dense to very dense native soils at higher elevations above the Highway 400 cut grade, to reduce the extent of excavation as compared with a “closed-end” structure configuration.

- **Steel H-piles driven to found within the “100-blow” lower clayey silt till deposit:** Steel 310 x 110 H-piles driven to within “100-blow” material are suitable and feasible for the support of the proposed abutments and central pier, and would allow for integral abutment construction. However there is some risk associated with penetrating through or the piles hanging up within the till deposit as a result of occasional “100-blow” material encountered at higher elevation across the site. Furthermore the varying depth to “100-blow” soil within the footprint of each foundation element will result in the potential for variable pile lengths, which will need to be accommodated in the contract documents.

- **Steel tube (pipe) piles to found within the “100-blow” lower clayey silt till deposit:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments and central pier, however, MTO does not allow the use of pipe piles for integral abutment construction. Pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the glacially-derived soils at this site.

- **Caissons founded within the “100-blow” lower clayey silt till deposit:** Consideration could be given to the use of caissons socketted into the hard clayey silt till for support of the new abutments and central pier. However, if deep foundations are required, from a foundations perspective the use of driven piles would be preferred over caissons due to the presence of water-bearing cohesionless soils (i.e., the sand and silt to silt till and the interlayers or lenses of sand to silt within the clayey silt till). Temporary or permanent liners would be required during caisson installation to control the ground and groundwater within these water-bearing cohesionless zones, which would result in the caisson foundations being less cost-effective than the installation of driven steel H-piles.

At the abutments, spread footings founded above the Highway 400 cut grade are the preferred foundation option but will only permit semi-integral abutment design. However, if the integral abutment option is considered for the proposed structure, the abutment should be supported on steel H-piles. At the central pier, spread footings are preferred if the geotechnical axial resistance available is considered adequate by the structural engineer, otherwise, deep foundations will be required to achieve a higher capacity.

Recommendations for the various foundation options for the abutments and central pier discussed above for the Highway 9 underpass structure are provided in the following sections.

6.2.1 Strip or Spread Footings

6.2.1.1 Founding Elevations

Strip or spread footings can be founded on the very stiff to hard clayey silt till or very dense silt till in either a closed-end structure configuration, in which the abutment footings would be founded below the Highway 400 cut grade, or an open structure configuration, in which the abutment footings would be "perched" within the native soil deposits above the Highway 400 cut grade. For both options, the centre pier foundations would be supported on the very stiff to hard clayey silt till. The proposed finished grade of Highway 400 in this area is between about Elevation 239 m and Elevation 240 m, and the proposed finished grade for Highway 9 is at approximately Elevation 248.8 m and 248.6 m near the abutments.

All footings should be founded at a minimum depth of 1.5 m below the adjacent final surface grade to provide adequate protection against frost penetration, in accordance with Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario).* Where the footings are "perched" above the Highway 400 cut grade in an open structure configuration, the required thickness of soil cover for frost protection is measured perpendicular from the face of the abutment foreslope to the edge of the underside of the footing (i.e., it is not simply a vertical dimension when the footing is adjacent to a slope). If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation shall be installed to compensate for the lack of soil cover. As a guide, the MTO has adopted an equivalency of 25 mm of rigid polystyrene foam insulation for every 0.3 m reduction in soil cover.

The following summarises the recommended maximum founding elevations for strip or spread footing for support of the centre pier and abutments in both "closed-end" and "open" structure configurations.

Foundation Element	Reference Borehole No.	Closed-end Structure Configuration (Lower Founding Elevation)		Open Structure Configuration (Higher Founding Elevation)	
		Founding Stratum	Maximum Founding Elevation	Founding Stratum	Maximum Founding Elevation
West Abutment	HN2, HN3 and 97-6	Hard clayey silt till	237.5 m	Hard clayey silt	244.0 m
Centre Pier	HN4, HN5, 97-2 and 97-3	Very stiff to hard clayey silt till	237.0 m	Hard clayey silt till	237.0 m
East Abutment	HN6, HN7 and 97-1	Hard clayey silt / Very dense silt till	237.5 m	Hard clayey silt till	243.5 m

For a "closed-end" structure configuration at the east abutment, as summarized in the above table, the northern portion of the footing subgrade would consist of hard clayey silt till, and the southern portion of the footing subgrade would consist of very dense silt till. The silt till will be water-bearing and groundwater control would be required to minimize disturbance to the silt subgrade. Due to its fine-grained nature, the silt till may be difficult to "dewater", and it may be more practical to sub-excavate the wet silt and replace it with compacted Granular 'A' or Granular 'B' Type II (SP 110S13 – Aggregates).

6.2.1.2 Geotechnical Resistances

The following factored axial geotechnical resistances at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) may be used for the design of a 4 m wide spread footing placed on the properly prepared, undisturbed native soil subgrade at or below the founding elevations provided in the preceding section.

	Closed-end Structure Configuration (Lower Founding Elevation)		Open Structure Configuration (Higher Founding Elevation)	
Foundation Element	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**	**Factored Geotechnical Resistance at ULS**	**Geotechnical Reaction at SLS (for 25 mm of Settlement)**
West Abutment	600 kPa	450 kPa	675 kPa	350 kPa
Centre Pier	500 kPa	375 kPa	500 kPa	375 kPa
East Abutment	600 kPa	450 kPa	675 kPa	350 kPa

These design values take into account the depth of footing embedment (the depth of the footing relative to the proposed adjacent grade) for the closed-end structure configuration, and assumes a minimum depth of embedment of 1.5 m and the presence of the 2H:1V abutment foreslope for the open structure configuration. The geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above.

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

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

6.2.1.3 Resistance to Lateral Loads

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

6.2.2 Driven Steel H-Piles or Tube Piles

Steel H-piles or steel tube (pipe) piles driven to found within the "100-blow" lower clayey silt till may be used for support of the abutments and the centre pier.

For the installation of the steel H-piles or steel tube piles, it is noted that in some of the boreholes advanced at the foundation elements, equivalent "100-blow" material was occasionally encountered higher than the proposed founding tip elevations. Also, consideration must be given on the potential presence of cobbles and boulders within the till deposits at the site. In this regard, steel H-piles are preferred over steel tube piles as tube piles are considered to pose a higher risk of "hanging-up" or being deflected away from their vertical or battered orientation during installation, due to their larger end area. It is recommended that the piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the pile. The steel H-piles should be reinforced with flange plates as per OPSD 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) for protection during driving as per OPSS 903 (Deep Foundations). Similarly, if steel tube piles are being considered, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe). The requirement for driving shoes should be included in the Contract Drawings.

6.2.2.1 Pile Founding Elevation

It is noted that during the preliminary foundation investigation carried out by Thurber Engineering Ltd. (Thurber) for this site, the soil samples were obtained using a manually-operated safety hammer (i.e. rope cathead), whereas during the current foundation investigation, the drill rig was equipped with an automated hammer with higher efficiency. In assessing the founding elevations for deep foundation options, the Standard Penetration Test (SPT) "N"-values from the current foundation investigation have been corrected to 60 per cent efficiency of hammer energy transfer.

The surface of the "equivalent "100-blow" lower clayey silt till was encountered at varying elevations across the site and within each foundation element. For design, the following range of pile tip elevations may be used based on the borehole results, assuming approximately 1.5 m to 2 m of penetration into materials having equivalent SPT "N" values of greater than 100 blows per 0.3 m of penetration.

Foundation Element	Reference Borehole Nos.	Founding Stratum	Estimated Pile Tip Elevation
West Abutment	HN2, HN3 and 97-6	Hard Clayey Silt Till	229.5 m to 226.0 m
Centre Pier	HN4, HN5, 97-2 and 97-3	Hard Clayey Silt Till	230.0 m to 228.0 m
East Abutment	HN6, HN7 and 97-1	Hard Clayey Silt Till	229.5 m to 226.0 m

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

6.2.2.2 Geotechnical Axial Resistances

For steel HP 310 x 110 piles driven to the estimated pile tip elevations provided above, the factored axial geotechnical resistance at ULS and the geotechnical reaction at SLS for 25 mm of settlement are given below.

Foundation Element	Founding Stratum	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS (for 25 mm of Settlement)
West Abutment	Hard Clayey Silt Till	1,500 kN	1,250 kN
Center Pier	Hard Clayey Silt Till	1,350 kN	1,150 kN
East Abutment	Hard Clayey Silt Till	1,500 kN	1,250 kN

Similar axial resistances may be used in the design for closed-end, concrete filled 324 mm (12 ¾ in.) diameter steel tube piles having a minimum wall thickness of 6.3 mm (¼ in.).

Given the very stiff to hard/dense to very dense nature of the overburden soils and the limited approach embankment loading, the magnitude of differential settlement in the area of the abutment piles will be negligible and therefore downdrag loads do not need to be taken into account in the pile design.

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 size and length of pile. The set criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO Standard Drawing SS103-11) during the final stages of driving to achieve an 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 3,000 kN per pile at the abutments and 2,700 kN per pile at the centre pier, but should be driven to no higher than 1.5 m above the design pile tip elevations shown below at each foundation element:
  - West Abutment – Elevation 229.5 m to 226.0 m
  - Centre Pier – Elevation 230.0 m to 228.0 m
  - East Abutment – Elevation 229.5 m to 226.0 m

Assessment of ultimate geotechnical resistance by the Hiley formula should commence once the pile reaches a depth of not more than 1.5 m above the design pile tip elevation 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.

Given the variable depth to the "100-blow" soils and the resulting variability in the pile founding elevations, it is recommended that the greater pile lengths be stipulated in the Contract Drawings for piles located between the north and south sides of the abutments to ensure that adequate pile lengths are available on site and to reduce splicing needs. It is also recommended that the axial capacity be calculated by the Hiley formula on every pile installed.

6.2.2.3 Resistance to Lateral Loads

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 (CFEM 1992 as noted in Section 6.8.7.1 of the *Commentary* to the *CHBDC, 2006*):

For cohesionless soils:

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

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) at any point along the pile; 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)
West Abutment	Very stiff to hard clayey silt till	247.2 to 244.3	-	150
	Hard clayey silt	244.7 to 239.4	-	200
	Hard clayey silt till	239.4 to 227.8	-	200
Center Pier	Stiff to hard clayey silt till	239.2 to 226.2	-	150
East Abutment	Stiff to hard clayey silt till	247.4 to 245.8	-	150
	Very stiff clayey silt	246.5 to 243.5	-	150
	Very stiff to hard clayey silt till	245.0 to 227.8	-	200
	Very dense sand and silt to silt till	239.4 to 237.4	11	-

For design of a single vertical HP310x110 pile embedded in hard clayey silt till driven to the highest design pile tip elevation given in Section 6.2.2.1 as specified above, a maximum factored lateral geotechnical resistance at ULS of 260 kN and a lateral geotechnical resistance at SLS of 200 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*. These values can be employed for piles supporting integral abutments below CSP filled with loose sand.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2, 1982) as follows:

Pile Spacing in direction of loading (D = Pile Diameter)	Subgrade Reaction Reduction Factor (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.

6.2.2.4 Frost Protection

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

6.2.3 Caissons

Caissons socketted into the “100-blow” lower clayey silt till could be considered for support of the abutments and centre pier, particularly if higher geotechnical resistances are required than can be obtained for driven steel H-pile foundations.

If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner would be required to support the soils during construction, to minimize disturbance and loss of ground in the water-bearing cohesionless soil zones (the sand to silt till and interlayers or lenses or sand to silt within the clayey silt till). If there is water infiltration such that there is standing water within the caisson excavation prior to concrete placement, the concrete must be placed using tremie techniques. After initial placement of concrete at the bottom of the caisson, the tremie discharge point should be maintained a minimum of 1 m below the surface of the wet concrete during placement. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to address the need for control of the ground and groundwater during caisson construction as discussed further under Construction Considerations in Section 6.6.

It is expected that the liner would be installed (and removed, if a temporary liner is used) using a vibratory hammer. In this case, vibration monitoring is recommended during liner installation and removal. The liner must be maintained tight to the sides of the bore to minimize seepage of water.

The performance of caissons will depend upon the final cleaning and verification of the subgrade quality (hard lower clayey silt till) 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 (2011) outlines appropriate safety procedures and requirements that must be implemented prior to entry of personnel into the caissons for inspection of the base or alternatively, the inspections may be carried out remotely using visual recording equipment.

6.2.3.1 Founding Elevation

The following caisson base elevations and strata may be used in the design, based on the lowest elevation within each foundation element to achieve at least 1.5 m of penetration into the "100-blow" lower clayey silt till soils:

Foundation Element	Boreholes No.	Founding Stratum	Estimated Caisson Founding Elevation
West Abutment	HN2, HN3 and 97-6	Hard clayey silt till	227.0 m
Central Pier	HN4, HN5, 97-2 and 97-3	Hard clayey silt till	228.0 m
East Abutment	HN6, HN7 and 97-1	Hard clayey silt till	227.0 m

6.2.3.2 Geotechnical Axial Resistances

The following provides the recommended factored geotechnical axial resistance at ULS and geotechnical reaction at SLS (for 25 mm of settlement) for caissons founded within the hard clayey silt till and socketted 2 m within the "100-blow" material.

Foundation Element	Caisson Diameter	Founding Stratum	Factored Geotechnical Resistance at Ultimate Limit States (ULS)	Geotechnical Reaction at Serviceability Limit States (SLS) for 25 mm of Settlement
West and East Abutments	0.9 m	Hard clayey silt till	2,400 kN	2,000 kN
	1.2 m		4,300 kN	3,600 kN
	1.5 m		6,500 kN	5,500 kN
Centre Pier	0.9 m	Hard clayey silt till	2,000 kN	1,600 kN
	1.2 m		3,500 kN	2,800 kN
	1.5 m		5,500 kN	4,400 kN

Given the very stiff to hard/dense to very dense nature of the overburden soils and the limited approach embankment loading, the magnitude of differential settlements in the area of the abutment piles will be negligible and therefore downdrag loads do not need to be taken into account in the caisson design.

6.2.3.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons (based on subgrade reaction theory), and the reductions due to group effects, may be determined as per Section 6.2.2.3.

At the foundation elements, the maximum factored lateral resistances at ULS of 260 kN and maximum lateral resistances at SLS of 200 kN (for 10 mm of horizontal deflection at pile cap level) are recommended for 0.9 m diameter caissons, based on Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC*. Values for alternative caisson diameters can be developed if larger diameter caisson foundations are adopted for support of foundation elements at this site.

6.2.3.4 Frost protection

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

6.3 Lateral Earth Pressures for Design

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

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

- Select, free draining granular fill in accordance with SP 110S13 Granular ‘A’ or Granular ‘B’ Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSD 3101.150 – *Wall, Abutments Backfill* and OPSD 3121.150 – *Walls Retaining, Backfill.*

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS 501. Other surcharge loadings should be accounted for in the design as required.

- For restrained structures, the granular fill may be placed either in a zone with the width equal to at least 1.4 m behind the back of the walls (see Case A in Figure C6.20 (a) of the *Commentary* to the *CHBDC*). For unrestrained structures, the granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Case B in Figure C6.20(b) of the *Commentary* to the *CHBDC*).

- For restrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of earth fill :

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

- For unrestrained structures, where the pressures are based on SP 110S13 granular fill behind the wall, the following parameters (unfactored) may be assumed:

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

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 (such as for a rigid frame structure), at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

A restrained structure is typically a concrete box culvert or a rigid frame bridge 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.

6.3.1 Seismic Considerations

6.3.1.1 Site Coefficient

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

6.3.1.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading may also need to be considered for the design of abutment stems/retaining walls and for the assessment of liquefaction potential of foundation soils in accordance with Section 4.6 of the *CHBDC,* as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. At this site, the requirements for seismic analysis are outlined as follows:

According to Table A3.1.1 of the *CHBDC*, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio for Aurora-Newmarket is 0.05. Based on experience, for the subsurface conditions at this site, a 20 per cent amplification of the ground motion may occur (i.e. Site Coefficient, S=1.2 for Soil Profile II from Table 4.4 of *CHBDC*), resulting in an increase in the peak horizontal ground acceleration (PHA) from 0.05 g to 0.06 g at the ground surface. Based on Section 4.4.4 of the *CHBDC*, this bridge structure is assigned Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the *CHBDC*, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.4 Retained Soil System (RSS) Walls

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

6.4.1 Founding Elevations

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. At its lowest point, the facing footing should be founded at or below Elevation 239.0 m to extend below the existing fill in this area. As the RSS wall is proposed to “step up” into cut slope, the facing footing may also be stepped up provided that it is founded below any topsoil or softened/disturbed soil; for design, a minimum founding depth of 0.8 m is recommended as the facing footing steps up into the cut slope.

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

The compacted Granular ‘A’ pad and the reinforced soil mass should be keyed into the existing embankment by benching into the embankment fill, as per OPSD 208.010 (Benching of Earth Slopes).

6.4.2 Global Stability

The static and seismic global slope stability of RSS walls adjacent to the Highway 9 underpass structure has been analyzed using the commercially-available program SLIDE, produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety. A target factor of safety of 1.3 against deep-seated global instability of the RSS walls is normally adopted by MTO for design under static conditions; under seismic conditions, a target Factor of Safety of 1.1 is used. These factors of safety are considered appropriate for the RSS walls at this site, considering the design requirements and the field data available.

The soil parameters used in the analysis, as given below, were estimated from empirical correlations using the results of in-situ Standard Penetration Tests (SPTs) (Bowles, 1984) and geotechnical classification testing. The groundwater table was taken at Elevation 242.0 m in the analyses.

Soil Type	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Angle of Internal Friction, φ' (degrees)
Existing Embankment Fill/Native Soil	21	--	35
Very Dense Sand and Silt Till to Silt Till	21	--	35
Very Dense Silty Sand	21	--	34
Hard Clayey Silt Till	22	200	32

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

Given the required RSS wall height(s), the minimum reinforced width of RSS wall required to obtain a factor of safety equal to 1.3 or greater against deep-seated global instability has been calculated. The ratio of minimum reinforced mass width to reinforced wall height for three RSS wall heights is provided below. The result of the analysis for the RSS wall adjacent to the abutment wall (a 9.8 m high wall) is shown on Figure 1 for the static condition.

RSS Wall Height	Ratio of Minimum Reinforced Mass Width to Wall Height
9.8 m	0.8
4.9 m	1.0
2.5 m	1.7

The above ratios for walls with a height of approximately 4.9 m or less are greater than the "typical" ratios that are used by wall designers (i.e. approximately 0.7 to 0.8 times the wall height), because of the presence of the 2H:1V slope in front of the wall. The contract drawings will need to specify the width of the reinforced soil mass.

Under seismic loading conditions, using a seismic coefficient of 50 per cent of the site-specific design peak horizontal ground acceleration (PHA) equal to 0.03g, the Factor of Safety is greater than 1.1. The result of an example seismic slope stability analysis for the reinforced mass width to wall height adjacent to the abutment wall is shown on Figure 2.

6.4.3 Geotechnical Resistances

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

Wall Height	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
9.8 m	500 kPa	400 kPa
4.9 m	300 kPa	250 kPa
2.5 m	275 kPa	225 kPa

6.4.4 Resistance to Lateral Loads

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

6.5 Approach Embankments and Cut Slopes

The natural ground surface at the site varies from approximately Elevation 244.5 m to 247.5 m. Highway 400 and the interchange ramps have been constructed in a cut, with the existing Highway 400 grade in the general area of the underpass varying between about Elevation 239.5 m and 240.3 m; this 5 m to 7 m deep cut will be widened by approximately 20 m to 25 m toward the west and east to accommodate the proposed widening of Highway 400. The current Highway 9 pavement grade behind the proposed new west and east abutment locations is at about Elevation 248.2 m and 248.0 m, and the proposed grade following the highway widening and construction of the two-span replacement structure is between approximately Elevation 248.8 m and 248.6 m. A slight grade increase of about 0.8 m is planned for the existing Highway 9 embankment, and it is to be widened by approximately 6 m toward the south, requiring placement of approximately 1 m up to a maximum of about 3 m of new fill above the current natural ground surface and south embankment slope face.

6.5.1 Subgrade Preparation and Embankment Construction

The existing Highway 9 embankment, which consists of layers of stiff to hard cohesive fill and compact to very dense cohesionless fill, is considered to be appropriate for incorporation into the widened Highway 9 approach embankments. However, to improve the performance of the widened embankment as related to reducing the potential for post-construction settlement, it is recommended that prior to the placement of the additional fill, all topsoil, organic matter and soft/loose fill should be stripped from below the approach embankment areas. Embankment fill should be placed and compacted in accordance with SP 206S03 (Excavation and Grading), OPSS 501 (Compacting) and SP 105S21 (Amendment to OPSS 501).

In accordance with MTO's standard practice, to minimize surficial erosion, a minimum 2 m wide bench should be provided where embankment slopes are greater than 8 m in height or where cut slopes are greater than 6 m in depth, consistent with OPSD 202.010 (Slope Flattening). To reduce the potential for erosion of the embankment side slopes due to surface water run-off, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS 572 (relocated to OPSS 804) (*Seed and Cover)*.

6.5.2 Approach Embankment and Cut Slope Stability

Static and seismic slope stability analyses of the proposed widened Highway 9 approach embankments and widened Highway 400 cut slopes were carried out using the commercially available program Slide (produced by Rocscience Inc.) to check that the target minimum factor of safety was achieved for the proposed embankment and cut slope heights and geometries. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally used in the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments and cut slopes at this site.

The soil parameters used in the analysis, as given below, were estimated from empirical correlations suggested by proposed by Kulhawy and Mayne (1990) using the results of in-situ Standard Penetration Tests (SPT) and geotechnical classification testing. For the purpose of analysis, earth fill or granular fill has been considered for the construction of the widened approach embankments. The groundwater table in the analyses was taken to be at Elevation 242.0 m, declining to below Elevation 239 m below the Highway 400 lanes.

Approach Embankment	Soil Type	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Angle of Internal Friction, φ' (degrees)
West Approach (Boreholes HN2 and HN3)	New embankment fill	21	--	35
	Existing fill	20	--	30
	Very stiff to hard clayey silt till (upper deposit)	21	150	32
	Hard clayey silt	21	200	32
	Hard clayey silt till (lower deposit)	22	200	32
	Very dense sand/silty sand interlayers	21	--	35
East Approach (Boreholes HN6 and HN7)	New embankment fill	21	--	35
	Existing fill	20	--	30
	Stiff to hard clayey silt till (upper deposit)	21	100	30
	Very stiff to hard clayey silt	21	150	32
	Hard clayey silt till (lower deposit)	22	200	32
	Very dense sand and silt to silt till	21	--	35

Assuming appropriate subgrade preparation and proper placement and compaction of fill for the Highway 9 embankment widening, the total slope height of approximately 9.8 m (representing a 6 m to 7 m deep cut slope plus 1 m to 3 m of embankment fill) maintained at 2H:1V will have a Factor of Safety of greater than 1.3 against deep-seated slope instability. A simplified representation of the Highway 400 cut slope and the Highway 9 embankment is shown on Figure 3.

Under seismic loading conditions with yield peak horizontal ground acceleration (PHA) equal to 0.03 g, the Factor of Safety is greater than 1.1 as shown on Figure 4.

6.5.3 Approach Embankment Settlement

Settlement of the widened Highway 9 approach embankments at the site will occur due to compression of the new embankment fill, as well as compression of the existing embankment fill and underlying native soils due to the widened embankment load. The compression for the Highway 9 approach embankments was modelled by estimating an elastic modulus of deformation based on the SPT "N"-values and correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). The values of the parameters given are based on the soil conditions encountered in Boreholes HN2 and HN3 drilled at the location of the proposed west abutment as this area contains the hard clayey silt deposit of up to about 4.8 m thick encountered beneath the existing fill embankment. The groundwater table in the analyses was taken to be Elevation 242.0 m.

Soil Deposit	Bulk Unit Weight	Estimated Deformation Properties
Hard Clayey Silt	21 kN/m^3	$m_v = 1 \times 10^{-5}$ kPa^{-1}
Hard Clayey Silt Till	22 kN/m^3	E = 75 MPa
Very Dense Silty Sand Interlayers	21 kN/m^3	E = 75 MPa
Very Dense Sand	21 kN/m^3	E = 100 MPa

The results of the analyses indicate a total settlement of less than 10 mm below the west and east approach embankments for the southward widening of the existing Highway 9 embankment, with an estimated maximum fill placement height of up to approximately 3 m. This settlement is expected to occur rapidly (i.e. during or shortly after construction) in response to filling based on very stiff to hard/dense to very dense nature of the subsoils at the site.

6.5.3.1 Settlement of Embankment Fill

A maximum thickness of about 3 m of additional fill will be required as part of the southward widening of Highway 9. Provided that the new fill is comprised of suitable earth or granular fill meeting the requirements of and placed and compacted in accordance with SP 206S03, the settlement of the additional fill itself is expected to be less than about 10 mm, and this settlement is expected to occur relatively quickly, during and immediately following construction.

6.6 Construction Considerations

6.6.1 Open-Cut Excavation

The foundation excavations at the abutments for spread footings or pile cap construction will extend through existing fill and into the till deposits, which contain zones, interlayers and lenses of water-bearing cohesionless soil. Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill materials are classified as Type 3 soil and the till is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

6.6.2 Temporary Roadway Protection

It is expected that temporary excavation support will be required to maintain traffic lanes in operation along Highway 9 during construction of the new abutments and retaining walls. The temporary excavation support systems should be designed and constructed in accordance with OPSS 539 *(Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided that any adjacent utilities can tolerate this magnitude of deformation.

The protection system is expected to be required for a maximum excavation depth of approximately 8 m (i.e., the difference in elevation between the Highway 400 and Highway 9 grades). It is considered that 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. It would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards where cohesionless soils are encountered below approximately Elevation 242 m to 243 m.

The soldier piles would have to be socketted to sufficient depth to provide the necessary passive resistance for the retained soil height of up to about 6 m. Lateral support to the soldier piles could be provided in the form of rakers or temporary anchors. The selection and design of the protection system will be the responsibility of the Contractor.

6.6.3 Groundwater Control

The groundwater level measured in the standpipe piezometers installed in the clayey silt/clayey silt till deposits in Boreholes 97-1, 97-6 and HN7 at the site varies between about 5 m and 8 m below the Highway 9 grade, corresponding to about Elevation 242.7 m to 240 m. Based on the water level measurements and observations of soil colour changes from brown to grey, it is expected that the stabilized groundwater level at the site is between approximately Elevation 242 m and 243 m. Therefore, it is expected that Highway 9 cut itself as well as excavations for spread footings for a “closed-end” structure configuration or for a pile cap would extend below the groundwater level. It is anticipated that water inflow from interlayers or lenses of cohesionless soil within the clayey silt till can be handled by pumping from filtered sump pumps placed at the base of the excavation; a dewatering system may be required for zones of water-bearing sand to silt till. It is recommended that an NSSP be included in the Contract Documents to warn the contractor of the groundwater conditions at this site; an example NSSP is presented in Appendix C.

6.6.4 Subgrade Protection

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

6.6.5 Ground and Groundwater Control for Caissons Installation

As discussed in Section 6.2.3, running or flowing of water-bearing cohesionless soils (the sand to silt till or sand to silt interlayers or lenses within the clayey silt till) could occur during or after drilling of the caissons. If caisson foundations are adopted for support of any of the foundation elements, temporary or permanent caisson liners would be required to support the soils during construction and permit inspection and cleaning of the caisson base. It is recommended that an NSSP be included in the Contract Documents to warn the contractor of these conditions and the need to control the ground and groundwater during caisson construction; an example NSSP is presented in Appendix C.

6.6.6 Obstructions During Pile Driving / Caisson Installation

It is anticipated that cobbles and/or boulders may be encountered within the till deposits, as noted in Borehole HN3 advanced at this site, which may affect the installation of steel H-piles and/or caissons. It is recommended that driving shoes be used on all steel H-piles to facilitate driving into the hard clayey silt till and very dense sand and silt till. In addition it is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils; an example NSSP is presented in Appendix C.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. T. Veronica Ayetan, P.Eng., a geotechnical engineer, and reviewed by Ms. Lisa C. Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.

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Ontario Provincial Standard Specifications (OPSS)

OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 572	(Relocated to OPSS804) Construction Specification for Seed and Cover
OPSS 903	Construction Specification for Deep Foundations
OPSS 1002	Material Specification for Aggregates - Concrete

Ontario Provincial Standard Drawings (OPSD)

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

Contract Design Estimating and Documentation (CDED)

SP 110S13	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
SP 206S03	Excavation and Grading; Excavation for Pavement Widening
SP 105S21	Amendment to OPSS 501
SP 599S22	Retained Soil System, Wall/Slope, High Performance

Occupational Health and Safety Act and Regulations, Construction Projects (O.Reg 213191), 2011.

TABLE 1

COMPARISON OF FOUNDATION ALTERNATIVES
HIGHWAY 9 UNDERPASS - HIGHWAY 400 WIDENING G.W.P. 2853-02-00

Option	Rank	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Strip or Spread Footing on very stiff to hard clayey silt / clayey silt till or very dense silt till	3	• Feasible for support of abutments and pier.	• Allows for semi-integral abutments; and • Negligible post-construction settlement.	• Up to about 10 m depth and 34 m long through the existing embankment fill and native soil would be required; resulting in traffic disruption during construction; • Traffic protection system required during construction; • Groundwater control (dewatering) required; • Lower bearing capacities compared to deep foundation options; and, • Precludes use of integral abutments; potentially greater maintenance required at abutments.	• Lower relative costs than deep foundations; and, • Additional cost for sub-excavation of existing embankment fill and native soil. (4 m wide x 35 m long x 1.8 m thick x 3 footings) @ \$ 600 / m^3 + (10 m deep x 12 m wide x 35 m long x 2 abutments) @ \$ 10 / m^3 + (2 m deep x 4 m wide x 35 m long x 1 pier) @ \$ 10 / m^3 $\simeq$ \$ 540,000	• Risk with control of groundwater at the east abutment due to presence of native silt till at the footing subgrade; and, • Potential traffic disruption during construction.
Strip or Spread Footing "perched" on hard clayey silt / clayey silt till	2	• Feasible for support of abutments.	• Negligible post-construction settlement; • Footing subgrade would not be disturbed by groundwater; and, • Reduce depth of existing embankment excavation compared to footings founded at lower founding elevation.	• Traffic protection system required during construction; • Lower bearing capacities compared to deep foundation options; • Longer bridge spans required; and, • Does not allow for integral abutment construction.	• Low cost option; and, • Relatively lower cost for excavation of existing embankment fill. (4 m wide x 35 m long x 1.8 m thick x 2 footings) @ \$ 600 / m^3 + (4 m deep x 6 m wide x 35 m long x 2 abutments) @ \$ 10 / m^3 $\simeq$ \$ 319,000 plus Granular 'A' Pad.	• Potential traffic disruption during construction.
Steel H-Piles driven within "100-blow" lower clayey silt till.	1	• Feasible for support of abutments and pier.	• Higher geotechnical axial resistance, compared to spread footings; • Negligible post-construction settlement; and, • Can be used for support of conventional or integral abutments.	• Requires 35 m long excavation for pile cap; • Traffic protection system required during construction; • Long piles may be required to reach "100-blow" materials; and, • Requirement for sand filter beneath the centre pier pile caps to reduce potential of migration of fines that may be carried along the piles due to high groundwater table present at the site.	• Higher cost than spread footings; and, • Installation costs could be impacted by presence of obstructions. Assume (36 piles x 8 m long) @ \$ 250 / m^3 $\simeq$ \$ 72,000 plus excavation and pile cap costs of about \$ 232,000.	• Potential traffic disruption during construction; • Negligible risk of post-construction settlement of underpass structure, or of differential settlement of foundation elements; • Risk of encountering obstructions that could impact pile installation; and, • Potentially less costly maintenance over life of the structure than semi-integral abutment structures.

Option	Rank	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel Tube Piles (closed-end, concrete filled) driven to found in "100-blow" lower clayey silt till.	4	• Feasible for support of abutments and pier.	• Higher geotechnical axial resistance, compared to spread footings; • Negligible post-construction settlement; and, • Can be used for support of conventional or integral abutments provided the pile size can accommodate the lateral resistance required for such abutment design.	• Requires sub-excavation for cap construction; • Traffic protection system required during construction; • Long piles may be required to reach "100-blow" materials; • Greater disturbances to immediately adjacent ground due to larger base area if end is closed; • Requires staged construction for driving, cleaning and concrete filling of tube; • Greater potential for crumpling if obstructions encountered; • Requirement for sand filter beneath the centre pier pile caps to reduce potential of migration of fines that may be carried along the piles due to high groundwater table present at the site; and, • MTO does not allow the use of pipe piles for integral abutment design.	• Higher cost than spread footings; • Cost for steel tube (pipe) piles slightly higher than for steel H-piles; and, • Installation costs could be impacted by presence of obstructions. Assume same cost as steel H-piles ~ $ 305,000.	• Potential traffic disruption during construction; • Negligible risk of post-construction settlement of underpass structure, or of differential settlement of foundation elements; and, • Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; resulting in piles "hanging up".
Caissons founded within "100-blow" lower clayey silt till.	5	• Feasible for support of abutments and pier.	• Higher geotechnical axial resistance compared to spread footings and piles; so reduced number of deep foundation elements compared to steel H- or tube piles. • Negligible post-construction settlement; and, • No excavation required for pile cap.	• Potential for blow-out of the caisson base due to the presence of the silt to sand and silt deposits under high hydrostatic head; • Caissons could encounter obstructions (cobbles and boulders) during installation; • Need for temporary or permanent liners; • Cleaning of the base below the water table could be difficult; • Potential requirement for placement of concrete by tremie method; • Traffic protection system required during construction; • Not suitable for integral abutment design for the standard MTO tube size; and, • Greater risk of encountering obstructions due to larger size of drill hole required.	• Higher cost than steel H-piles; and, • Installation cost could be impacted by need for liner to minimize disturbance and loss of ground andd for tremie concrete placement. Assume (9 caissons / element x 8 m long x 3) @ $ 2,000 / m^3 ~ $ 432,000.	• Risk of disturbance of water-bearing sand and silt till soils, requiring special construction procedures including use of temporary or permanent liners; • Significant traffic disruption during construction due to space required for caisson drilling equipment; • Negligible risk of post-construction settlement of overpass structure, or of differential settlement of foundation elements; and, • Risk of encountering obstructions that could impact caisson installation/costs.

Prepared By: TVA

Reviewed By: LCC/JMAC

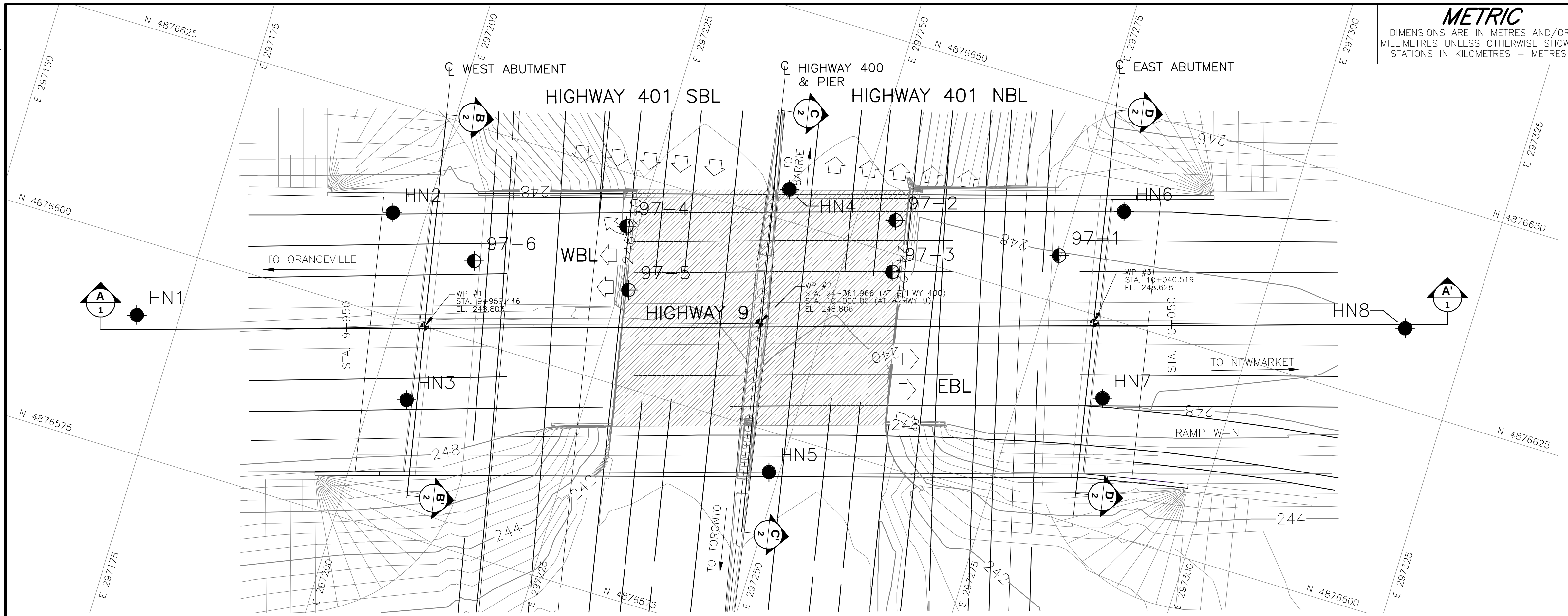
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2835-02-00

SHEET

HIGHWAY 400
HIGHWAY 9 UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

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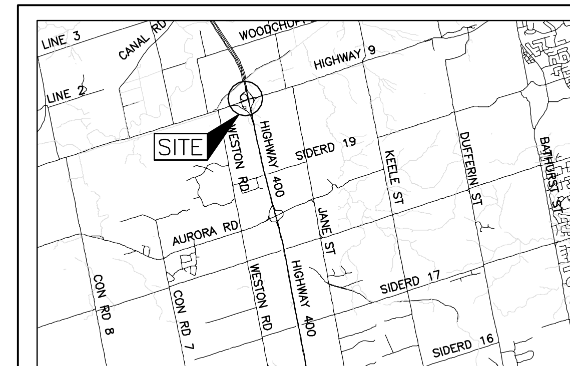


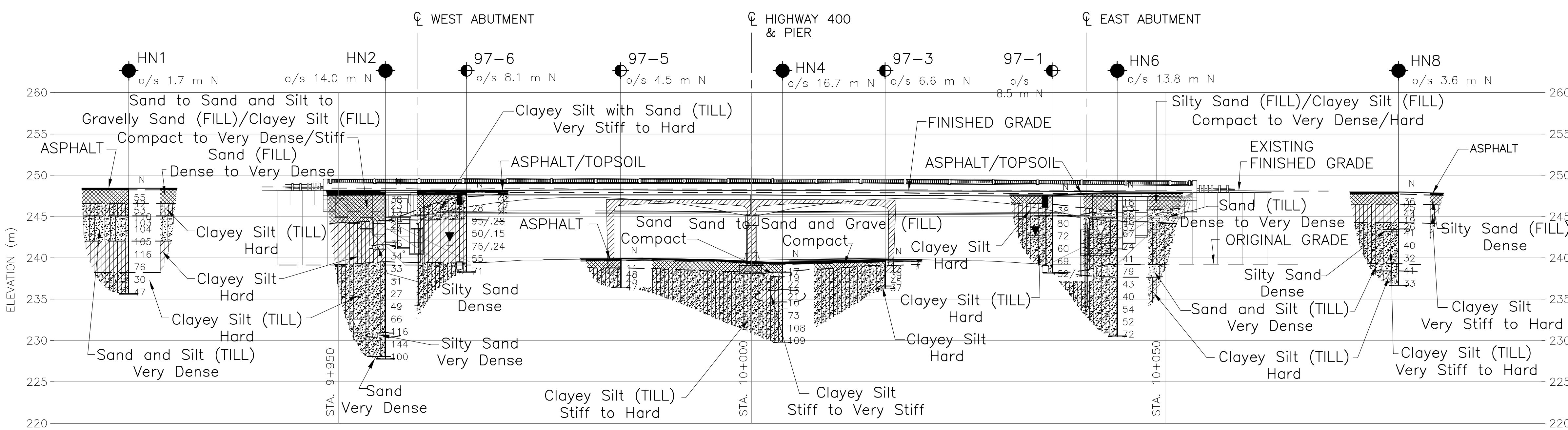
KEY PLAN
SCALE
2 0 2 km

LEGEND

- Borehole - Current Investigation
- Borehole - Thurber Engineering Ltd. Report (WP 3-95-01, Site No. 73-33), dated May 8, 1997.
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on Dec. 02, 2010
- WL upon completion of or during drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
97-1	247.6	4876630.6	297274.8
97-2	239.7	4876628.9	297254.6
97-3	239.8	4876622.8	297256.0
97-4	239.8	4876618.8	297223.6
97-5	239.9	4876611.4	297226.1
97-6	247.9	4876609.4	297207.2
HN1	248.4	4876591.2	297170.0
HN2	248.2	4876612.1	297196.1
HN3	248.2	4876590.9	297204.2
HN4	239.5	4876628.8	297241.2
HN5	240.3	4876595.3	297248.8
HN6	247.9	4876638.0	297280.8
HN7	248.0	4876615.6	297284.9
HN8	248.0	4876634.4	297317.5



A-A' 1 CENTRELINE PROFILE

SCALE
6 0 6 12 m

LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA Nov. 22, 2012 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Nov. 22, 2012 PROVINCE OF ONTARIO

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by URS, drawing file no. Highway 9 Underpass GA.dwg, received November 17, 2010.

NO.	DATE	BY	REVISION

Geocres No. 31D-551			
HWY. 400	PROJECT NO. 09-1111-0018		DIST.
SUBM'D. TVA	CHKD. TVA	DATE: 11/22/2012	SITE:
DRAWN: JFC	CHKD. LCC	APPD. JMAC	DWG. 1

PLOT DATE: November 22, 2012
FILENAME: T:\Projects\2009\09-1111-0018 (URS, York Region)\--BB-- (HIGHWAY 9)\0911110018BB001.dwg

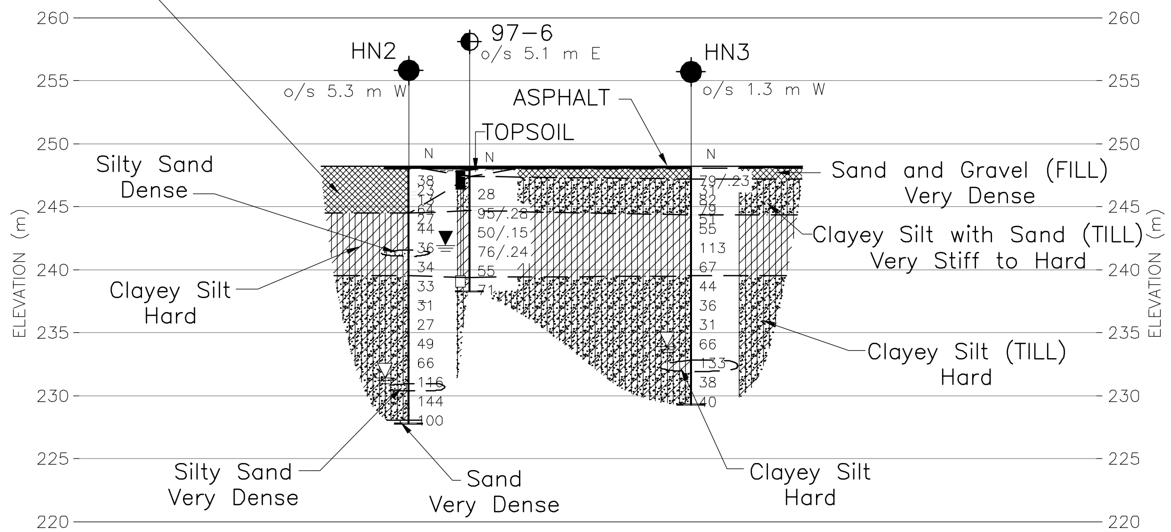
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2835-02-00

HIGHWAY 400
HIGHWAY 9 UNDERPASS
SOIL STRATA

SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



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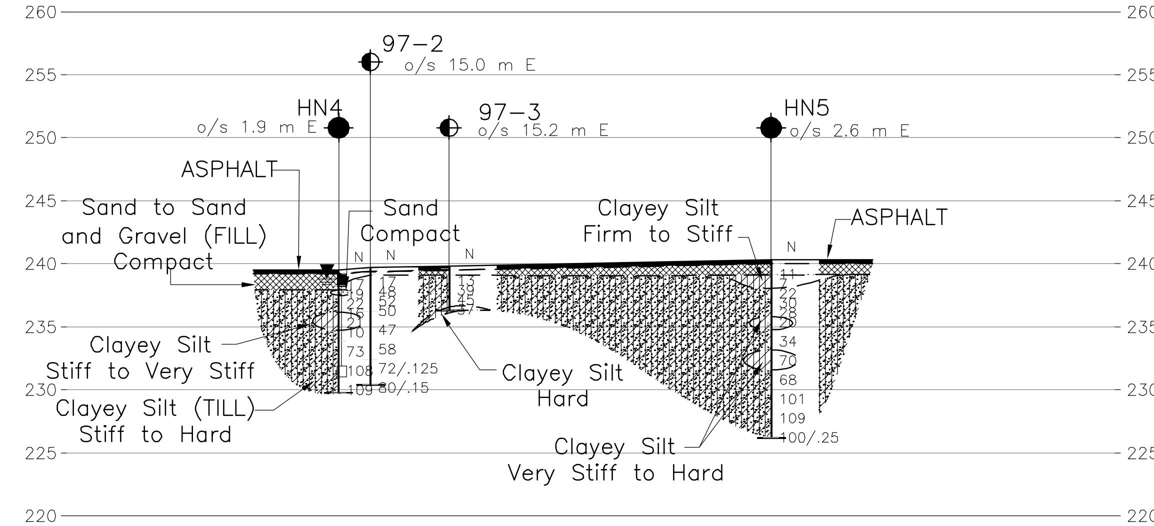

C-C' 1 CENTRE PIER
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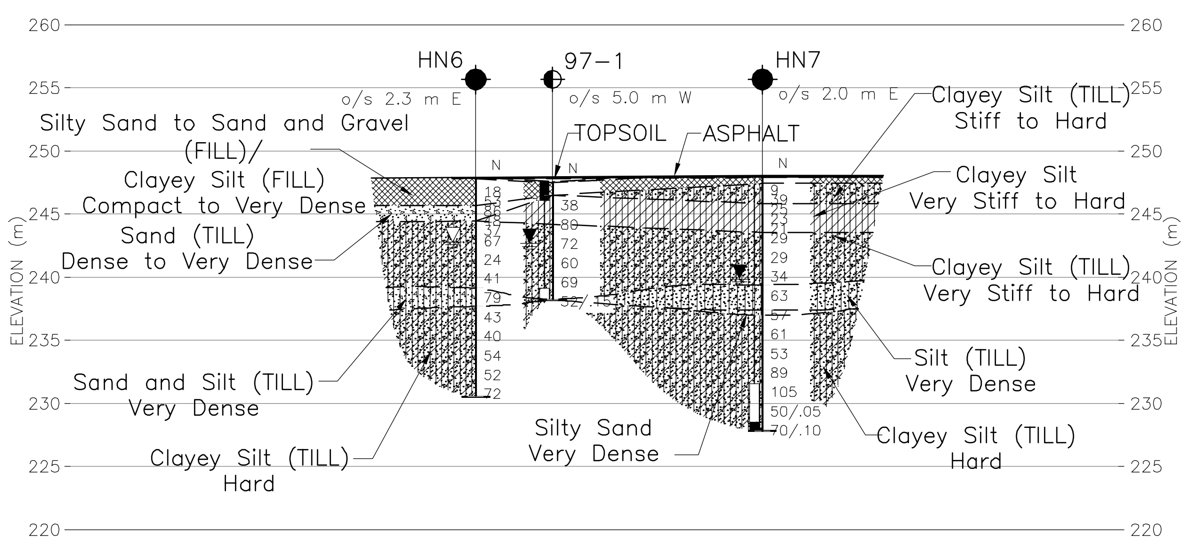
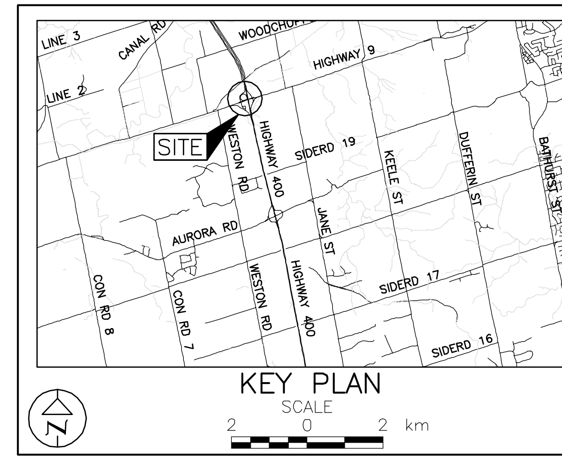


D-D' 1 EAST ABUTMENT
SCALE 6 0 6 12 m



KEY PLAN
SCALE
2 0 2 km

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HN3	248.2	4876590.9	297204.2
HN4	239.5	4876628.8	297241.2
HN5	240.3	4876595.3	297248.8
HN6	247.9	4876638.0	297280.8
HN7	248.0	4876615.6	297284.9

NOTES

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LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA Nov. 22, 2012 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Nov. 22, 2012 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D-551

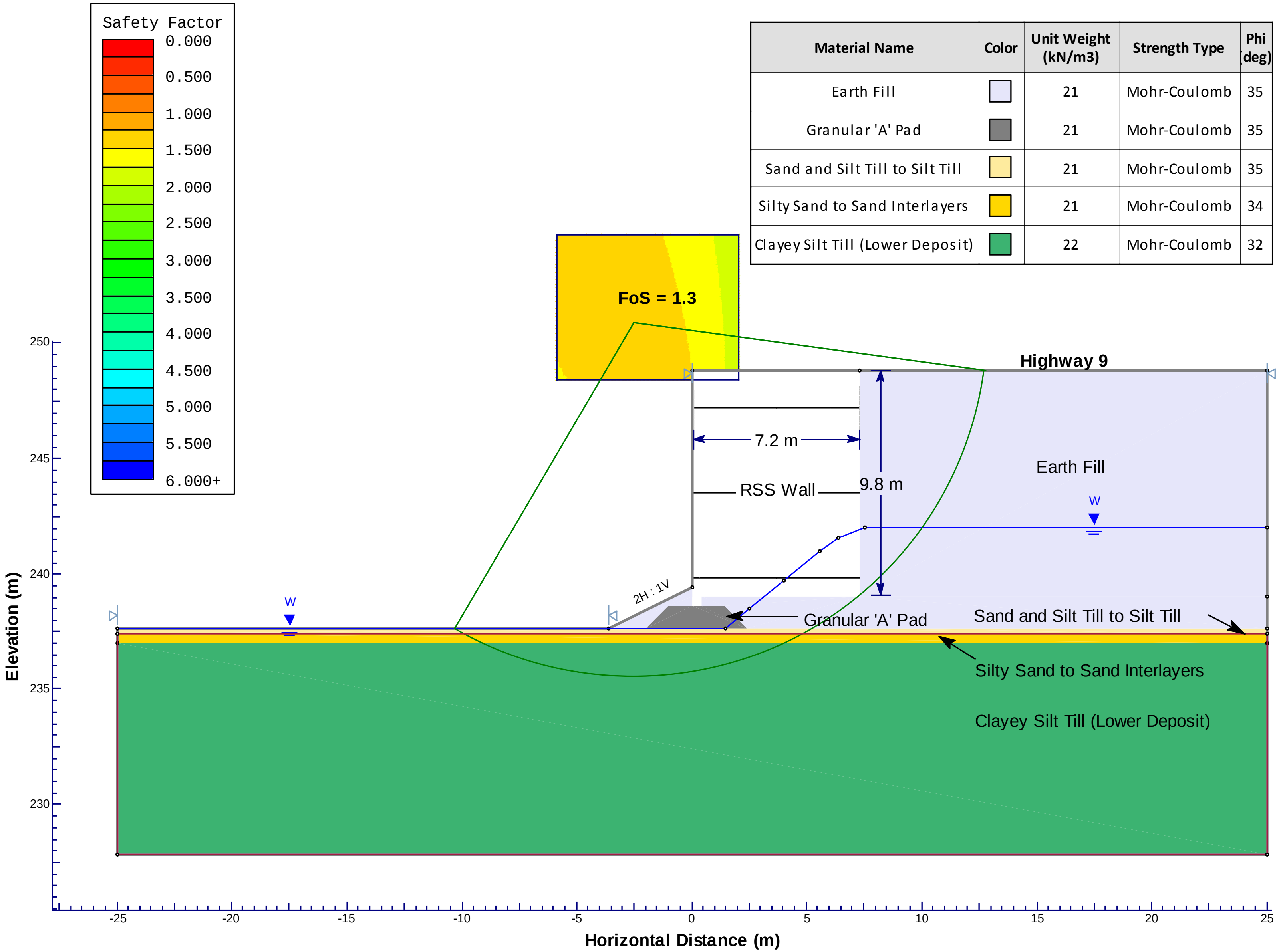
HWY. 400	PROJECT NO. 09-1111-0018		DIST.
SUBM'D. TVA	CHKD. TVA	DATE: 11/22/2012	SITE:
DRAWN: JFC	CHKD. LCC	APPD. JMAC	DWG. 2

MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: November 22, 2012
FILENAME: T:\Projects\2009\09-1111-0018 (URS, York Region)\~BB~ (HIGHWAY 9)\0911110018B002.dwg

Highway 9 Underpass – Hwy 400 Widening
RSS Wall Static Global Stability – 9.8 m High Wall

Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Phi (deg)
Earth Fill		21	Mohr-Coulomb	35
Granular 'A' Pad		21	Mohr-Coulomb	35
Sand and Silt Till to Silt Till		21	Mohr-Coulomb	35
Silty Sand to Sand Interlayers		21	Mohr-Coulomb	34
Clayey Silt Till (Lower Deposit)		22	Mohr-Coulomb	32

Date: November 2012
Project No: 09-1111-0018-2

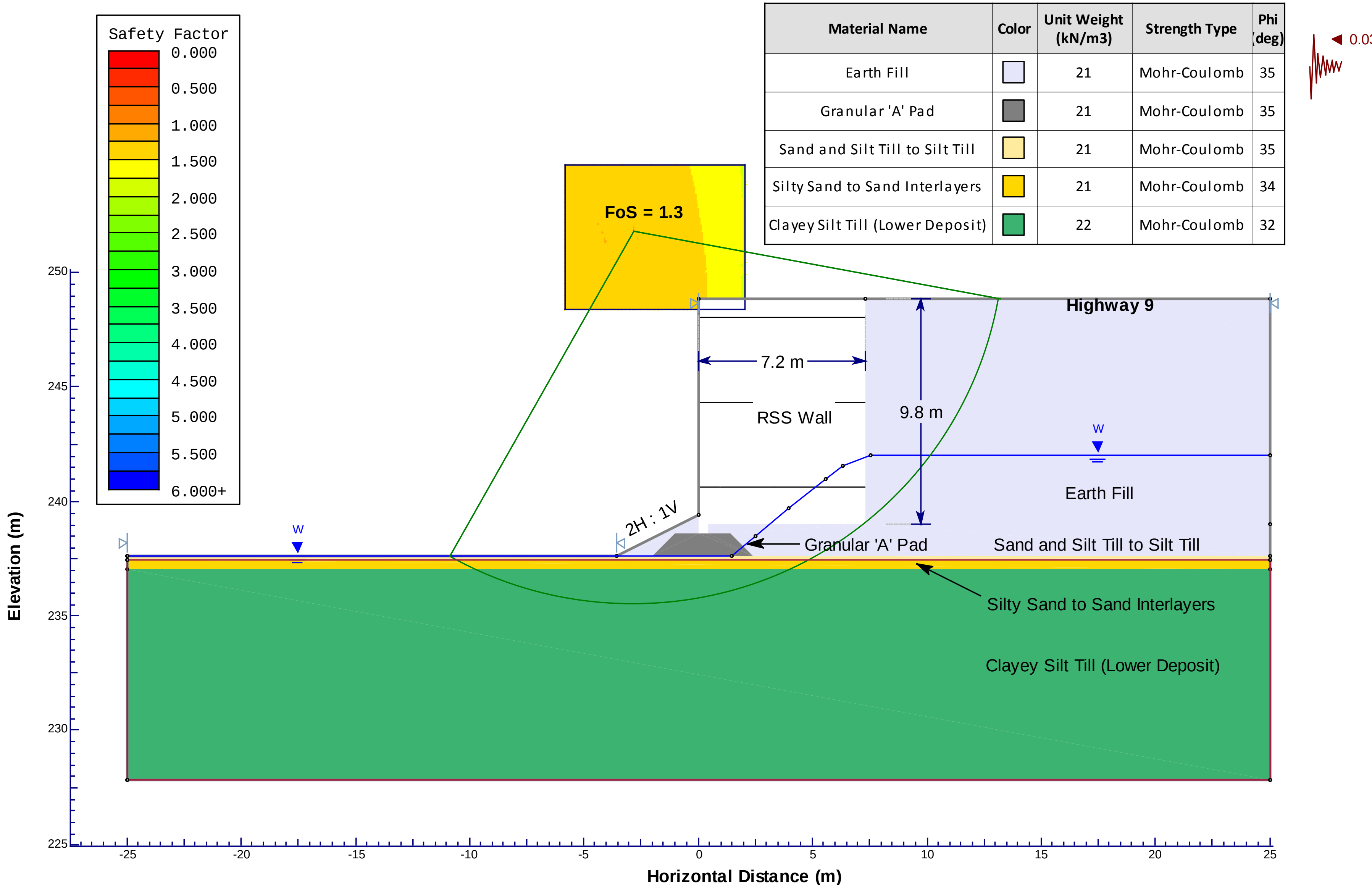
Analysis By: SKB/TVA **Reviewed By:** LCC/JMAC

Golder Associates

Highway 9 Underpass – Hwy 400 Widening
RSS Wall Seismic Global Stability – 9.8 m High Wall

Figure 2

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Phi (deg)
Earth Fill		21	Mohr-Coulomb	35
Granular 'A' Pad		21	Mohr-Coulomb	35
Sand and Silt Till to Silt Till		21	Mohr-Coulomb	35
Silty Sand to Sand Interlayers		21	Mohr-Coulomb	34
Clayey Silt Till (Lower Deposit)		22	Mohr-Coulomb	32



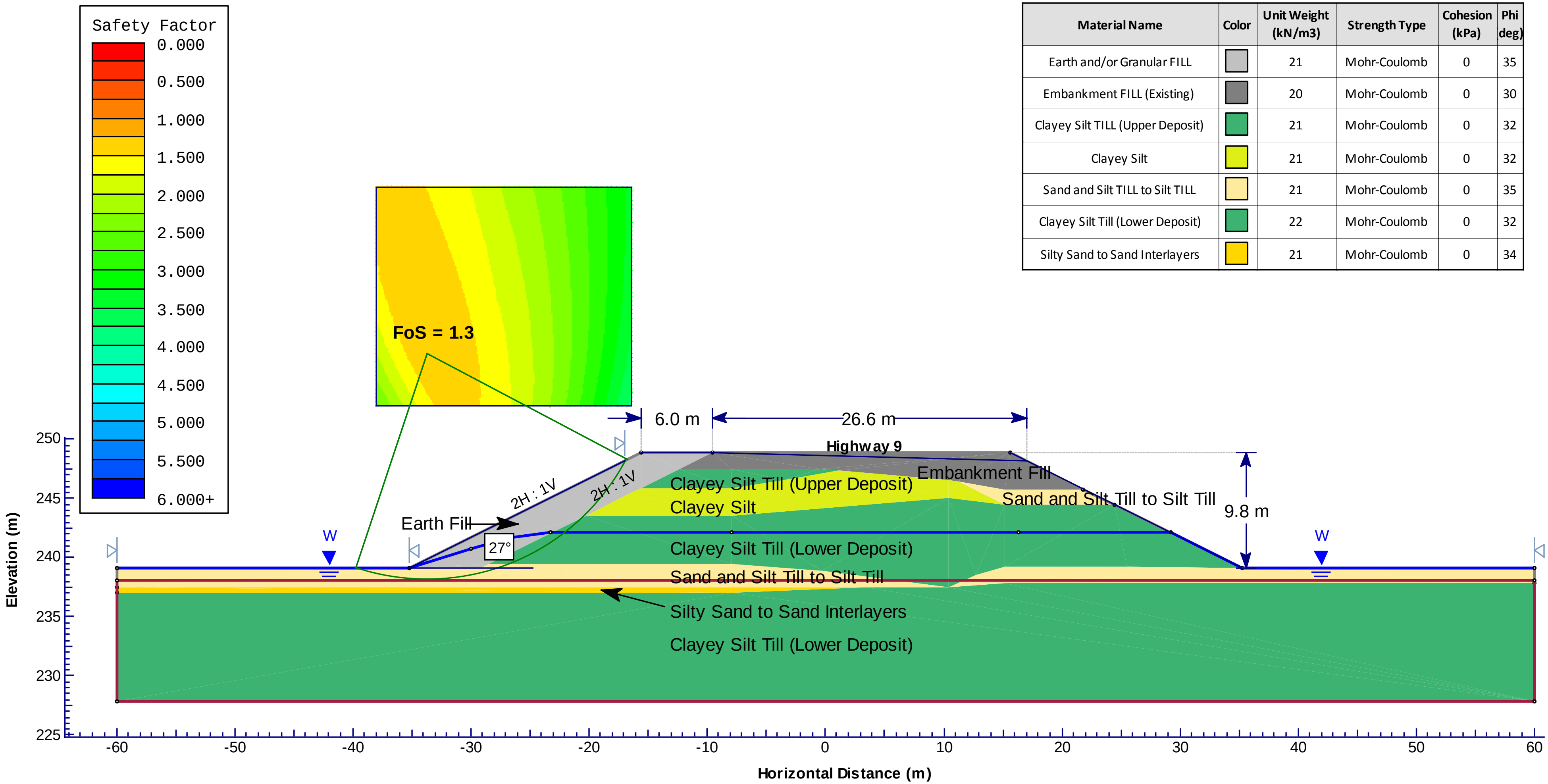
Date: November 2012
Project No: 09-1111-0018-2

Analysis By: SKB/TVA **Reviewed By:** LCC/JMAC

Highway 9 Underpass – Hwy 400 Widening
East Approach Embankment Static Global Stability

Figure 3

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Earth and/or Granular FILL		21	Mohr-Coulomb	0	35
Embankment FILL (Existing)		20	Mohr-Coulomb	0	30
Clayey Silt TILL (Upper Deposit)		21	Mohr-Coulomb	0	32
Clayey Silt		21	Mohr-Coulomb	0	32
Sand and Silt TILL to Silt TILL		21	Mohr-Coulomb	0	35
Clayey Silt Till (Lower Deposit)		22	Mohr-Coulomb	0	32
Silty Sand to Sand Interlayers		21	Mohr-Coulomb	0	34



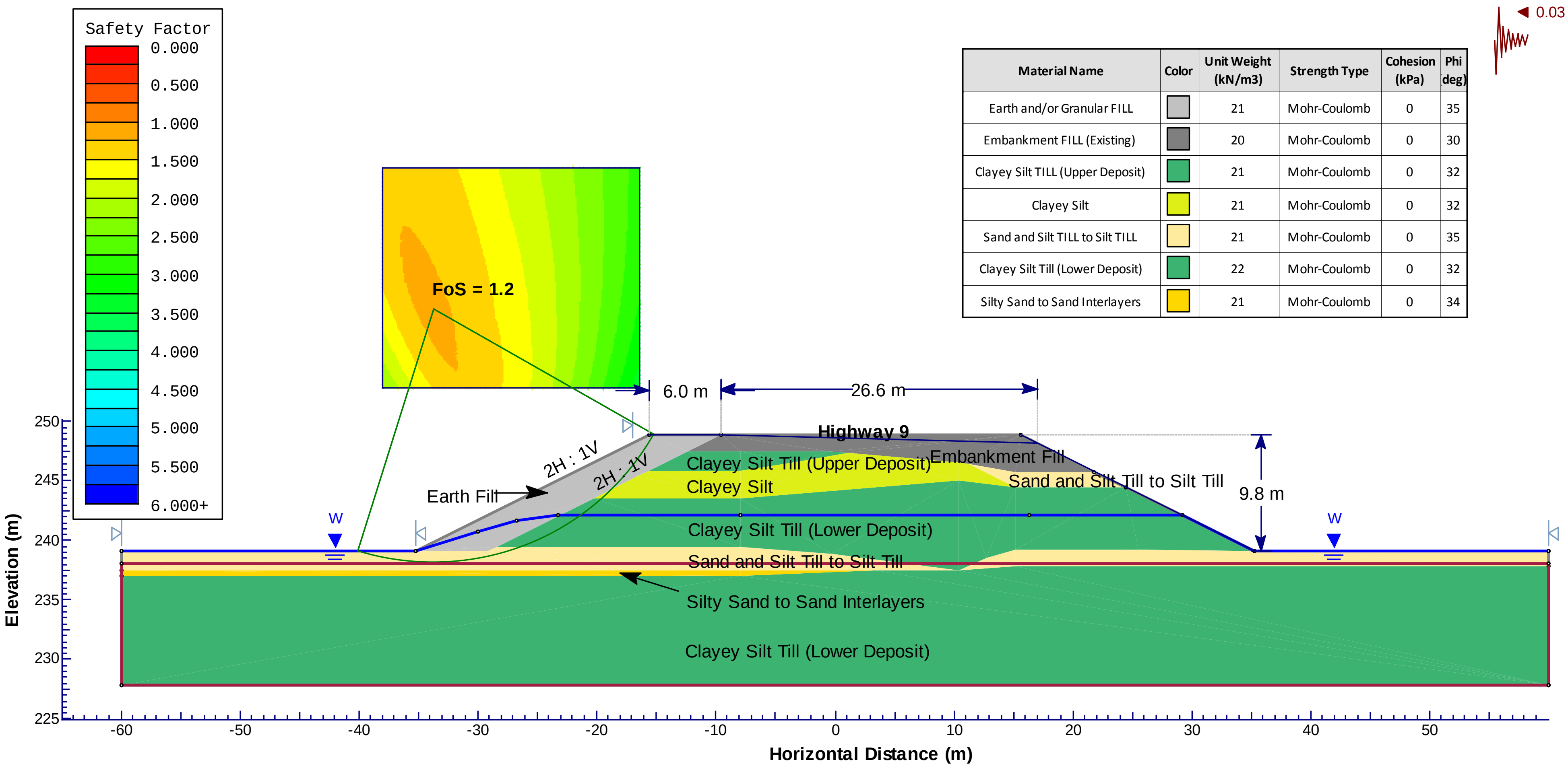
Date: November 2012
Project No: 09-1111-0018-2

Analysis By: SKB/TVA **Reviewed By:** LCC/JMAC

Golder Associates

Highway 9 Underpass – Hwy 400 Widening
East Approach Embankment Seismic Global Stability

Figure 4



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Earth and/or Granular FILL		21	Mohr-Coulomb	0	35
Embankment FILL (Existing)		20	Mohr-Coulomb	0	30
Clayey Silt TILL (Upper Deposit)		21	Mohr-Coulomb	0	32
Clayey Silt		21	Mohr-Coulomb	0	32
Sand and Silt TILL to Silt TILL		21	Mohr-Coulomb	0	35
Clayey Silt Till (Lower Deposit)		22	Mohr-Coulomb	0	32
Silty Sand to Sand Interlayers		21	Mohr-Coulomb	0	34

Date: November 2012
Project No: 09-1111-0018-2

Analysis By: SKB/TVA **Reviewed By:** LCC/JMAC

APPENDIX A

Record of Borehole Sheets and Laboratory Test Results

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 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

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 = σ'_p / σ'_{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 = $\tan\delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

Golder Associates

RECORD OF BOREHOLE No HN1

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0018
G.W.P. 2835-02-00(a) LOCATION N 4876591.2 ;E 297170.0 ORIGINATED BY TT
DIST Central HWY 400 BOREHOLE TYPE 108 mm Outside Diameter Continuous Flight Solid Stem Auger COMPILED BY SKB/SMM
DATUM Geodetic DATE October 27, 2010 CHECKED BY TVA

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
248.4	GROUND SURFACE							
0.0	ASPHALT							
0.2	Sand, trace to some silt, trace clay, trace to some gravel (FILL) Dense to very dense Brown Moist	1	SS	55				
		2A 2B	SS	42	7	78	12	3
246.5 / 1.9	CLAYEY SILT, trace gravel, trace to some sand (TILL) Hard Brown to grey Moist	3	SS	53				
		4A 4B	SS	110				
245.1 / 3.4	SAND and SILT, trace to some clay, trace gravel (TILL) Very dense Grey Moist	5	SS	103	2	49	37	12
		6	SS	104				
		7A 7B	SS	105	0	8	62	30
242.0 / 6.4	CLAYEY SILT, trace to some sand, containing silt seams Hard Grey Moist	8	SS	116				
		9	SS	76				
238.2 / 10.2	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist	10A 10B	SS	30				
		11	SS	47				
235.6 / 12.8	END OF BOREHOLE							

NOTE:

1. Open borehole dry upon completion of drilling.

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (248, 247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236); 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

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

RECORD OF BOREHOLE No HN2

METRIC

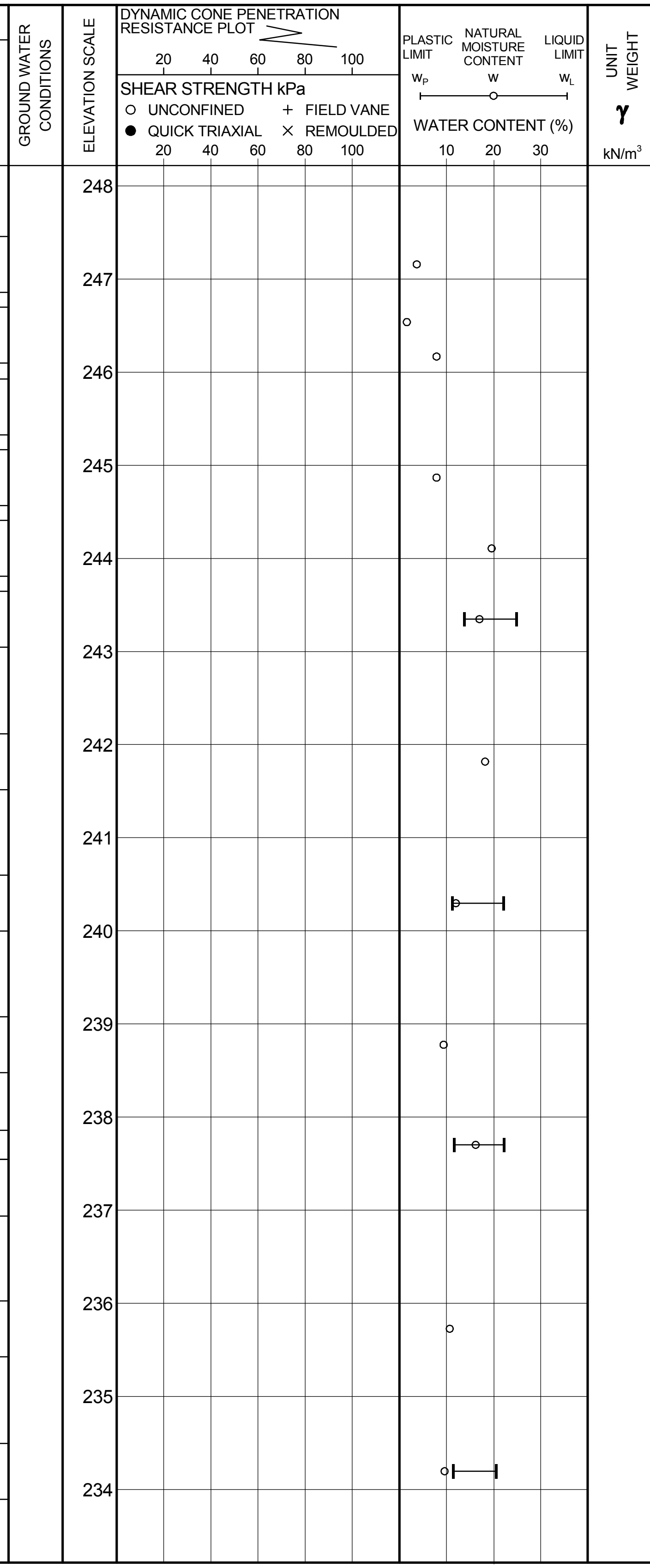
PROJECT	09-1111-0018		
G.W.P.	2835-02-00	LOCATION	N 4876612.1 ;E 297196.1
DIST	Central HWY 400	BOREHOLE TYPE	108 mm Inside Diameter Continuous Flight Hollow Stem Auger
DATUM	Geodetic	DATE	October 25 & 26, 2010
ORIGINATED BY	TT		
COMPILED BY	SKB/SMM		
CHECKED BY	TVA		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
248.2	GROUND SURFACE
0.0	ASPHALT
0.2	Sand, some silt, trace to some gravel (FILL) Compact to dense Brown Moist
246.2	Sand and silt, trace to some clay, trace to some gravel (FILL) Compact Brown Moist
245.6 / 2.6	Clayey silt with sand, trace gravel (FILL) Stiff Brown Moist
244.5 / 3.7	Gravelly sand, some silt, trace clay (FILL) Compact to very dense Brown Moist Containing clayey silt layers below a depth of 3.1 m
	CLAYEY SILT, trace to some sand, containing silt and sand seams and zones of oxidation staining Very stiff to hard Brown Moist Becoming grey below a depth of 4.9 m
241.6 / 6.7	Silty SAND, trace gravel, trace clay, containing clayey silt layers Dense Grey Wet
241.0 / 7.2	CLAYEY SILT, trace sand, containing silt seams Hard Grey Moist
239.5 / 8.7	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Grey Moist Containing silt and sand seams at a depth of 13.3 m

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	38				
2A 2B	SS	23	9	44	38	9
3A 3B	SS	13				
4	SS	64	24	58	14	4
5	SS	27				
6	SS	44	0	8	65	27
7A 7B	SS	36				
8	SS	34				
9	SS	33				
10A	AS	-				
10B	SS	31				
11	SS	27				
12	SS	49	0	15	61	24



Continued Next Page

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

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RECORD OF BOREHOLE No HN2

SHEET 2 OF 2 **METRIC**

PROJECT	09-1111-0018		
G.W.P.	2835-02-00	LOCATION	N 4876612.1 ;E 297196.1
ORIGINATED BY	TT		
DIST	Central	HWY	400
BOREHOLE TYPE	108 mm Inside Diameter Continuous Flight Hollow Stem Auger	COMPILED BY	SKB/SMM
DATUM	Geodetic	DATE	October 25 & 26, 2010
CHECKED BY	TVA		

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Grey Moist	13	SS	66		
	Becoming wet at a depth of 16.3 m				∇	
230.9 / 17.3	Silty SAND, trace clay Very dense Grey Wet	14A / 14B	SS	116		
230.4 / 17.8	CLAYEY SILT, some sand, containing sand layers (TILL) Hard Grey Wet	15	SS	144		0 19 63 18
228.0	SAND, some silt Very dense Grey Wet	16A / 16B	SS	100		
20.4	END OF BOREHOLE					

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

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

NOTES:

1. Water level in open borehole at a depth of 16.8 m below ground surface (Elevation 231.4 m) upon completion of drilling.

2. Borehole caved to a depth of 17.4 m upon completion of drilling.

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

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Foundation Design

RECORD OF BOREHOLE No HN3

SHEET 1 OF 2 **METRIC**

PROJECT 09-1111-0018

G.W.P. 2835-02-00

LOCATION N 4876590.9 ;E 297204.2

ORIGINATED BY TWB/CS

DIST Central HWY 400

BOREHOLE TYPE 108 mm Inside Diameter Continuous Flight Hollow Stem Auger

COMPILED BY SKB

DATUM Geodetic

DATE October 15, 2010

CHECKED BY TVA

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
248.2	GROUND SURFACE					
0.0	ASPHALT					
247.2 / 0.2	Sand and gravel, trace silt (FILL) Very dense Brown Moist	1A 1B	SS	79/.23		
244.3 / 1.0	CLAYEY SILT with SAND, trace gravel (TILL) Hard Brown Moist Augers grinding at a depth of 1.4 m	2	SS	31		
		3	SS	82		
		4	SS	79		
239.5 / 3.9	CLAYEY SILT, trace sand, containing silty clay, sandy silt and wet silty sand interlayers Hard Grey Moist	5	SS	51		
		6	SS	55		0 3 70 27
		7	SS	113		
		8	SS	67		
8.7	CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Moist	9	SS	44		
	Augers grinding between depths of 10.1 m and 10.4 m	10	SS	36		
	Augers grinding between depths of 11.3 m and 12.2 m	11	SS	31		
		12	SS	66		1 27 60 12

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 248 247 246 245 244 243 242 241 240 239 238 237 236 235 234

Continued Next Page

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

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

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RECORD OF BOREHOLE No HN3

SHEET 2 OF 2 **METRIC**

PROJECT 09-1111-0018			
G.W.P. 2835-02-00	LOCATION N 4876590.9 ;E 297204.2		ORIGINATED BY TWB/CS
DIST Central HWY 400	BOREHOLE TYPE 108 mm Inside Diameter Continuous Flight Hollow Stem Auger		COMPILED BY SKB
DATUM Geodetic	DATE October 15, 2010		CHECKED BY TVA

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---					233	
232.8 / 15.4	CLAYEY SILT, trace sand, containing silty clay and sandy silt interlayers. Hard. Grey. Moist	13A 13B	SS	133			0 4 63 33
						232	
231.9 / 16.3	CLAYEY SILT, some sand, trace gravel, containing silty sand interlayer between depths of 17.0 m and 17.2 m (TILL). Hard. Grey. Moist	14	SS	38		231	
						230	
		15	SS	40			
229.3 / 18.9	END OF BOREHOLE						

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

NOTES:

1. Water level in open borehole at a depth of 14.3 m below ground surface (Elevation 233.9 m) upon completion of drilling.

2. Borehole caved to a depth of 17.4 m upon completion of drilling.

3. Augers could not be advanced past a depth of 1.4 m in original borehole. Borehole redrilled 0.8 m west of the original location.

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

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RECORD OF BOREHOLE No HN4

SHEET 1 OF 1 **METRIC**

PROJECT	09-1111-0018		
G.W.P.	2835-02-00	LOCATION	N 4876628.8 ;E 297241.2
DIST	Central HWY 400	BOREHOLE TYPE	108 mm Outside Diameter Continuous Flight Solid Stem Auger
DATUM	Geodetic	DATE	November 3, 2010
ORIGINATED BY	TT		
COMPILED BY	SKB		
CHECKED BY	TVA		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
239.5	GROUND SURFACE
239.2 / 0.0	ASPHALT
0.3	Sand, trace to some silt, trace gravel (FILL) Compact Brown Moist
238.3 / 238.0 / 237.7 / 1.8	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff Grey Moist
	SAND, trace to some silt, containing silty clay pockets Compact Brown Moist
	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff Grey Moist
236.2 / 3.4	CLAYEY SILT, containing silt seams Stiff to very stiff Grey Moist
234.6 / 4.9	CLAYEY SILT, trace sand, trace gravel, containing silt seams (TILL) Stiff to hard Grey Wet
229.8 / 9.8	END OF BOREHOLE

NOTE:

1. Open borehole dry upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1A 1B	SS	17				
2A 2B	SS	19				
3	SS	22				
4A 4B	SS	16				
5	SS	21	0	0	63	37
6A 6B	SS	10				
7	SS	73				
8	SS	108	0	23	56	21
9	SS	109				

ELEVATION SCALE: 239, 238, 237, 236, 235, 234, 233, 232, 231, 230

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

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m³

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

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

RECORD OF BOREHOLE No HN5

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0018
G.W.P. 2835-02-00
DIST Central HWY 400
DATUM Geodetic

LOCATION N 4876595.3 ;E 297248.8
BOREHOLE TYPE 108 mm Outside Diameter Continuous Flight Solid Stem Auger
DATE November 4, 2010

ORIGINATED BY TT
COMPILED BY SKB
CHECKED BY TVA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
240.3	GROUND SURFACE				
240.0 / 0.0	ASPHALT				
0.3	Sand, some silt, trace gravel (FILL) Compact Brown Moist	1A 1B	SS	11	
239.1 / 1.2	CLAYEY SILT, some sand, trace gravel Firm to stiff Grey Moist	2	SS	7	
238.1 / 2.2	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff Grey Moist	3	SS	22	
		4	SS	30	1 27 53 19
		5	SS	28	
235.8 / 4.5	CLAYEY SILT, containing silt seams and layers Very stiff Grey Moist	6	SS	20	
234.7 / 5.6	CLAYEY SILT, some sand. trace gravel, containing wet zones of silty clay (TILL) Hard Grey Moist	7	SS	34	
233.1 / 7.2	CLAYEY SILT, some sand, containing silt pockets Hard Grey Wet	8	SS	70	0 11 54 35
231.6 / 8.7	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist	9	SS	68	
		10	SS	101	0 18 59 23
		11	SS	109	
	NOTE: 1. Open borehole dry upon completion of drilling.	12	SS	100/.25	
226.2 / 14.1	END OF BOREHOLE				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (240 to 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/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Golder Associates

RECORD OF BOREHOLE No HN6

SHEET 1 OF 2 **METRIC**

PROJECT 09-1111-0018			
G.W.P. 2835-02-00	LOCATION N 4876638.0 ;E 297280.8	ORIGINATED BY TT	
DIST Central HWY 400	BOREHOLE TYPE 108 mm Inside Diameter Continuous Flight Hollow Stem Auger	COMPILED BY SKB/SM	
DATUM Geodetic	DATE October 22 & 25, 2010	CHECKED BY TVA	

ELEV / DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
247.9	GROUND SURFACE						
0.0	ASPHALT						
0.1	Silty sand, trace to some gravel, containing clayey silt with sand seams (FILL) Compact to very dense Brown Moist	1	SS	18		247	
		2	SS	53		246	
245.7 / 2.2	SAND, some gravel, some silt, trace clay (TILL) Dense to very dense Brown Moist	3	SS	86		245	16 66 14 4
		4A / 4B	SS	48			
244.5 / 3.5	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Brown Moist	5	SS	37		244	
	Containing zones of oxidation staining between depths of 4.6 m and 5.2 m	6	SS	67	∇	243	0 23 56 21
	Becoming grey below a depth of 5.6 m					242	
		7	SS	24		241	
		8	SS	41		240	
239.2 / 8.7	SAND and SILT, trace gravel, trace to some clay (TILL) Very dense Grey Moist					239	
		9	SS	79		238	1 55 36 8
237.7 / 10.2	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist	10	SS	43		237	
						236	
		11	SS	40		235	
		12	SS	54		234	1 18 56 25
						233	

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

Golder Associates

RECORD OF BOREHOLE No HN6

METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00

LOCATION N 4876638.0 ;E 297280.8

ORIGINATED BY TT

DIST Central HWY 400

BOREHOLE TYPE 108 mm Inside Diameter Continuous Flight Hollow Stem Auger

COMPILED BY SKB/SM

DATUM Geodetic

DATE October 22 & 25, 2010

CHECKED BY TVA

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER (SAMPLES)	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 ---										
	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist		13	SS	52						
							232				
							231				
			14	SS	72						
230.5 17.4	END OF BOREHOLE										

NOTE:

1. Water level in open borehole at a depth of 5.1 m below ground surface (Elevation 242.8 m) upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No HN7

METRIC

PROJECT 09-1111-0018
G.W.P. 2835-02-00
LOCATION N 4876615.6 ;E 297284.9
ORIGINATED BY TWB
DIST Central HWY 400
BOREHOLE TYPE 108 mm Inside Diameter Continuous Flight Hollow Stem Auger
COMPILED BY SKB
DATUM Geodetic
DATE October 20 & 21, 2010
CHECKED BY TVA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
248.0	GROUND SURFACE				
0.0	ASPHALT				
0.2	Sand and gravel, trace silt, trace clay (FILL) Brown Moist				
247.4 0.6	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to hard Brown Moist	1	SS	9	
		2	SS	39	
245.8 2.2	CLAYEY SILT, some sand, containing sandy silt and silty sand interlayers Very stiff Brown Moist	3	SS	25	
		4	SS	23	
		5	SS	21	
243.5 4.5	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Brown Moist. Becomes grey below a depth of 5.6 m	6	SS	29	
		7	SS	29	0 21 65 14
		8	SS	34	
239.3 8.7	SILT, trace clay, trace sand (TILL) Very dense Grey Moist	9	SS	63	0 1 93 6
237.3	Silty SAND, trace clay, trace gravel Very dense Grey Moist	10A 10B	SS	57	
236.9 11.1	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist	11	SS	61	1 21 58 20
		12	SS	53	

Continued Next Page

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

GTA-MTO 001 09111110018.GPJ GAL-GTA.GDT 11/22/12 SIB

RECORD OF BOREHOLE No HN7

SHEET 2 OF 2 **METRIC**

PROJECT 09-1111-0018
G.W.P. 2835-02-00 LOCATION N 4876615.6 ;E 297284.9 ORIGINATED BY TWB
DIST Central HWY 400 BOREHOLE TYPE 108 mm Inside Diameter Continuous Flight Hollow Stem Auger COMPILED BY SKB
DATUM Geodetic DATE October 20 & 21, 2010 CHECKED BY TVA

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist	13	SS	89		w ≈ 9
					232	
		14	SS	105	231	w ≈ 9, w_P–w_L ≈ 10–14
	Containing silty sand seams between depths of 17.8 m and 19.2 m				230	
		15	SS	50/.05		w ≈ 9
					229	
227.8		16	SS	70/.10	228	w ≈ 10
20.2	END OF BOREHOLE					

NOTES:

1. Water level in borehole at a depth of 13.0 m below ground surface (Elevation 235.0 m) upon completion of drilling.

2. Water level measurement in the piezometer:

Date	Depth (m)	Elev. (m)
11/25/10	1.9	246.1
12/02/10	8.2	239.8

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

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

Golder Associates

RECORD OF BOREHOLE No HN8

SHEET 1 OF 1

METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00(a)

LOCATION N 4876634.4 ;E 297317.5

ORIGINATED BY TT

DIST Central HWY 400

BOREHOLE TYPE 108 mm Outside Diameter Continuous Flight Solid Stem Auger

COMPILED BY SKB

DATUM Geodetic

DATE October 27 & November 19, 2010

CHECKED BY TVA

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
248.0	GROUND SURFACE
0.0	ASPHALT
0.3	Silty sand, trace to some gravel (FILL) Dense Brown Moist
246.6 / 1.5	CLAYEY SILT, trace to some sand, containing silt and sand pockets Very stiff to hard Brown Moist
244.3 / 3.7	CLAYEY SILT, trace to some sand, trace gravel (TILL) Very stiff Brown Moist
243.5	Silty SAND, trace clay, trace gravel Dense Brown Moist
243.2 / 4.8	CLAYEY SILT with SAND, trace gravel (TILL) Hard Brown Moist Becoming grey at a depth of 5.6 m Containing sand seams at a depth of 6.1 m
239.3 / 8.7	SAND and SILT, trace clay, trace gravel (TILL) Dense Grey Moist
238.5 / 9.6	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist
236.7 / 11.3	END OF BOREHOLE NOTE: 1. Open borehole dry upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	36	
2	SS	25	
3	SS	44	
4	SS	19	0 10 61 29
5	SS	26	
6A / 6B	SS	41	
7	SS	40	2 31 51 16
8	SS	32	
9A / 9B	SS	41	
10	SS	33	

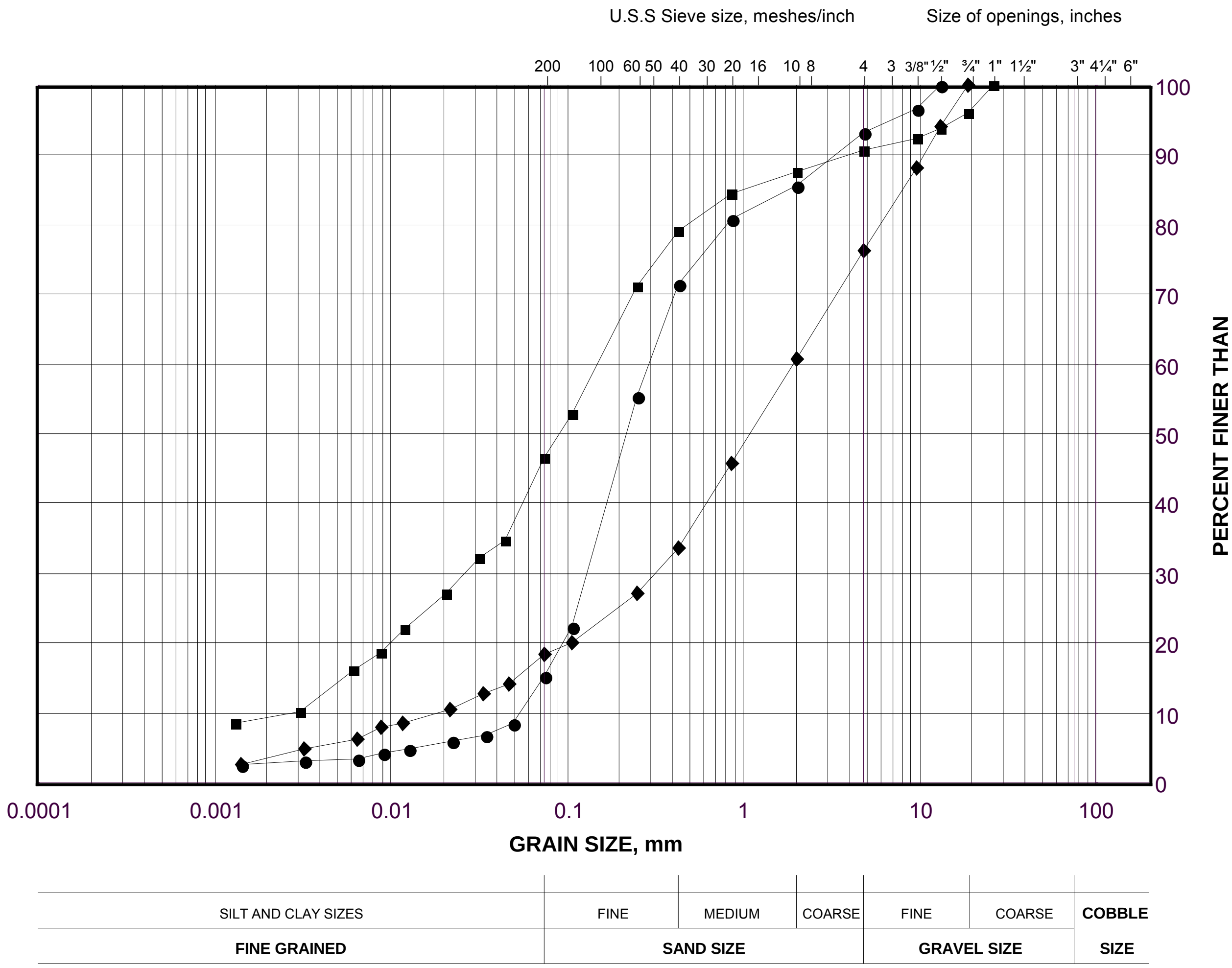
Columns: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

GRAIN SIZE DISTRIBUTION

Sand to Gravelly Sand Fill

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HN1	2A	246.7
■	HN2	2B	246.2
◆	HN2	4	244.9

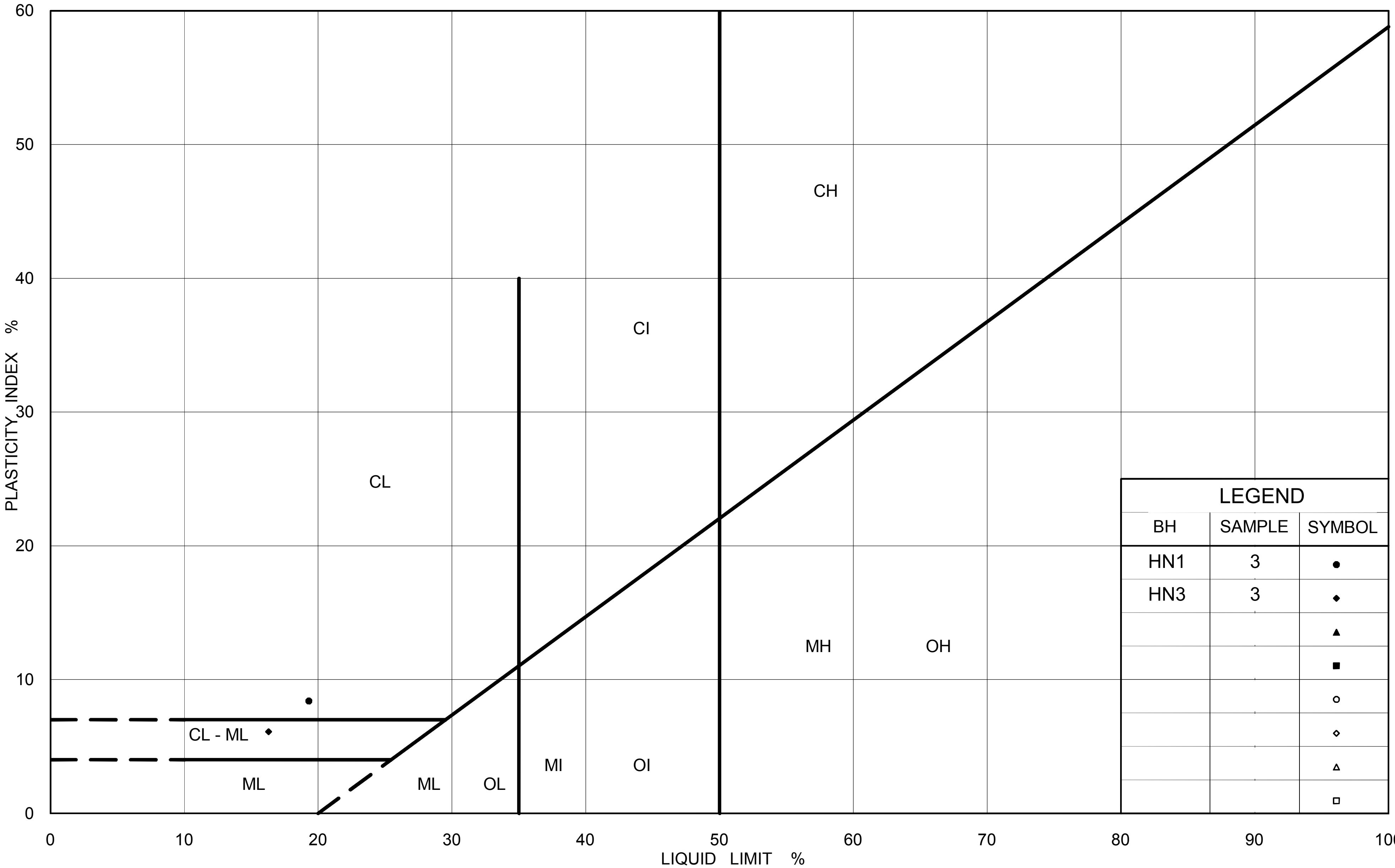
Project Number: 09-1111-0018

Checked By: TVA

Golder Associates

Date: 13-Jun-11

Oct 75, FF-S-21



Ministry of Transportation

Ontario

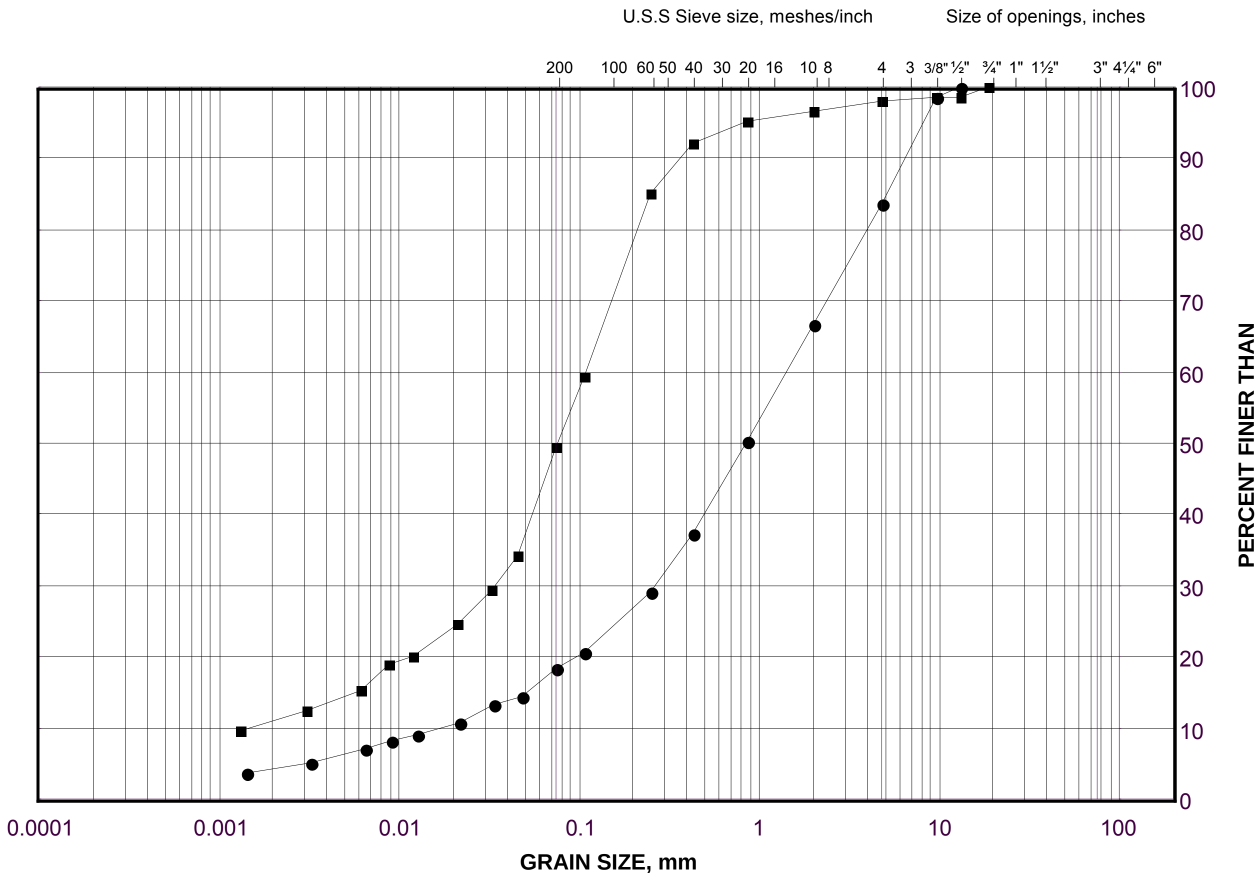
PLASTICITY CHART
Clayey Silt Till (Upper Deposit)

Figure No. 2

Project No. 09-1111-0018

Checked By:TVA

GRAIN SIZE DISTRIBUTION Sand to Sand and Silt Till (Upper Deposit)	FIGURE 3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HN6	3	245.3
■	HN1	5	244.3

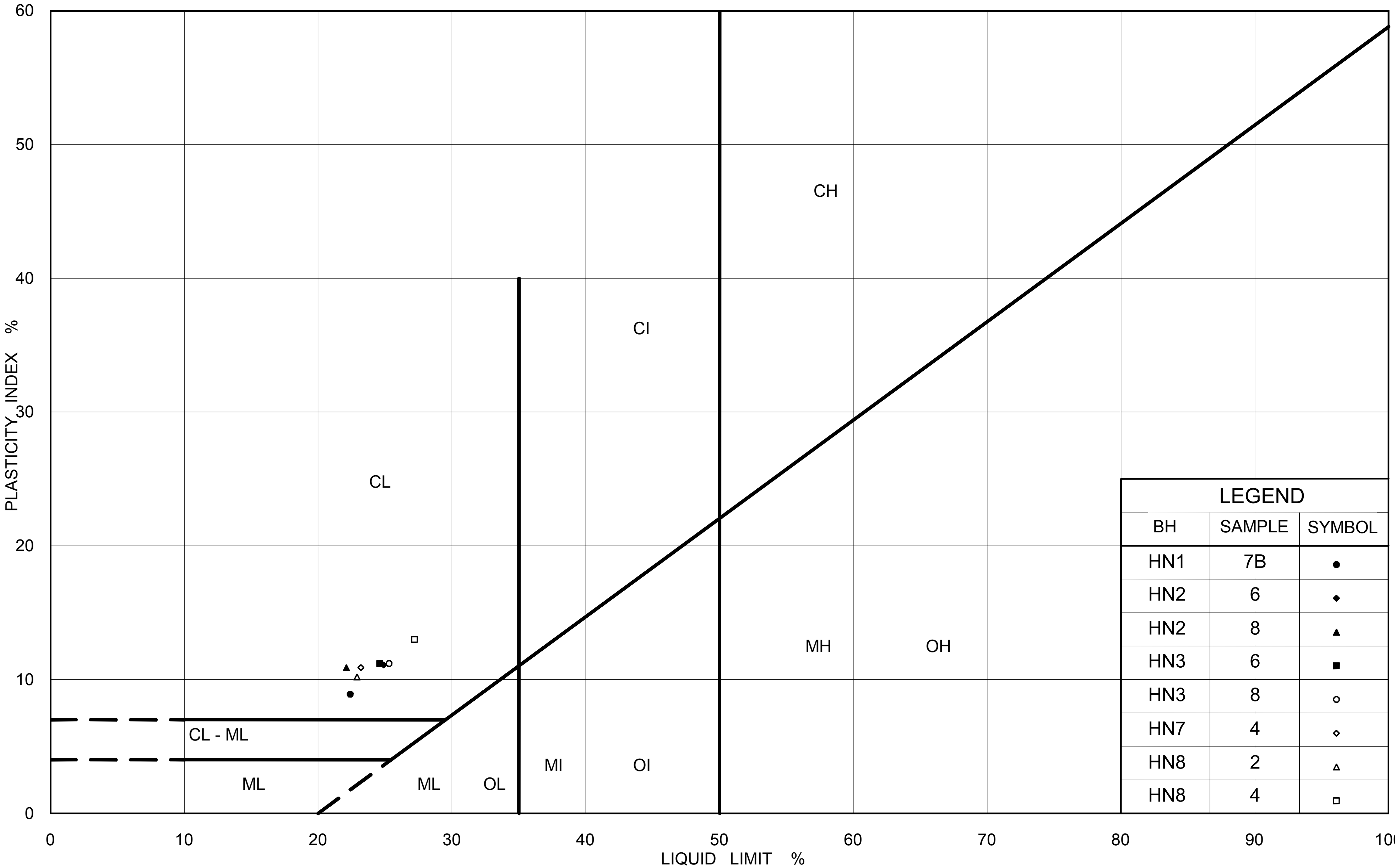
Project Number: 09-1111-0018

Checked By: TVA

Golder Associates

Date: 13-Jun-11

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
HN1	7B	●
HN2	6	◆
HN2	8	▲
HN3	6	■
HN3	8	○
HN7	4	◇
HN8	2	△
HN8	4	□

Ministry of Transportation

Ontario

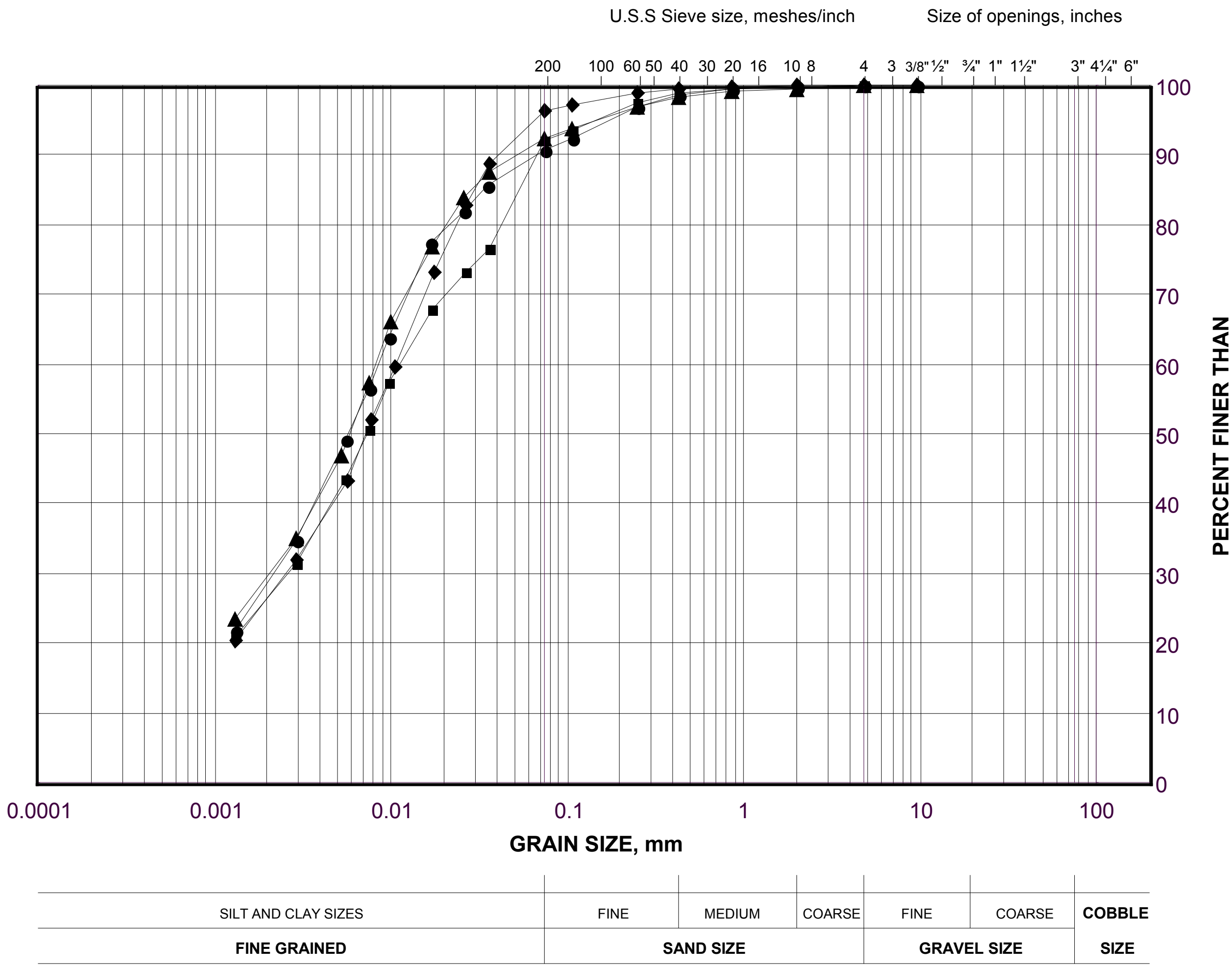
PLASTICITY CHART
Clayey Silt

Figure No. 4

Project No. 09-1111-0018

Checked By: TVA

GRAIN SIZE DISTRIBUTION Clayey Silt	FIGURE 5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HN8	4	244.7
■	HN2	6	243.3
◆	HN3	6	243.3
▲	HN1	7B	241.9

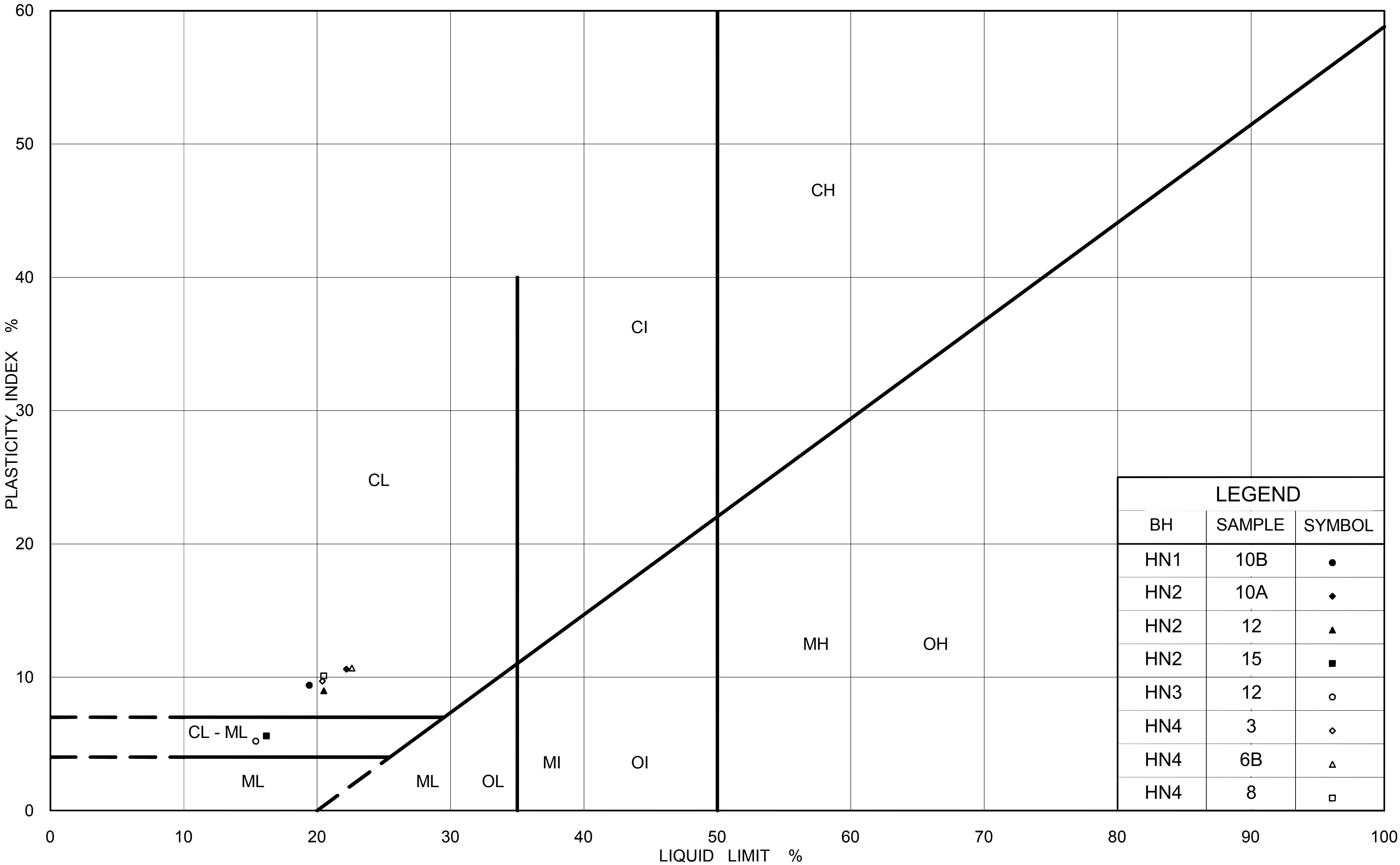
Project Number: 09-1111-0018

Checked By: TVA

Golder Associates

Date: 13-Jun-11

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
HN1	10B	●
HN2	10A	◆
HN2	12	▲
HN2	15	■
HN3	12	○
HN4	3	◇
HN4	6B	△
HN4	8	□

Ministry of Transportation

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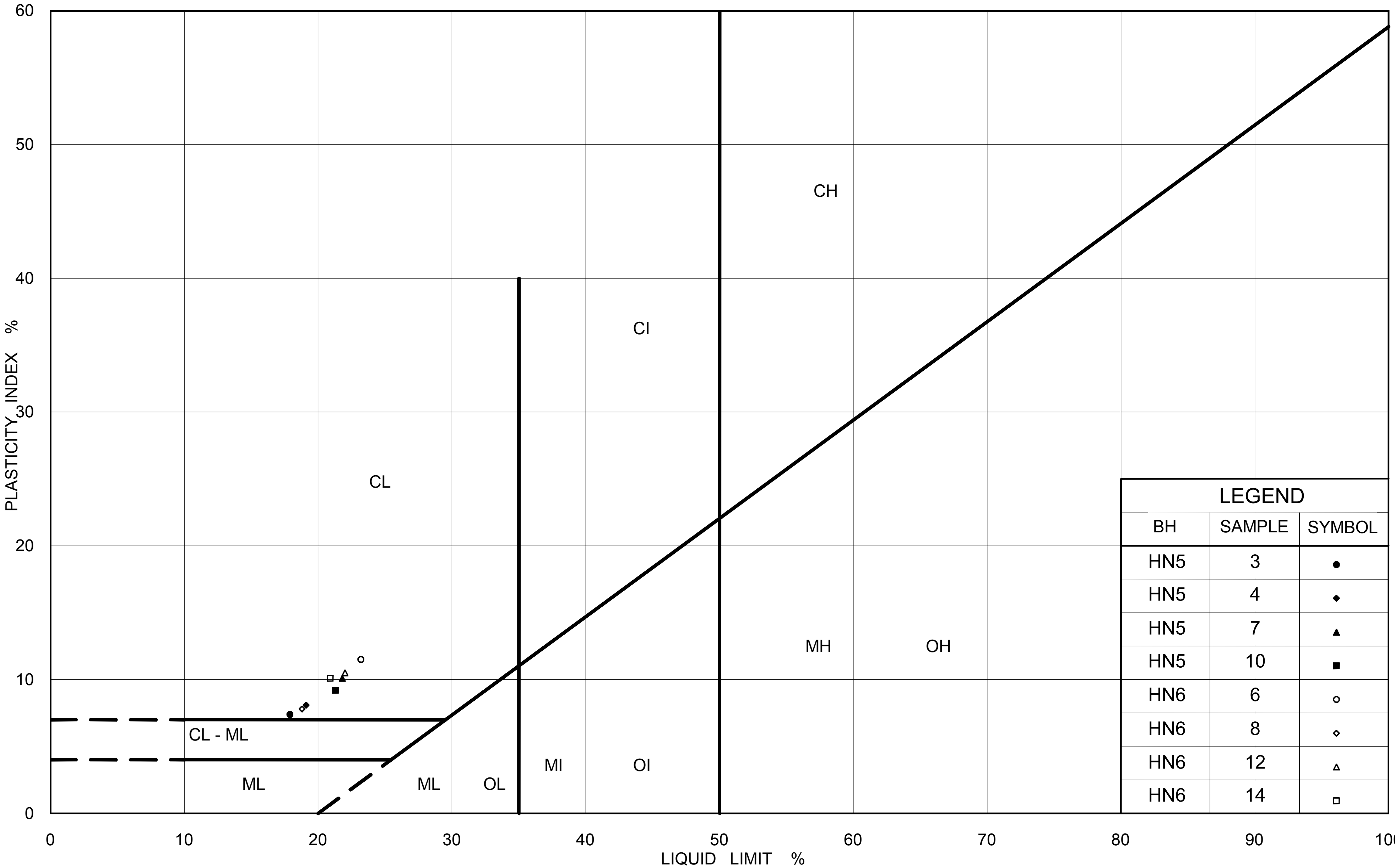
PLASTICITY CHART
Clayey Silt Till (Lower Deposit)

Figure No. 6A

Project No. 09-1111-0018

Checked By: TVA

Oct 75, FF-S-21



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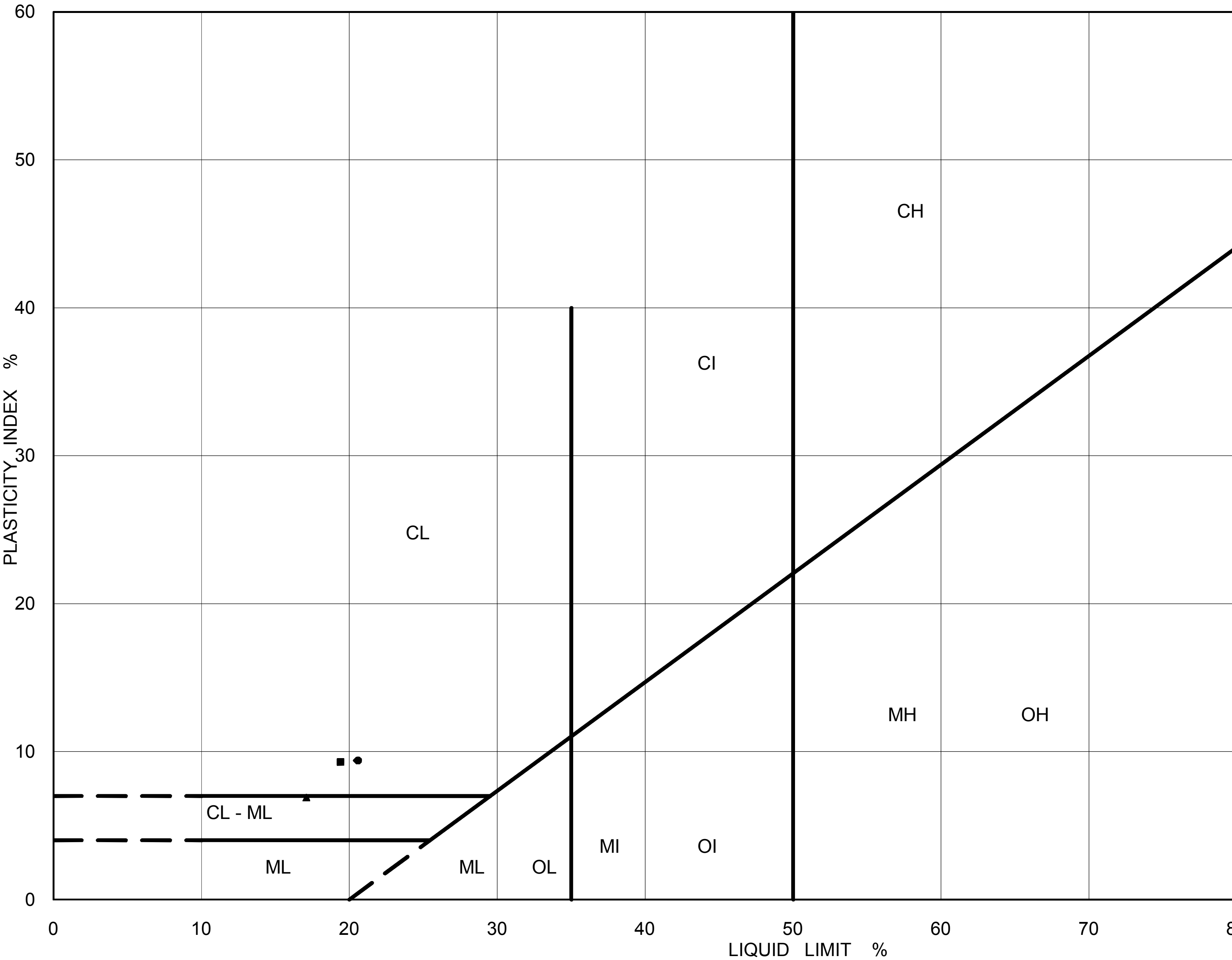
PLASTICITY CHART
Clayey Silt Till (Lower Deposit)

Figure No. 6B

Project No. 09-1111-0018

Checked By: TVA

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
HN7	7	●
HN7	11	◆
HN7	14	▲
HN8	7	■
		○
		◇
		△
		□

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PLASTICITY CHART
Clayey Silt Till (Lower Deposit)

Figure No. 6C

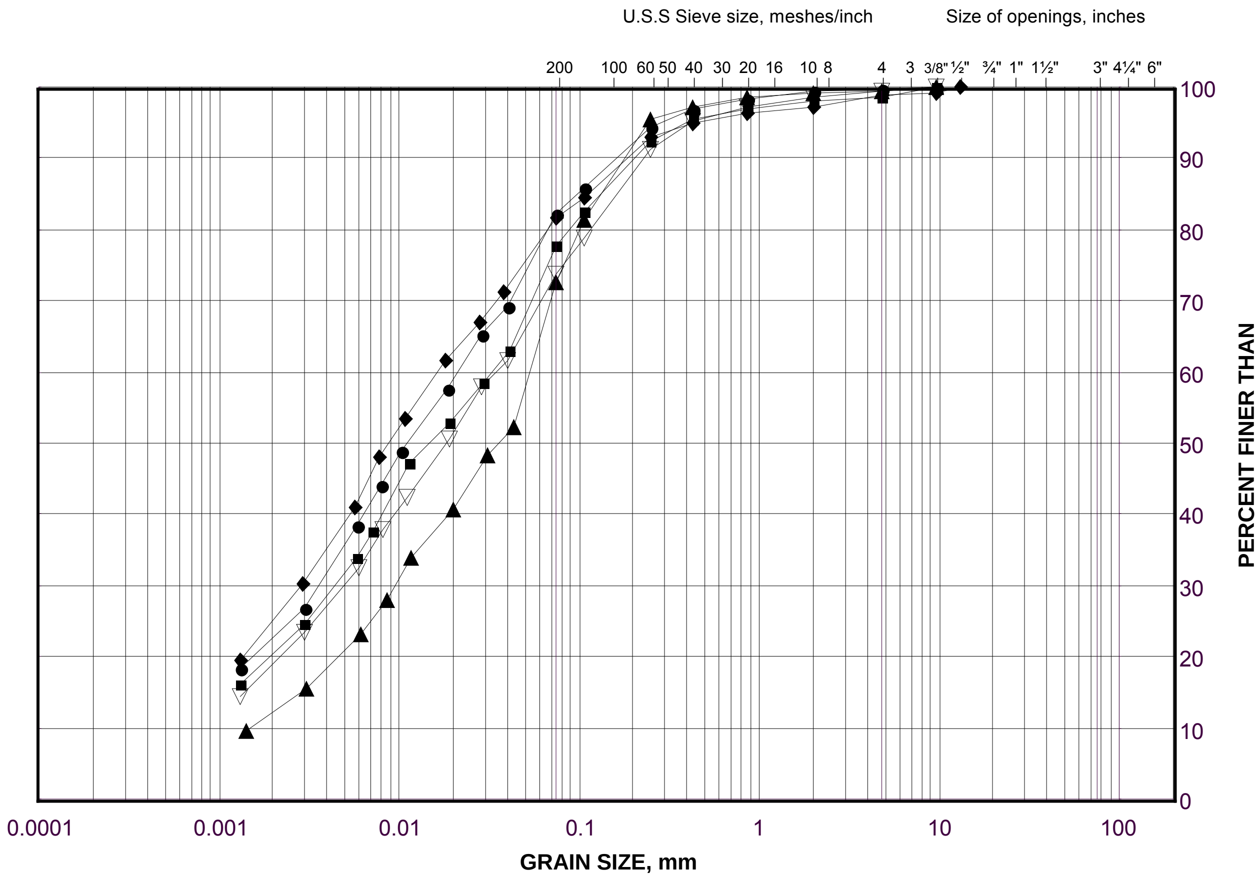
Project No. 09-1111-0018

Checked By: TVA

GRAIN SIZE DISTRIBUTION

Clayey Silt Till (Lower Deposit)

FIGURE 7A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HN5	10	229.3
■	HN7	11	235.6
◆	HN6	12	233.9
▲	HN3	12	234.2
▽	HN5	4	237.0

Project Number: 09-1111-0018

Checked By: TVA

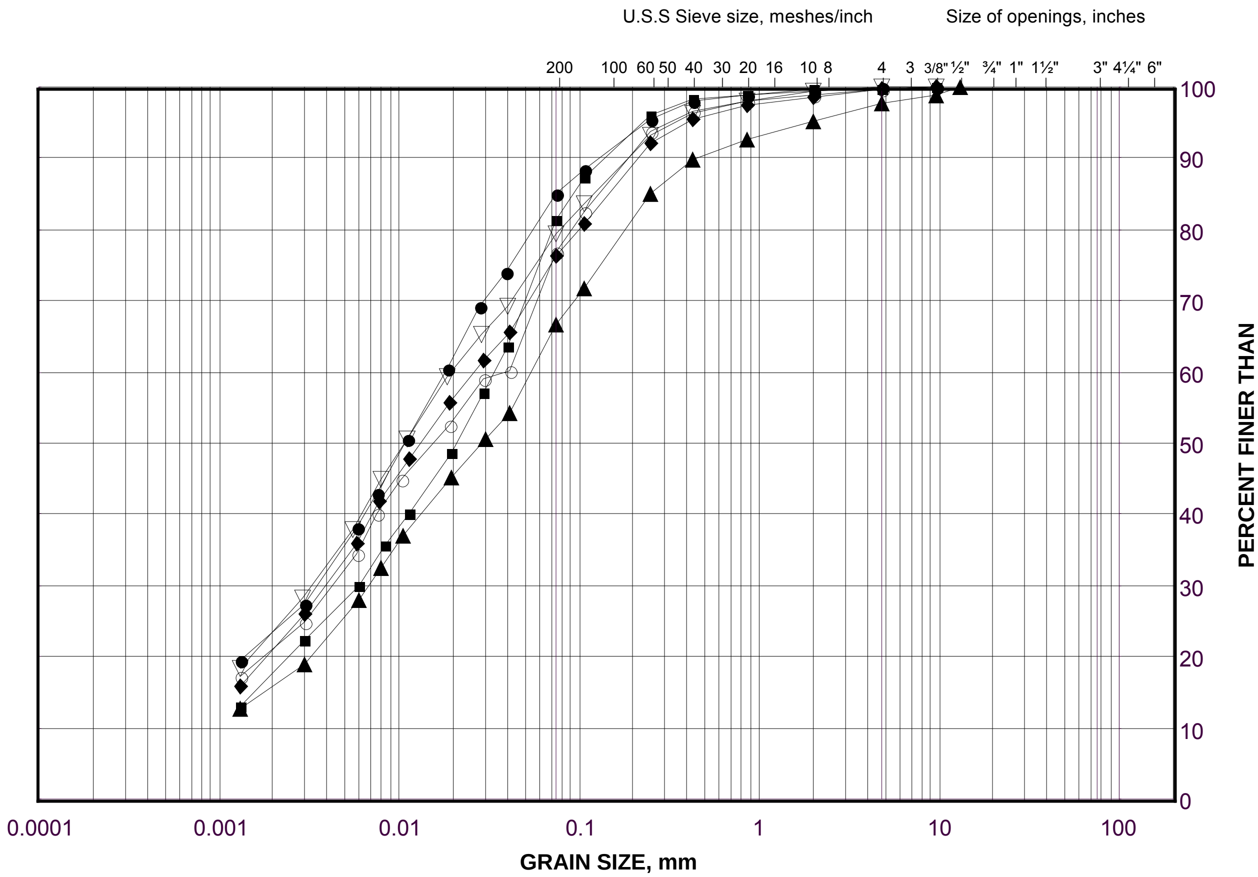
Golder Associates

Date: 13-Jun-11

GRAIN SIZE DISTRIBUTION

Clayey Silt Till (Lower Deposit)

FIGURE 7B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HN2	12	234.2
■	HN2	15	229.6
◆	HN6	6	243.1
▲	HN8	7	241.7
▽	HN7	7	241.7
❍	HN4	8	231.6

Project Number: 09-1111-0018

Checked By: TVA

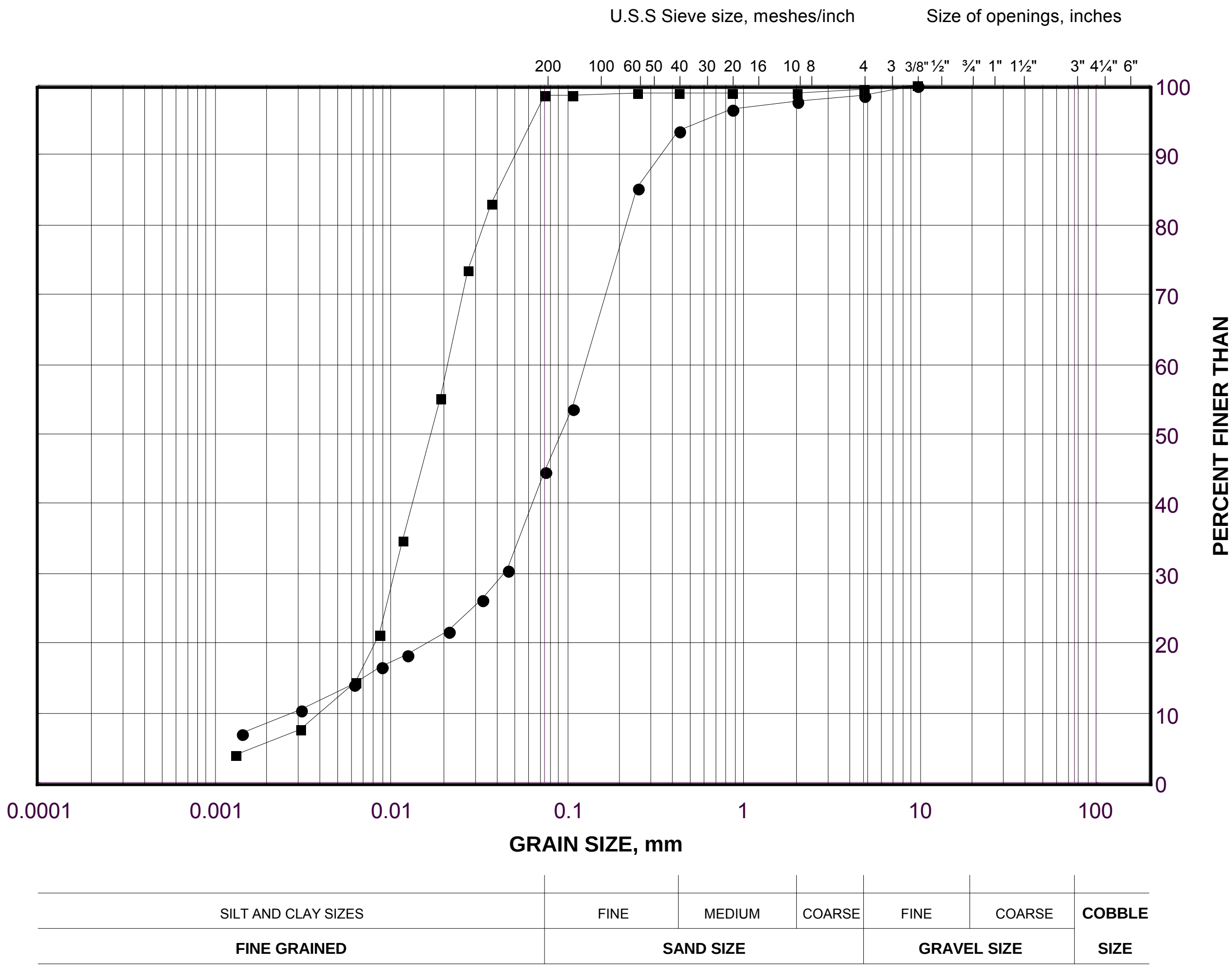
Golder Associates

Date: 13-Jun-11

GRAIN SIZE DISTRIBUTION

Silt to Sand and Silt Till (Lower Deposit)

FIGURE 8



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HN6	9	238.5
■	HN7	9	238.6

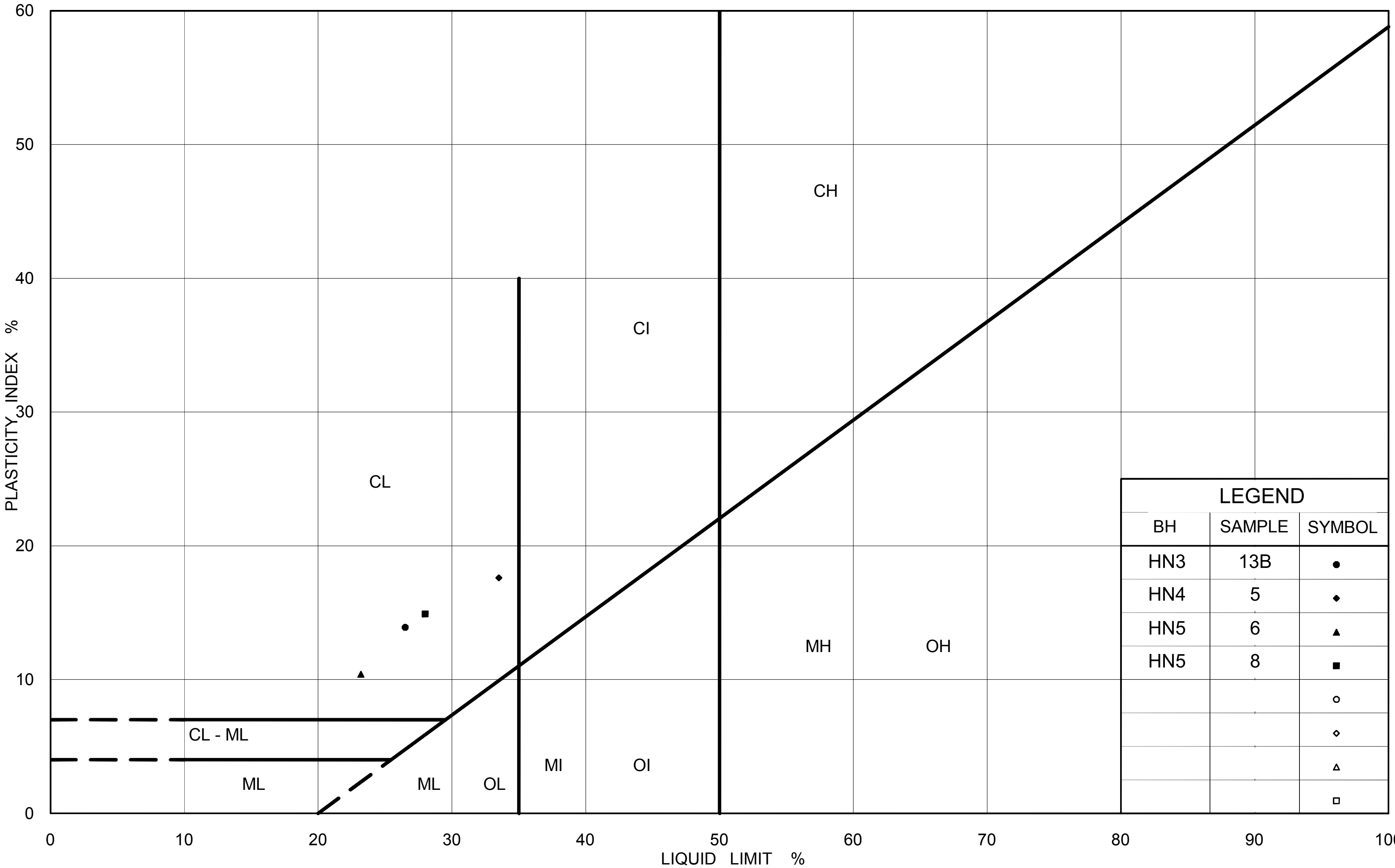
Project Number: 09-1111-0018

Checked By: TVA

Golder Associates

Date: 13-Jun-11

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt Interlayer

Figure No. 9

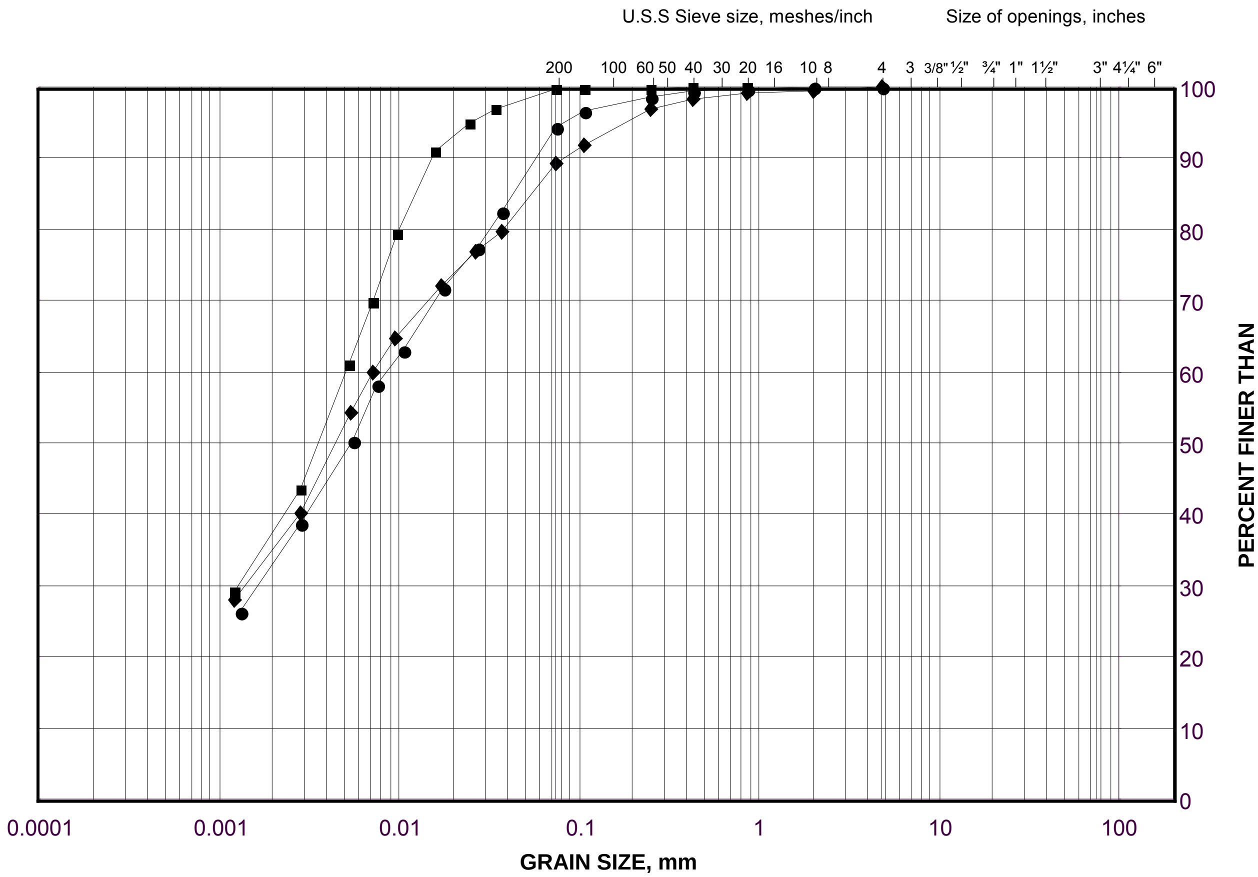
Project No. 09-1111-0018

Checked By: TVA

GRAIN SIZE DISTRIBUTION

Clayey Silt Interlayer

FIGURE 10



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HN3	13B	232.6
■	HN4	5	235.4
◆	HN5	8	232.4

Project Number: 09-1111-0018

Checked By: TVA

Golder Associates

Date: 13-Jun-11

APPENDIX B

Record of Boreholes 97-1 to 97-6, Thurber Engineering Ltd. Report No. 15-64-2, dated May 8 1997.

RECORD OF BOREHOLE No 97-1

1 OF 1 **METRIC**

W.P. 3-95-01 LOCATION Coords: N 4 876 404.618 E 297 256.358 ORIGINATED BY EDK

DIST CR HWY 400 & 9 BOREHOLE TYPE 110mm SOLID STEM AUGERS COMPILED BY DWP

DATUM Geodetic DATE 97.02.01 - CHECKED BY PKC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
247.6					
247.5 / 0.1	TOPSOIL (50mm)	1	SS		
	CLAYEY SILT FILL, with pockets of sand and organics Brown				
246.5 / 1.1	CLAYEY SILT, trace Sand, some clay pockets and thin sand seams Mottled Brown Hard	2	SS	38	
245.0 / 2.5	CLAYEY SILT, TILL, sandy, trace gravel Brown Hard	3	SS	80	
		4	SS	72	
241.5 / 6.1	Becoming Grey	5	SS	60	
		6	SS	69	0 25 53 22
238.1 / 9.4		7	SS	52/.150	

END OF BOREHOLE AT 9.4m
UPON COMPLETION OF DRILLING:
Slough Level = 8.3m

Piezometer installation consists of 19mm diameter schedule 40 PVC pipe, with a 0.76m slotted tip.

WATER LEVEL READINGS

DATE	DEPTH (m)	ELEVATION (m)
97/02/01	9.17	238.39
97/02/10	5.66	241.90
97/02/26	4.93	242.67

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (247, 246, 245, 244, 243, 242, 241, 240, 239); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 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

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 97-2

METRIC

W.P. 3-95-01	LOCATION Coords: N4 876 403.137 E297 238.913		ORIGINATED BY EDK
DIST CR	HWY 400 & 9	BOREHOLE TYPE 110mm SOLID STEM AUGERS	COMPILED BY DWP
DATUM Geodetic		DATE 97.01.28 -	CHECKED BY PKC

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
239.7 / 0.0	**ASPHALT** (75mm)
239.2 / 0.5	**CONCRETE** (125mm) **SAND and GRAVEL FILL** Frozen, Brown CLAYEY SILT TILL, sandy trace gravel Grey Very Stiff to Hard
236.5 / 3.2	laminated silty clay, occasional wet sandy silt partings
236.2 / 3.5	CLAYEY SILT TILL sandy trace gravel Grey Hard
232.4 / 7.3	CLAYEY SILT and SAND TILL trace gravel Grey Hard
230.4 / 9.3	END OF BOREHOLE AT 9.3m

SAMPLES

NUMBER	TYPE	"N" VALUES
~~1~~	~~SS~~	
2	SS	17
3	SS	48
4	SS	52
5	SS	50
6	SS	47
7	SS	58
8	SS	72/.125
9	SS	60/.150

ELEVATION SCALE: 239, 238, 237, 236, 235, 234, 233, 232, 231

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 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
0	31	52	17

GROUND WATER CONDITIONS

UPON COMPLETION OF DRILLING:
Slough Level = 9.0m
Piezometer installation consists of 19mm diameter schedule 40 PVC pipe with a 0.76m slotted tip.

WATER LEVEL READINGS

DATE	DEPTH (m)	ELEVATION (m)
97/01/28	Dry	
97/01/31	4.56	235.15
97/02/10	1.03	238.68
97/02/26	0.91	238.79

+³, ×³: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 97-3

METRIC

W.P. 3-95-01 LOCATION Coords: N4 876 397.931 E297 240.915 ORIGINATED BY EDK

DIST CR HWY 400 & 9 BOREHOLE TYPE 110mm SOLID STEM AUGERS COMPILED BY DWP

DATUM Geodetic DATE 97.01.28 - CHECKED BY PKC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
239.8					
239.6 / 0.2	ASPHALT and CONCRETE (200mm)	1	SS		
239.1 / 0.7	SAND and GRAVEL FILL, Frozen				239
	CLAYEY SILT TILL, sandy, trace gravel, Grey, Stiff to Hard; trace sand	2	SS	13	
		3	SS	39	238
		4	SS	45	237
236.6 / 3.2	CLAYEY SILT, trace sand, laminated, Grey, Hard	5	SS	37	
236.3 / 3.5	END OF BOREHOLE AT 3.5m				

END OF BOREHOLE AT 3.5m
UPON COMPLETION OF DRILLING:
Slough Level = 2.9m
Borehole Dry

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 × LAB VANE 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

+ ³, × ³: Numbers refer to Sensitivity   15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 97-4

METRIC

W.P. 3-95-01 | LOCATION Coords: N4 876 393.443 E297 211.680 | ORIGINATED BY EDK

DIST CR HWY 400 & 9 | BOREHOLE TYPE 110mm SOLID STEM AUGERS | COMPILED BY DWP

DATUM Geodetic | DATE 97.01.28 - | CHECKED BY PKC

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.8					
239.6 / 0.0	ASPHALT (250mm)				
0.2	SAND FILL, some gravel, Brown, Compact	1	SS	14	
238.3 / 1.5	CLAYEY SILT TILL, sandy, trace gravel, Grey, Stiff to Hard	2	SS	13	
		3	SS	31	0 26 53 21
		4	SS	46	
		5	SS	58	
233.7 / 6.1	CLAYEY SILT TILL, trace sand, trace gravel, Grey, Hard	~~6~~	~~SS~~	~~65/~~ .150	
232.5 / 7.3	SILTY SAND, Grey, Very Dense, -free water at 7.8m, N value for sample #7 not representative due to disturbance by water pressure	~~7~~	~~SS~~	~~WH~~	
230.3		8	SS	96	
9.4	END OF BOREHOLE AT 9.4m UPON COMPLETION OF DRILLING: Water Level = 5.2m Slough Level = 6.1m				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (239, 238, 237, 236, 235, 234, 233, 232, 231); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 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.

$+^3, \times^3$: Numbers refer to Sensitivity   15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 97-5

METRIC

W.P. 3-95-01 | LOCATION Coords: N4 876 389.204 E297 212.631 | ORIGINATED BY EDK

DIST CR HWY 400 & 9 | BOREHOLE TYPE 110mm SOLID STEM AUGERS | COMPILED BY DWP

DATUM Geodetic | DATE 97.01.28 - | CHECKED BY PKC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.9									
239.7 / 0.2	ASPHALT (250mm)								
	SAND FILL, some gravel, Brown, Compact	1	SS	11		239			
238.8 / 1.1	CLAYEY SILT TILL, sandy, trace gravel, Grey, Stiff to Hard	2	SS	48		238			
		3	SS	45		237			
236.4 / 3.5		4	SS	47					
3.5	END OF BOREHOLE AT 3.5m UPON COMPLETION OF DRILLING: Slough Level = 3.0m Borehole Dry								

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 97-6

METRIC

W.P. 3-95-01 LOCATION Coords: N4 876 385.447 E297 196.444 ORIGINATED BY EDK

DIST CR HWY 400 & 9 BOREHOLE TYPE 110mm SOLID STEM AUGERS COMPILED BY DWP

DATUM Geodetic DATE 97.01.30 - CHECKED BY PKC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
247.9 0.0	SANDY TOPSOIL						
247.4 0.5	some rootlets and plant matter Brown, Frozen to 0.3m	1	SS				
	CLAYEY SILT AND SAND TILL, trace gravel Brown Very Stiff to Hard				247		
		2	SS	28	246		
244.7 3.2	CLAYEY SILT with clay, silt, and fine sand laminations Brown Hard	3	SS	85/.28	245		
243.4 4.5	Becoming Grey	4	SS	50/.150	244 243		
		5	SS	76/.24	242		0 5 69 27
		6	SS	55	241 240		
239.4 8.5	CLAYEY SILT TILL, sandy trace gravel Grey	7	SS	71	239		
238.3 9.6	Hard						

END OF BOREHOLE AT 9.6m
UPON COMPLETION OF DRILLING:
Borehole Dry with no Slough
Piezometer installation consists of 19mm diameter schedule 40 PVC pipe with a 0.76m slotted tip.

WATER LEVEL READINGS

DATE	DEPTH (m)	ELEVATION (m)
97/01/30	Dry	
97/01/31	5.96	241.95
97/02/10	5.96	241.95
97/02/26	5.97	241.93

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 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

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

20 / 15 ◆ 5 / 10 (%) STRAIN AT FAILURE

APPENDIX C

Non-Standard Special Provisions

UNWATERING FOR FOUNDATION EXCAVATION - Item No.

Non-Standard Special Provision

The contractor shall be alerted that high groundwater table was encountered at the proposed Highway 9 Bridge site over Highway 400 widening. It is estimated that the base of temporary excavations for the foundations may be up to 5 m below the groundwater level as measured in a piezometers installed in Boreholes 97-1, 97-6 and HN7. The subsoil conditions generally consist of clayey silt /clayey silt till containing confined water-bearing sand and silt till to silt tills. Construction of shallow foundations / pile caps must be carried out in the dry. Dewatering within the foundation excavations will be required and the excavation shall be kept stable during the work. It is considered that a combination of adequately sized pumped pressure relief wells and perimeter ditches / trenches is required to lower the groundwater.

Basis of Payment

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

END OF SECTION

SUBGRADE PROTECTION - Item No.

Non-Standard Special Provision

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

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

Basis of Payment

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

END OF SECTION

GROUND AND GROUNDWATER CONTROL DURING CAISSON INSTALLATION - Item No.

Non-Standard Special Provision

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

Basis of Payment

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

END OF SECTION

OBSTRUCTIONS - Item No.

Non-Standard Special Provision

Grinding of augers was encountered at the site as indicated in the Record of Borehole sheet HN3. Consideration of the presence of these obstructions possibly as a result of presence of cobbles or boulders must be made in the selection of appropriate equipment and procedures for driving Steel H-Piles or caissons and pre-augering for deep foundations.

Basis of Payment

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

END OF SECTION

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

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