July 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

South Canal Bridges
Highway 400 Widening from North of King Road to North of South Canal Road, Regional Municipality of York
GWP 2025-13-00

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REPORT

Golder Associates

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PART A

FOUNDATION INVESTIGATION REPORT
SOUTH CANAL BRIDGES
HIGHWAY 400 WIDENING
FROM NORTH OF KING ROAD TO SOUTH CANAL ROAD
REGIONAL MUNICIPALITY OF YORK
GWP 2025-13-00

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 foundation engineering services in support of the detail design of the widening of Highway 400 from north of King Road to north of South Canal Road in the Regional Municipality of York, Ontario.

This report addresses the results of the foundation investigation carried out for the replacement of the existing northbound and southbound Highway 400-South Canal bridges. The current investigation was supplemented with information from a previous investigation at this structure site, as follows:

- **MTO GEOCRES No. 31D-029**: Report titled “Foundation Investigation Report for Proposed Extensions to the Overpass Structures at the Crossing of Hwy. #400 and the South Drainage Canal and Road, Township of King – County of York, District No. 6 (Toronto), W.O. 7C-11089 – W.P. 105-70-04”, by the Department of Highways Ontario (DHO), Foundations Section, Materials and Testing Office, dated December 8, 1970.

The previous boreholes as used in this report have been renumbered to show the MTO GEOCRES reference number followed by the original borehole designation. For this site, the boreholes from MTO GEOCRES 31D-029 have been renumbered to “29-X”, where “X” is the original borehole number.

The terms of reference and scope of work for the foundation investigation are outlined in MTO’s Request for Proposal (RFP) dated May 2008, and MTO’s revised Terms of Reference in an Addendum dated October 14, 2011. 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 existing South Canal bridges are located approximately 0.5 km north of Highway 9 in King Township, in the Regional Municipality of York. The South Canal bridges span over an approximately 18 m wide excavated canal and South Canal Bank Road. Both bridges consist of six-span structures constructed in 1948, with the original structures supported on driven timber piles. The bridges were widened toward the outside in 1971, with the widened portion supported on driven steel H-piles.

In general, the terrain at the South Canal site and in the adjacent Holland Marsh is relatively flat-lying, at approximately Elevation 219 m to 220 m. The natural ground surface rises immediately to the south of the structure site, to the higher “tableland” of the Oak Ridges Moraine. At the structure site, Highway 400 has been constructed on embankment fill approximately 5 m to 7 m high, with the highway grade rising toward the south from approximately Elevation 226 m to 228 m.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out in November 2011, May 2012 and June 2012, during which time thirteen boreholes (Boreholes SC-1 to SC-5, SC-7 to SC-11, SC-13, SC-14, and BO-9) were drilled associated with the NBL and SBL bridges, supplemented with four boreholes (Boreholes OHS7, F8-2, F8-3 and F8-6) drilled in January, March and April 2011 associated with an adjacent overhead sign and fill embankments. The boreholes were advanced using a CME-55 truck-mounted drill rig, D-25 track-mounted drill rig, and a D-90 truck-mounted drill rig supplied and operated by DBW Drilling Ltd. of North York, Ontario and Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using 108 mm inside diameter hollow stem augers and 76 mm outer diameter wash boring using N-size casing.

Soil samples were obtained at 0.75 m and 1.5 m intervals of depth in the boreholes, using a 50 mm outside diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure *(ASTM D1586-08a – Standard Test Method for Standard Penetration Test).* In situ field vane testing, using MTO standard "N"-sized vanes, was carried out in the soft to stiff portions of cohesive soils, where encountered, to measure the undrained shear strength of the deposit. Thin-walled Shelby tube samples (ASTM) were also taken within the cohesive materials at selected intervals.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and a standpipe piezometer was installed in one borehole (Borehole SC-1) to permit monitoring of the groundwater level. The piezometer consist of 50 mm diameter PVC pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled to the ground surface with bentonite pellets. The piezometer installation details and water level readings are indicated on the record for Borehole SC-1 contained in Appendix A. Boreholes where artesian groundwater conditions were noted were backfilled with cement grout and all other boreholes were backfilled with bentonite, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member of Golder's staff who located the boreholes in the field, cleared these locations for underground utilities, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits and grain size distributions were carried out on selected soil samples. In addition, three one-dimensional consolidation (oedometer) tests were carried out on selected samples of the clayey silt to silt deposit.

The borehole locations were established in the field by Golder personnel relative to site features. The ground surface elevation at each borehole was estimated from the digital terrain model for the site as provided by URS. The borehole locations (referenced to the MTM NAD83 co-ordinate system) and ground surface elevations (referenced to Geodetic datum) are summarized below and are shown on Drawing 1. Drawing 1 also shows the locations of boreholes advanced as part of the previous investigation at the site.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
SC-1	4,877,070.0	297,189.1	223.0	12.8
SC-2	4,877,082.3	297,188.1	222.0	17.4
SC-3	4,877,124.8	297,177.2	220.1	17.2
SC-4	4,877,151.8	297,171.4	220.8	27.9
SC-5	4,877,176.1	297,165.0	221.1	15.9
SC-7	4,877,117.8	297,113.1	220.7	40.1

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
SC-8	4,877,130.1	297,103.5	220.5	12.8
SC-9	4,877,070.2	297,116.5	221.0	20.4
SC-10	4,877,033.5	297,122.5	222.1	15.9
SC-11	4,877,019.1	297,122.9	221.8	14.3
SC-13	4,877,052.4	297,161.4	229.0	30.9
SC-14	4,877,041.9	297,120.6	222.0	18.1
BO-9	4,877,161.8	297,169.1	221.0	26.5
OHS7	4,877,043.3	297,113.2	220.4	6.7
F8-2	4,877,031.6	297,183.6	229.2	15.8
F8-3	4,877,098.8	297,187.5	221.0	6.7
F8-6	4,877,028.4	297,140.7	229.1	17.2
29-1	4,877,089.0	297,166.0	227.4	18.4
29-2	4,877,083.0	297,134.0	223.9	18.3
29-3	4,877,114.0	297,164.0	220.6	13.9
29-4	4,877,119.0	297,129.0	221.1	16.9
29-5	4,877,149.0	297,157.0	221.1	20.0
29-6	4,877,146.0	297,122.0	225.8	20.3
29-8	4,877,128.0	297,143.0	221.1	20.3
29-9	4,877,092.9	297,183.9	221.1	7.0

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). 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 the 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, cuts exposed up to about 10 m of till overlying the sand and gravel deposits.

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 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

As part of the current subsurface investigation, thirteen boreholes (Boreholes SC-1 to SC-5, SC-7 to SC-11, SC-13, SC-14, and BO-9) were advanced at the South Canal bridges site, supplemented with three boreholes (F8-2, F8-3 and F8-6) drilled for adjacent high fill embankment areas, and one borehole (OHS7) drilled for a nearby overhead sign. The borehole locations, ground surface elevations and interpreted stratigraphic conditions are shown on Drawings 1 to 4.

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigations and the results of in situ and laboratory testing are given on the borehole records contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B14 contained in Appendix B. The borehole information from the previous MTO investigations is presented in Appendix C.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile and cross-sections on Drawings 2 to 4 are inferred observations of drilling progress and from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the site consist of surficial layers of topsoil, asphalt and roadway base granular fill and cohesive fill in the vicinity of Highway 400, underlain by a clayey silt deposit containing silty sand to sandy silt interlayers. The clayey silt deposit is underlain by a clayey silt till deposit in places underlain by or interlayered with a sand and silt till deposit. A silty sand to sand and silt to sand and gravel deposit with clayey silt interlayers underlies the till deposits.

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

4.2.1 Topsoil

Approximately 100 mm to 200 mm of topsoil was encountered immediately below the existing ground surface in Boreholes BO-9, SC-1 to SC-3, SC-8 to SC-11, SC-14 and F8-3 which were drilled at the toes of the Highway 400 embankment.

4.2.2 Fill

Approximately 100 mm to 300 mm of asphalt was encountered immediately below the existing ground surface in Borehole SC-5, which was drilled on Wist Road east of Highway 400, and at Boreholes F8-2 and F8-6 drilled on the shoulders of Highway 400. An approximately 0.6 m to 10.7 m thick layer of fill was encountered immediately below ground surface in Boreholes SC-4, SC-7, SC13 and OHS7, below the asphalt in Boreholes SC-5, F8-2 and F8-6 and underlying the topsoil in Boreholes SC-1, SC-3, SC-11 and F8-3. The base of the fill layer was encountered between Elevation 223.6 m and 218.3 m.

The fill generally consists of an upper layer of non-cohesive material, where present, underlain by cohesive material. The non-cohesive fill consists of sand and silt to sandy silt containing trace to some gravel and trace clay, and sand and gravel containing some silt and trace clay. Organic matter, rootlets and wood fragments were noted in the non-cohesive fill in Boreholes SC-1, SC-3 and SC-4. The cohesive fill consists of clayey silt containing trace to some sand and trace to some gravel. Organic matter, rootlets and wood fragments were noted in the cohesive fill in Boreholes SC-3, SC-4, SC-7 and SC-11. The results of grain size distribution tests completed on two selected samples of the non-cohesive fill and three selected samples of the cohesive fill are shown on Figures B1A and B1B, respectively, in Appendix B.

Atterberg limits testing was carried out on seven selected samples of the cohesive fill, and measured plastic limits ranging from 13 per cent to 16 per cent, liquid limits ranging from 18 per cent to 26 per cent, and plasticity indices ranging from 5 per cent to 11 per cent. These results, which are plotted on Figure B2 in Appendix B, confirm that the cohesive fill consists of clayey silt of low plasticity. The natural water content measured on selected samples of the cohesive fill ranges from about 10 to 26 per cent.

The measured Standard Penetration Test (SPT) “N”-values within the non-cohesive fill range from 4 blows to 34 blows per 0.3 m of penetration, indicating a loose to dense relative density. The measured SPT “N”-values within the cohesive fill range from 4 blows to 47 blows per 0.3 m of penetration, suggesting a firm to hard consistency.

4.2.3 Peat/Organic Sandy Silt

An approximately 0.1 m to 1.5 m thick layer of peat or organic sandy silt was encountered below the fill in Boreholes SC-4, SC-5, SC-7, SC-11, OHS-7 and F8-6, and below the clayey silt deposit in Boreholes BO-9, SC-1, SC-2, SC-8, SC-10 and SC-14. The base of the peat/organic layer was encountered between Elevation 220.3 m and 218.2 m.

The water content and organic content of one tested sample of the organic sandy silt is about 32 percent and 7 percent respectively.

The measured SPT “N”-values within the peat/organic deposit range from 2 blows to 12 blows per 0.3 m of penetration, suggesting a soft to stiff consistency.

4.2.4 Clayey Silt (Upper Deposit)

An approximately 0.4 m to 12.2 m thick deposit of clayey silt was encountered underlying the topsoil, fill, peat/organics or granular interlayers in all boreholes, except Borehole F8-6. Boreholes SC-8 and OHS7 terminated within this deposit, penetrating the clayey silt deposit for a thickness of 9.1 m and 1.1 m, respectively. The base of the deposit was encountered between Elevation 222.0 m and 206.2 m.

The deposit consists of clayey silt with sand to trace sand, containing trace to some gravel. Organic matter, wood fragments and rootlets were encountered in the upper portions of the deposit in Boreholes F8-2, SC-1, SC-4, SC-7, SC-9, SC-10 and SC-14. Silty sand and sand lenses and seams were also encountered in Boreholes SC -2, SC-4 and SC-5. The results of grain size distribution testing completed on nineteen selected samples of the deposit are shown on Figure B3A to B3C in Appendix B.

Atterberg limits testing was carried out on twenty-six selected samples of the deposit and measured plastic limits of 9 per cent to 18 per cent, liquid limits of 15 per cent to 31 per cent, and plasticity indices that are generally over 5 per cent to 15 per cent. These results, which are plotted a plasticity chart on Figures B4A to B4D in Appendix B, confirm that the deposit consists of clayey silt of low plasticity. However, two of the tested samples have plasticity indices of approximately 3 per cent, and these results indicate that portions of the deposit grade to a slightly plastic silt. The natural water content measured on selected soil samples ranges from about 14 to 27 per cent.

The measured SPT "N"-values within the clayey silt deposit range from 0 blows (weight of hammer) to 44 blows per 0.3 m of penetration. Field vane tests measured undrained shear strengths ranging from about 30 kPa (but generally greater than 40 kPa) to greater than 120 kPa, and sensitivities between about 1.5 and 6. The SPT "N"-values together with the vane undrained shear strength results indicate that the deposit is firm to very stiff in consistency.

Laboratory consolidation tests were carried out on three thin-walled Shelby tube samples of the clayey silt deposit. The consolidation test results are presented on Figures B5 to B7 in Appendix B, and are summarized below.

Borehole/ Sample No.	Sample Depth/Elev. (m)	Unit Weight (kN/m^3)	σ_{vo}' (kPa)	σ_p' (kPa)	σ_p' - σ_{vo}' (kPa)	OCR	e_o	C_c	C_r	C_v* (cm^2/s)
SC-3 / S1	5.6 / 214.5	20.7	50	280	230	5.6	0.64	0.12	0.025	$3.5x10^{-3}$
SC-5 / T1	12.4 / 208.7	20.1	112	135	23	1.2	0.64	0.16	0.022	$2.1x10^{-3}$
SC-7 / 11	11.0 / 209.7	20.6	99	150	51	1.5	0.63	0.16	0.026	$1.1x10^{-3}$

where σ_p' Estimated preconsolidation stress; σ_{vo}' Computed existing vertical effective stress
C_c Compression index; C_r Recompression index
e_o Initial void ratio; OCR Overconsolidation ratio
C_v Coefficient of consolidations in the normally consolidated range, for approximate stress range 20 kPa ≤ σ_v' ≤ 150 kPa

4.2.5 Silty Sand to Sandy Silt Interlayers

A number of silty sand to sandy silt interlayers, approximately 0.1 m to 3.0 m thick were encountered within the clayey silt deposit in all boreholes.

The interlayers consist of sandy silt to silty sand to sand and silt to sand, in places containing trace gravel and trace clay. The results of grain size distribution testing completed on six selected samples of the sandy silt to silty sand interlayers are shown on Figure B8 in Appendix B.

The measured SPT "N"-values within the sandy silt to silty sand interlayers range from 2 blows to 35 blows per 0.3 m of penetration, but are generally between 10 and 30 blows per 0.3 m of penetration, indicating a very loose to dense relative density.

4.2.6 Clayey Silt Till to Sandy Silt/Silty Sand Till

A till deposit was encountered underlying the clayey silt deposit in all boreholes, except Boreholes BO-9 and F8-6. Boreholes SC-2 SC-5, SC-10, SC-11, SC-13 and SC-14 terminated within this deposit, penetrating it for a thickness of 1.0 m to 11.4 m. The thickness of the till deposit is 1.3 m to 10.9 m in the remaining boreholes. Where the deposit was fully penetrated, the base of the deposit was encountered between Elevation 208.4 m and 200.6 m, but it extended to at least Elevation 198.1m where it was not fully penetrated.

The till deposit generally consists of clayey silt with sand to trace sand, containing trace gravel. In Boreholes SC-2, SC-4 and F8-6, it grades into or is comprised of sandy silt to silty sand containing trace to some clay and trace gravel. Cobbles and boulders were also encountered within the till deposit, as inferred from grinding augers and/or the split-spoon sampler bouncing during sampling operations; these instances are noted on the borehole records contained in Appendix A. The results of grain size distribution testing completed on sixteen selected samples of the clayey silt till to sandy silt to silty sand till are shown on Figures B9A to B9C in Appendix B.

Atterberg limits testing was carried out on twenty-one selected samples of the clayey silt portion of the till and two samples of the sandy silt portion of the till deposit, and measured plastic limits ranging from 9 per cent to 12 per cent, liquid limits ranging from 15 per cent to 21 per cent, and plasticity indices ranging from 3 per cent to 11 per cent. These results, which are plotted on plasticity charts on Figures B10A to B10C in Appendix B, confirm that the cohesive portion of the till deposit consists of clayey silt of low plasticity and the fine portion of the “non-cohesive” till consists of silty sand to sandy silt of slight plasticity. The natural water content measured on selected samples of the clayey silt portion of the till ranges from 7 to 25 per cent, but is typically in the range of 7 to 13 per cent; the natural water content measured on selected samples of the sandy silt to silty sand portion of the till ranges from 4 to 15 per cent.

The measured SPT “N”-values within the clayey silt till deposit range from 6 blows per 0.3 m of penetration to 121 blows per 0.3 m of penetration, but are generally over 25 blows per 0.3 m of penetration, with one “N”-value of 133 blows per 0.23 m of penetrations. These SPT “N”-values suggest that the consistency of the clayey silt till deposit varies from firm to hard, but is generally very stiff to hard. The measured SPT “N’-values within the sandy silt portion of the till deposit range from 55 blows to 108 blows per 0.3 m penetration, with one “N” value of 23 blows per 0.3 m of penetration at the interface with the overlying organic sandy silt deposit, indicating that this portion of the till deposit generally has a very dense relative density.

4.2.7 Sand and Silt to Sand and Gravel

A non-cohesive soil deposit varying in composition from sand and silt to sand and gravel was encountered underlying the till deposit and/or interlayered within the till deposit in Boreholes BO-9, SC-3, SC-4, SC-7, SC-9, and SC-13. The thickness of the granular deposit and interlayers ranges from 1.2 m to at least 18.8 m. Boreholes SC-3, SC-4, SC-7, SC-9 and BO-9 terminated within this deposit. The base of the granular deposit/interlayers was encountered between Elevation 202.9 m and 180.6 m.

The deposit varies in composition from sand and silt to silty sand containing trace to some clay and trace to some gravel, to sand and gravel containing some silt and trace clay. The results of grain size distribution testing completed on twelve selected samples of the silty sand to sand and silt to sand and gravel deposit and interlayers are shown on Figures B11A and B11B in Appendix B.

Atterberg limits testing was carried out on two selected samples of the sand and silt portion of the deposit, to check whether these finer soils would behave as plastic materials. This testing measured plastic limits of 16 per cent and 17 per cent, liquid limits of 20 per cent, and plasticity indices of 3 per cent and 4 per cent. These results, which are plotted on Figure B12 in Appendix B, indicate that some of the fine portions of the sand and silt deposit consists of silt of slight plasticity; however, overall this deposit is expected to behave as non-plastic.

The measured SPT “N”-values within this deposit range from 11 blows (but generally over 60 blows) to 112 blows per 0.3 m of penetration, with three SPT “N”-values greater than 100 blows per 0.08 m of penetration. These SPT “N”-values indicate that the deposit has a compact to very dense, but generally very dense, relative density.

4.2.8 Clayey Silt Interlayers (Lower Deposit)

Approximately 0.9 m to 6.1 m thick clayey silt interlayers were encountered within the sand and silt to sand and gravel deposit in Boreholes BO-9, SC-4 and SC-7. The base of these interlayers was encountered between Elevation 196.2 m and 182.3 m.

The interlayers consist of clayey silt with sand to trace sand, containing trace to some gravel. Silt and silty sand seams were observed within these interlayers in Boreholes SC-4 and SC-7. The results of grain size distribution testing completed on two selected samples of the clayey silt interlayers are shown on Figure B13 in Appendix B.

Atterberg limits testing was carried out on two selected samples of the clayey silt interlayers and measured plastic limits of 11 per cent and 15 per cent, liquid limits of 19 per cent and 30 per cent and corresponding plasticity indices of 8 per cent and 15 per cent. These results, which are plotted on a plasticity chart on Figure B14 in Appendix B, confirm that the interlayers consist of clayey silt of low plasticity. The natural water content measured on selected samples of this lower clayey silt deposit ranges from about 13 to 22 per cent.

The measured SPT “N”-values within the clayey silt interlayers range from 52 blows to 100 blows per 0.3 m of penetration, suggesting a hard consistency.

4.3 Groundwater Conditions

The observed/recorded water levels in the open boreholes following completion of drilling and in the standpipe piezometers installed in Borehole SC-1 are shown on the borehole records and are summarized as follows:

Borehole No.	Approximate Depth/Elevation at which Artesian Groundwater Pressures Encountered (m)	Approximate Depth to Water Level* (m)	Approximate Groundwater Elevation (m)	Date
SC-1	-	2.8 0.3	222.7	Jun 11, 2012 (Completion of drilling) Jun 12, 2012 (Piezometer)
SC-2	13.7/208.3	+2.0 ags*	224.0	Jun 8, 2012 (Completion of drilling)
SC-3	11.7/208.4	+3.6 ags*	223.7	May 25, 2012 (Completion of drilling)
SC-4	22.9/197.9	Not Recorded	Not Recorded	-
SC-5	15.2/205.9	4.2	216.9	Nov 15, 2012 (Completion of drilling)
SC-7	15.8/205.0	+1.5 ags*	222.2	Nov 9, 2012 (During casing removal)
SC-8	-	5.5	215.0	Nov 7, 2012 (Completion of drilling)
SC-9	18.3/202.7	+4.1 ags*	225.1	May 16, 2012 (Completion of drilling)
SC-10	-	2.1	220.0	May 14, 2012 (Completion of drilling)
SC-11	-	Dry	-	May 11, 2012 (Completion of drilling)
SC-13	-	1.0	228.0	Jun 7, 2012 (Completion of drilling)
SC-14	-	3.7	218.3	May 22, 2012 (Completion of drilling)
BO-9	25.9/195.1	1.6	222.6	Nov 15, 2011
OHS7	-	1.1	219.3	Apr 6, 2012 (Completion of drilling)
F8-2	-	15.2	214.0	Apr 1, 2011 (Completion of drilling)
F8-3	-	3.2	217.8	Jan 18, 2011 (Completion of drilling)
F8-6	-	14.9	214.2	Mar 31, 2011 (Completion of drilling)
29-1	-	3.7	223.7	Nov 17, 1970
29-2	-	1.5	222.2	Nov 17, 1970
29-3	-	0.0	220.7	Nov 10, 1970
29-4	-	1.9	219.2	Nov 17, 1970
29-5	-	1.3	219.5	Oct 20, 1970
29-6	-	3.7	222.2	Nov 19, 1970
29-8	-	1.6	219.8	Nov 17, 1970
29-9	-	1.2	219.9	Nov 19, 1970

NOTES: * ags = above ground surface

The water levels observed in the open boreholes on completion of drilling may not represent long-term stabilized groundwater levels. The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.

Artesian groundwater conditions (i.e., groundwater levels above the ground surface at the site) were encountered within the lower non-cohesive deposit primarily in boreholes on the east side of the bridge structures on both sides of the canal. The artesian conditions were encountered at the time of borehole drilling at depths between 11.7 m and 25.9 m below ground surface, corresponding to approximately Elevation 205.9 m to 195.1 m in the northeast quadrant, approximately Elevation 208.5 m to 208.0 m in the southeast quadrant, and approximately Elevation 205.0 m to 202.5 m on the west side of the bridge structures. The groundwater level in the casing rose to between 1.5 m and 4.1 m above ground surface, to between about Elevation 225 m and 222 m. In boreholes where artesian groundwater conditions were not encountered associated with the lower non-cohesive deposit, the observed water level varied from approximately Elevation 228 m to 218 m. The approximate depth to and elevation of the surface of the artesian groundwater stratum is summarized below:

Foundation Element	Approximate Depth to Surface of Stratum Where Artesian Pressures Encountered (m)	Approximate Elevation of Surface of Stratum Where Artesian Pressures Encountered (m)
North abutment – NBL	22.9 to 25.9	195 to 198
North abutment – SBL	15.8	205
Centre pier	11.7 to 18.3	208 to 203
South abutment – NBL	13.7	208
South abutment – SBL	-	-

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Nikol Kochmanová, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Jorge Costa, P.Eng., a Principal with Golder and Designated MTO Foundations Contact, conducted an independent review of this report.

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PART B

FOUNDATION DESIGN REPORT
SOUTH CANAL BRIDGES
HIGHWAY 400 WIDENING
FROM NORTH OF KING ROAD TO SOUTH CANAL ROAD
REGIONAL MUNICIPALITY OF YORK
GWP 2025-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation recommendations for the detail design of the proposed replacement and widening of the northbound and southbound lanes of the Highway 400 bridges over the realigned South Canal and South Canal Bank Road. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current and previous subsurface investigations at this site. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out detail design of the foundations for the proposed structure replacement and widening.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on 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.

6.2 Foundation Options

As part of the future widening of Highway 400 from north of King Road to north of South Canal Road in the Regional Municipality of York, and as part of the southward realignment of the existing South Canal channel, the existing six-span South Canal bridges will require replacement. Based on the General Arrangement drawings provided by URS, the existing structures are proposed to be replaced with two-span bridges along the same highway alignment; however, the new bridges are to be shifted approximately 35 m to the south to accommodate the southward re-alignment of the South Canal channel and the adjacent South Canal Road. Highway 400 is proposed to be widened by approximately 8 m to 15 m on each side. Retaining walls, approximately 5.5 m to 7 m high, are proposed adjacent to the abutments/wingwalls, parallel to Highway 400.

6.2.1 South Canal Bridges

The existing NBL and SBL bridges consist of six-span structures constructed in 1948, with the original structures supported on 12 m to 18 m long driven timber friction piles. The bridges were widened toward the outside in 1971, with the widened portion supported on steel H-piles driven to "practical refusal" within the very dense/hard till deposit.

Based on the proposed bridge geometry and the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments and centre pier for the new South Canal bridges. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip or spread footings:** Shallow foundations are not considered suitable or feasible for support of the structures due to the presence of relatively weak (firm) clayey silt at shallow depths.

- **Driven steel H-piles or pipe (tube) piles driven to/terminating on the till deposit:** Friction piles that terminate within the till deposit above the surface of the artesian stratum could be considered for the support of the abutments, associated wingwalls, and the centre piers at this site. This option would avoid or reduce some of the risks associated with deeper, end-bearing piles, such as the potential for loss of fine soil particles along the pile shafts or negative impacts on the aquifer and nearby wells. However, the geotechnical resistance for friction piles terminating in the clayey silt deposit above the aquifer will be relatively low, and may not be practical/sufficient for support of the new bridges.

- **Driven steel H-piles or pipe (tube) piles driven to practical refusal within the till deposit:** Driven steel H-piles or steel pipe (tube) piles driven to practical refusal within the till deposit, penetrating into the artesian groundwater zone, are feasible for the support of the abutments, associated wingwalls, and the centre piers at this site. This foundation type will offer a higher geotechnical resistance than friction piles terminated above the artesian zone of the aquifer. However, the piles' penetration close to or into the artesian stratum does carry some risks, including the potential for upward migration of fine soil particles along the pile shafts, and potential impacts on the aquifer and nearby water wells. The risk of migration of fine soil particles along the pile shafts could be mitigated with a granular drainage blanket below the pile caps, or potentially other treatments such as grouting.

- **Caissons:** Caissons are feasible for the support of the abutments. However, this foundation option is expected to be more difficult to construct than driven pile foundations, due to the potential for soil squeeze of the firm clayey silt deposit, the potential for loss of ground in water-bearing soils, and the potential for basal heave due to the artesian groundwater pressures. It is anticipated that permanent liners extending the full depth to the founding stratum would be required to protect the open caisson hole from soil squeeze and base/side wall loosening due to the groundwater pressure.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments and center pier for the replacement bridges on driven, end-bearing piles.

6.2.2 Retaining Walls

It is understood that the replacement of the existing South Canal bridges will require new retaining walls adjacent to the north and south abutments and wingwalls, oriented parallel to Highway 400 along the outside edges of the widened northbound and southbound lanes, and approximately 5.5 m to 7 m high. Due to the presence of relatively weak (firm) silty clay to clay soils underlying the site, settlement is anticipated in the soils underlying the widened portions of the embankment and therefore underlying the proposed new retaining walls.

Based on the proposed retaining wall geometry and the subsurface conditions at this site, various wall and foundation types have been considered. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 2 following the text of this report.

- **Conventional concrete retaining walls:** Concrete walls supported on deep foundations (driven piles or caissons) are considered feasible from a geotechnical/foundations perspective. The use of shallow foundations (strip or spread footings) is not considered suitable or feasible due to the presence of relatively weak (firm) clayey silt at shallow depths at this site and the potential for significant settlement under the widened Highway 400 embankment loading.

- **Reinforced soil system (RSS) walls:** RSS walls are geotechnically feasible for the proposed retaining walls adjacent to the abutments at this site. It is anticipated that the RSS walls can be constructed in conjunction with the widening of the existing Highway 400 embankment slopes. Temporary steepening of the existing Highway 400 embankment side slopes will be required in conjunction with the use of temporary protection systems parallel to Highway 400 to construct the reinforced soil mass. The existing peat/organic soils will be subexcavated to promote global stability of the retaining walls and minimize settlement associated with the organic deposits; however, due to the presence of weak/soft, compressible soils underlying the proposed embankment widening/retaining wall, total settlements on the order of 300 mm to 350 mm are anticipated if conventional fill materials are used. Various settlement mitigation methods, such as preloading, surcharging, wick drains and/or use of lightweight fill, have been considered through the detail design stage of this project. The preferred alternative involves the use of lightweight fill materials to reduce the total magnitude of settlement (as discussed further in Section 6.6), in conjunction with a two-stage RSS wall to preload the area, then affix the facing panels once the majority of settlement has been completed to maintain the aesthetic appearance of the wall face.
- **Soldier pile and concrete panel wall:** A soldier pile and concrete panel wall is not considered applicable at this site as this type of wall is generally more advantageous in “top-down” construction applications (i.e., as part of a cut widening, rather than for an embankment widening). In addition, this type of wall may not satisfy aesthetic requirements.

Based on the above considerations, RSS walls are considered to be the most practicable and cost-effective option from a geotechnical/foundations perspective for the new retaining walls at this site. However, as noted above and as discussed further in Section 6.6.3, it is estimated that total settlements of up to approximately 300 mm to 350 mm will occur under the widened Highway 400 embankment loading if conventional earth and granular fill materials are used. Settlement mitigation techniques are required, such as preloading and/or surcharging the RSS wall area (with the use of a two-stage RSS wall system, to eliminate the requirement for reconstruction of the preloaded/surcharged area with the reinforced soil mass), in conjunction with lightweight fill materials to reduce the total magnitude of settlement.

Alternatively, to minimize the requirements for preloading/surcharging or other settlement mitigation options within the retaining wall areas, concrete or RSS walls supported on deep foundations are considered to be a technically feasible option from a geotechnical/foundations perspective. This option would have a shorter construction timeline (i.e., no need to wait for a preloading/surcharging period to be completed or to remove preload/surcharge fills), and a lower risk around settlement.

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

Driven steel H-pile or steel pipe (tube) pile foundations are feasible for support of the abutments and center pier for the replacement bridges, as well as associated wingwalls and retaining walls. For the installation of either friction or end-bearing piles, consideration must be given to the artesian groundwater pressures associated with the deep granular deposit below the clayey soils and the till deposit.

As described in Section 4.3, a stratum with artesian groundwater pressures was encountered between approximately Elevation 208 m and 203 m in the vicinity of the south abutment, centre pier and Highway 400 SBL north abutment, and between about Elevation 195 m and 198 m in the vicinity of the Highway 400 NBL north abutment. For comparison, the depth to practical refusal, defined as soils having Standard Penetration Test (SPT) “N” values greater than 100 blows per 0.3 m of penetration, is generally similar to or deeper than the stratum in which artesian groundwater pressures were observed, as summarized in the following table:

Foundation Element	Approximate Depth to Practical “Refusal” (“N”>100 blows per 0.3 m of penetration)	Approximate Elevation of Practical “Refusal” (“N”>100 blows per 0.3 m of penetration)
North abutment	15 m to 28 m	206 m to 193 m
Centre pier	13 m to 20 m	208 m to 201 m
South abutment	13.5 m to 31 m	209.5 m to 198 m

Driven pile foundations for support of the abutments and centre pier could consist of either friction piles that terminate above the surface of the artesian stratum, or end-bearing piles driven to practical refusal within the “100-blow” soils. If end-bearing piles are adopted, their penetration into the artesian stratum could result in a number of impacts and issues including the following:

- Potential upward migration of fine soil particles along the pile shafts, with resulting potential for settlement of the piles. This can be mitigated by the use of granular filters/drainage blankets under the pile caps and/or potentially with the use of grouting, as discussed further in Section 6.8 (Construction Considerations).
- Potential impacts on the aquifer, including quantity and/or quality impacts on nearby water wells. In this regard, it is understood that the property owner located at approximately Station 10+110 on Wist Road reportedly experienced a decline in well capacity following pile driving during the 1971 structure widening.

For either friction or end-bearing piles, steel HP 310x110 piles or 324 mm (12 ¾ in.) diameter, 9.5 mm (⅜ in.) wall thickness closed-end pipe piles have been considered for supporting the new abutments and the centre piers. Either conventional, integral (using steel H-piles) or semi-integral (using steel pipe piles) abutments are applicable at this site.

6.3.1 Founding Elevations

The pile caps for the new abutments should be founded at a minimum depth of 1.5 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Penetration Depths for Southern Ontario)*.

6.3.1.1 *Friction Piles*

For design of friction pile foundations, driven to or terminating within the till deposit approximately 3 m above the surface of the artesian stratum, the pile tip elevations provided below may be used:

Foundation Element	Friction Pile Design Tip Elevation (m)
North Abutment – NBL and SBL	208
Center Pier – NBL and SBL	210
South Abutment – NBL and SBL	210

6.3.1.2 *End-Bearing Piles*

Alternatively, provided that appropriate measures are adopted to mitigate risks associated with penetrating into the artesian groundwater zone, end-bearing piles could be driven into the “100-blow” soil at the following design tip elevations:

Foundation Element	End-Bearing Pile Design Tip Elevation (m)
North Abutment – NBL	204
North Abutment – SBL	203
Center Pier – NBL and SBL	204
South Abutment – NBL	200
South Abutment – SBL	200

Consideration must be given to the potential presence of cobbles and boulders within the glacially-derived soils at this site, as inferred from auger grinding and hammer bouncing during split-spoon sampling in some boreholes. The piles should be reinforced at the tip to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 *(Deep Foundations)*. In very dense/hard and/or bouldery soils, as may be encountered at greater depths at this site, driving shoes (such as Titus Standard “H” Bearing Pile Points) are preferred over flange plates. If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II *(Steel Tube Pile Driving Shoe).*

As discussed further in Section 6.8 (Construction Considerations), due to the presence of artesian groundwater pressures observed during the subsurface investigation, a granular filter/drainage blanket is recommended to mitigate the upward migration of fine soil particles along the pile shafts for end-bearing piles.

In addition, vibration monitoring is recommended during pile driving, on the existing portions of the South Canal bridges that remain in place near the new foundation elements. This is discussed further in Section 6.8.

6.3.2 Axial Geotechnical Resistance

6.3.2.1 Friction Piles

If friction piles are considered for support of the replacement bridges, it is recommended that a pile load test be conducted due to the complexities of the subsurface conditions at this site, to confirm and revise/refine if necessary the geotechnical resistance recommendations. However, at this stage, for HP 310x110 friction piles driven to or terminating within the till deposit above the artesian stratum, the factored axial geotechnical resistance at Ultimate Limit States (ULS) may be taken as 500 kN. The axial geotechnical resistance at Serviceability Limit States (SLS, for 25 mm of settlement) may be taken as 425 kN. The following note, or similar, should be shown on the Contract Drawings, assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (Note 2 from the *Structural Manual*, Section 3.3.3 (MTO, 2008)):

> *"Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 1,000 kN per pile."*

6.3.2.2 End-Bearing Piles

For end-bearing steel H-piles driven into "100-blow" soil, the factored axial geotechnical resistance at ULS and the axial geotechnical resistance at SLS (for 25 mm of settlement) may be taken as follows:

Pile Size	Factored Axial Geotechnical Resistance at ULS (kN)	Axial Geotechnical Resistance at SLS* (kN)
HP 310x110	1,275	1,050
HP 310x132	1,400	1,150
HP 310x152	1,600	1,350

* For 25 mm of settlement

The same axial resistances may be used in the design of closed-end, concrete-filled 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.) founded at the same design pile tip elevations. Golder has also investigated with deep foundation contractors the applicability and constructability of 610 mm diameter closed-end, concrete-filled steel pipe piles at this site, and it is considered that it would be very difficult to reach the target founding elevation and geotechnical resistance for such large diameter closed-end piles given the subsurface conditions at this site.

The following note should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (Note 2 from the *Structural Manual*, Section 3.3.3 (MTO, 2008)):

Pile Size	Note on Contract Drawing
HP 310x110	*Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 2,550 kN per pile.*
HP 310x132	*Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 2,800 kN per pile.*
HP 310x152	*Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 3,200 kN per pile.*

Pile installation should be in accordance with OPSS 903 *(Deep Foundations).* The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should be verified in the field by the use of the Hiley formula (MTO Standard Drawing SS-103-11) during the final stages of driving. Assessment of the ultimate axial 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 provided in Section 6.3.1, and at 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate capacity as determined by the Hiley formula is not achieved within the 1.5 m interval down to the design pile tip elevation, the Contractor should stop pile driving and notify the Contract Administrator. At this depth, the pile should be allowed to rest for 48 hours and the Hiley formula should then be applied immediately upon re-striking the pile. If the ultimate capacity is still not achieved after the 48 hour wait period, the Contract Administrator should be notified and authorization given prior to driving the pile below the design pile tip elevation.

6.3.3 Downdrag Loads

With earth or granular fill, and even with lightweight slag fill, the widened embankment loading will cause consolidation settlement of the soft to stiff clayey silt deposit (as discussed further in Section 6.6), and potentially lateral spreading of the deposit that could impact the existing or new piles. Downdrag loads (due to the consolidation of the soft to stiff clayey soils present at the site under or adjacent to any area of increased loading) will develop along the portion of the pile shaft that is embedded within the clayey deposits, unless measures to eliminate downdrag loads are adopted as discussed below. Downdrag will predominantly affect the new north abutments which are being shifted southward from the existing location, and the outer portions of the south abutments where the Highway 400 embankment is being widened. Downdrag loads have also been assessed at the centre pier, assuming some potential for loading increase depending on the final configuration of the local road and berm in this area; however, in the final design, it appears that there will be a net unloading in the vicinity of the centre pier, and so no downdrag loading will apply at this foundation element. Downdrag forces could also impact the existing north abutment or north pier piles during the initial stage of construction of the new outside portions of the bridges.

In calculating the magnitude of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual (CFEM, 2006) as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" (Briaud and Tucker, 1994) were considered. Given the larger predicted settlement of the clayey silt deposit in comparison to the elastic shortening of the pile, the neutral plane used in the analysis of downdrag was assumed to be at the underside of the clayey silt deposit. For design, the downdrag load on a single end-bearing HP 310x110, 310x132 or 310x152 pile may be taken as follows:

Foundation Element	Unfactored Downdrag Load (kN)
North Abutment	375
South Abutment	155

Alternatively, downdrag loads could be minimized or eliminated with the use of lightweight expanded polystyrene (EPS) or cellular concrete fill as backfill behind the abutments and associated retaining walls. Consideration could also be given to the use of bitumen coating on the piles to eliminate the downdrag loads. However, the use of bitumen coating increases the pile costs by approximately 20 to 45 per cent depending on the size of the job, and would likely require an asphalt plant on site. For this widening project, it is estimated that the cost increase would be closer to the upper limit.

For this project, it is recommended that, if space permits relative to the South Canal and local road during the construction staging, preloading of the abutment areas be carried out prior to the installation of the piles to reduce or potentially eliminate the downdrag loads that would develop on the pile shaft. However, based on the construction schedule, construction of the outside portions of the bridges will have been completed in the first stage just prior to paving and opening of the widening portion to traffic for the winter shutdown period, and there is insufficient time for preloading to reduce the magnitude of settlement over the immediate approach to the north abutment. Therefore, it is recommended that EPS be used behind the north abutment to mitigate the settlement in this area (as discussed further in Section 6.6); this will also mitigate the downdrag loads and potential lateral spreading effects in the north abutment area.

6.3.4 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by the horizontal component of battered piles. If vertical piles are used for integral abutments, the resistance to lateral loading is derived from the soil in front of the piles. The resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is determined based on the equation given below (CFEM, 1992, as noted in Section C6.8.7.1 (Table C6.5) and in Section C6.8.7.3 of the *Commentary to CHBDC*).

For non-cohesive soils:

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

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

The following values of n_h and s_u may be assumed in the structural analyses, using the interpreted stratigraphic conditions as shown on the profiles on Drawings 2 to 4. The resistance to lateral loading should be neglected within the zone of frost penetration (i.e., within 1.5 m below the lowest surrounding grade in front of the piles).

Soil Unit	n_h (kPa/m)	s_u (kPa)
Embankment fill (assuming engineered earth fill)	5,000	-
Soft to firm clayey silt	-	25
Stiff to very stiff clayey silt	-	100
Hard clayey silt	-	250
Compact to very dense silt to silty sand to sand and silt to sand	10,000	-
Very dense sand and gravel	20,000	
Firm to stiff clayey silt till	-	50
Very stiff clayey silt till	-	100
Hard clayey silt till	-	400
Very dense sand and silt till	20,000	-
Compact sand and gravel	8,000	-

A maximum factored lateral resistance of 120 kN at ULS, and a maximum lateral resistance of 35 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310x110 piles. These values are based on the “Assessed Horizontal Passive Resistance Values for Various Pile Types” provided in Table C6.8.7.1(a) of the *Commentary* to the *CHBDC*. The above recommendations based on subgrade reaction theory and assessed values can be refined based on soil-structure interaction modelling using a software program such as L-Pile, if necessary, as the detail design of the deep foundations proceeds.

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

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor
8d	1.00
6d	0.70
4d	0.40
3d	0.25

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

6.4 Caisson Foundations

6.4.1 Founding Elevations

Caisson foundations are geotechnically feasible for support of the abutments and center pier for the replacement bridges and associated wingwalls and retaining walls. For the installation of caissons, consideration must be given to the artesian groundwater pressures associated with the deep granular deposit (below the clayey soils and the till deposit), as described in Section 6.3. Similar to the driven pile foundations, the caissons may be founded within the till deposit approximately 3 m above the surface of the artesian stratum or, alternatively, taking into consideration the issues associated with penetrating the artesian groundwater zone, the caissons may be founded within the "100-blow" soils at the design founding elevations given in Section 6.3.1.

It is anticipated that caissons will experience greater construction challenges than driven pile foundations due to the artesian groundwater conditions at the site, including the potential for ground disturbance and base heave. If caisson foundations are adopted, a temporary or permanent liner would be required to support the soils during construction and to permit cleaning of the caisson base. In addition, appropriate methods would be required to minimize and control base disturbance due to the groundwater pressures, such as the use of drilling mud or an adequate head of water (i.e., within a casing stick-up above ground surface) throughout all stages of the caisson excavation and the placement of concrete by tremie methods.

6.4.2 Axial Geotechnical Resistance

Caissons founded within the till deposit approximately 3 m above the surface of the artesian stratum (i.e., "friction caissons") should be designed using the following axial geotechnical resistances/reactions:

Friction Caissons		
Caisson Diameter (m)	**Factored Axial Geotechnical Resistance at ULS (kN)**	**Axial Geotechnical Resistance at SLS* (kN)**
0.9	1,300	1,050
1.2	2,300	1,900

* For 25 mm of settlement

Alternatively, end-bearing caissons founded within the “100-blow” soils should be designed using the following axial geotechnical resistances:

End-Bearing Caissons		
Caisson Diameter (m)	Factored Axial Geotechnical Resistance at ULS (kN)	Axial Geotechnical Resistance at SLS* (kN)
0.9	2,500	2,100
1.2	4,500	3,750

* For 25 mm of settlement

6.4.3 Downdrag Loads

With the use of earth, granular or slag fill for the embankment construction, the widened embankment loading will cause consolidation settlement and lateral spreading in the soft to stiff clayey silt deposit (as discussed further in Section 6.6), particularly in the vicinity of the north abutment. Downdrag loads (due to the consolidation of the soft to stiff clayey soils present at the site under or adjacent to any area of increased loading) will develop along the portion of the caisson shaft that is embedded within the clayey deposits, unless measures to eliminate downdrag loads are adopted as discussed below. Downdrag will predominantly affect the new north abutments (which are being shifted southward from the existing location) and the outer portions of the south abutments (where the Highway 400 embankment is being widened). Downdrag loading was also assessed at the centre pier, assuming some potential for loading increase depending on the final configuration of the local road and berm in this area; however, in the final design, it appears that there will be a net unloading in the vicinity of the centre pier, and so no downdrag loading will apply at this foundation element.

For design, the downdrag load on a single end-bearing caisson may be taken as follows:

Foundation Element	Unfactored Downdrag Load (kN)	
	0.9 m Diameter Caisson	1.2 m Diameter Caisson
North Abutment	900	1,200
South Abutment	400	500

As for a driven pile foundation option, it is recommended that, if space permits relative to the South Canal and local road during the construction staging, preloading of the abutment areas be carried out prior to the installation of the caissons to reduce or potentially eliminate the downdrag loads that develop on the caisson shafts. Alternatively, downdrag loads could be eliminated with the use of EPS fill as backfill behind the abutments. As was discussed in Section 6.3.3, based on the construction staging for this project, EPS fill will be used behind the north abutment to minimize the differential settlement in this area due to the short period available for preloading; therefore, downdrag loading would not apply at the north abutment.

6.4.4 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 6.3.4.

6.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems, wingwalls and any associated concrete 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 the 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 abutment walls, wingwalls and retaining walls. These design recommendations and parameters assume level backfill and ground surface behind the walls.

- Select, free-draining granular fill meeting the specifications of SP105S13 *(Aggregates)* Granular A or Granular B Type II (but with less than 5 percent passing the 200 sieve) should be used as backfill behind the walls.
- 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 *(Compacting).* Other surcharge loadings should be accounted for in the design as required.
- The granular fill may be placed either in a zone with the width equal to at least 1.5 m behind the back of the walls ("Case I" – see Figure C6.20(a) of the *Commentary* to the *CHBDC*), or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing ("Case II" – see Figure C6.20(b) of the *Commentary* to the *CHBDC*).
- For Case I, the parameters or coefficients provided below may be used. At the north abutment, as discussed in Section 6.3.3 and 6.6, granular fill will be placed over the abutment pile cap and EPS fill will be used behind the granular zone, in order to minimize settlement following backfilling, due to the construction schedule associated with the first stage of the bridge replacement.

	Existing Fill	North Abutment – EPS (Plus Granular Fill Over Pile Cap)
Soil unit weight:	20 kN/m^3	0.5 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.33	0.21
At rest, K_o	0.50	0.36

- For Case II, where the pressures are based on OPSS 1010 Granular A or Granular B Type II fill behind the wall, the following parameters or coefficients may be assumed:

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

Where the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where the wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure(s). 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*.

6.5.1 Seismic Considerations

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

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

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

where
- K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
- K_{AE} is the seismic active earth pressure coefficient;
- γ' is the effective unit weight of the soil (kN/m^3)
  - taken as soil unit weights given above for fill materials
  - taken as 21 kN/m^3 for the native materials
- d is the depth below the top of the wall (m); and
- H is the height of the wall above the toe (m).

According to Table C4.2 of the *Commentary* to the *CHBDC*, this site is located in Seismic Zone 1, and the site-specific zonal acceleration ratio (A) for the Aurora-Newmarket area is 0.05. The site-specific peak ground acceleration (PGA) is 0.022g based on the NRC (2010) website; however, the more conservative *CHBDC* value has been used in the assessments presented below. The Site Coefficient (S) may be taken as 1.2, consistent with Soil Profile Type II in accordance with Section 4.4.6 and Table 4.4 of *CHBDC* (2006). Based on the subsurface conditions at the site, a 20 per cent amplification of the ground motion is recommended for design, resulting in an increase in the ground surface acceleration to approximately 0.06g.

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

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case A	Case B	
	Existing Fill	**Granular A**	**Granular B Type II**
Yielding Wall	0.32	0.26	0.26
Non-Yielding Wall	0.36	0.30	0.30

Notes:

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

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

6.6 Approach Embankments

6.6.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil, peat/organic soil, and existing surficial fill materials be stripped from the footprint of the proposed approach embankment widenings. Based on the boreholes results at the abutments and approach embankment/retaining wall areas, subexcavation of peat/organic soil will be required to the following elevations within the footprint of the approach embankments and retaining walls:

Area	Subexcavation Elevation (m)
Highway 400 NBL – North Approach/Wall (Northeast quadrant)	217.0
Highway 400 NBL – South Approach/Wall (Southeast quadrant)	220.0 (Essentially no subexcavation beyond what is required to construct wall in this area)
Highway 400 SBL – North Approach/Wall (Northwest quadrant)	218.0
Highway 400 SBL – South Approach/Wall (Southwest quadrant)	219.3

Staged subexcavation, in strips of limited width, will be required to maintain the stability of the excavation in the area north of South Canal, to protect the Highway 400 embankment and the local roads. It is envisaged that this subexcavation will be completed in "wet conditions" (i.e., without dewatering), as follows:

- Removal of the peat/organic soils and the overlying fill materials within the approach embankment or wall footprint is to be carried out in short "strip" sections perpendicular to the Highway 400 and local road alignments, with the base of the excavation (as measured parallel to the toe of the Highway 400 embankment or local road) not wider than 3 m.
- Temporary excavation side slopes or back slopes through the peat/organic soils and overlying fill materials shall be no steeper than 1 horizontal to 1 vertical (1H:1V).
- Excavation and backfilling operations are to be carried out simultaneously in a manner that the excavation is not left open for more than the 3 m "strip" width at any given time.

An Operational Constraint is provided in Appendix D to address this requirement, for inclusion in the Contract Documents. The subexcavation areas should be backfilled with Granular B Type II, which will minimize segregation of the soil particles during placement assuming wet conditions in the strip excavations.

Additional fill for construction of the Highway 400 embankment widening could consist of clean earth fill or granular fill or, as discussed in the succeeding sections regarding settlement mitigation options, lightweight or ultra-lightweight slag fill. The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010 *(Benching of Earth Slopes)*. The fill for the widened embankment should be placed and compacted in accordance with OPSS 501 *(Compacting)*, with inspection and field density testing by qualified personnel during placement operations to confirm that appropriate materials are used and that adequate levels of compaction are achieved. Where slag fill materials are used for the embankment widening, their placement and compaction should be in accordance with the Non-Standard Special Provision contained in Appendix D.

In accordance with MTO's standard practice and OPSD 202.010 *(Slope Flattening),* a minimum 2 m wide bench should be provided where embankment side slopes are equal to or greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS 804 *(Seed and Cover).*

6.6.2 Approach Embankment Stability

Static and seismic slope stability analyses have been completed for the proposed embankment widening using the commercially available program *Slide* from Rocscience, to check that the target minimum factor of safety is achieved. These analyses assume embankments with side slopes oriented at 2 horizontal to 1 vertical (2H:1V); however, as assessed throughout the detail design process, retaining walls will be required in order to minimize impacts on adjacent property owners associated with moving local roads. The global stability of retaining walls is addressed in Section 6.7 of this report. Other embankment geometries and heights are addressed in the Foundation Investigation and Design Report for the embankment widening and retaining walls.

6.6.2.1 Static Stability Analysis

A target minimum factor of safety of 1.3 is normally used in the design of slopes under static conditions. This minimum factor of safety is considered appropriate for the proposed Highway 400 embankment widening at the South Canal bridge site, considering the design requirements and the available field and laboratory testing data.

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

Soil Deposit	Short-term (Undrained) Analysis			Long-tem (Drained) Analysis		
	Bulk Unit Weight (kN/m^3)	Effective Friction Angle, φ'	Undrained Shear Strength (kPa)	Unit Weight (kN/m^3)	Effective Friction Angle, φ'	Cohesion (kPa)
Embankment fill	21	34°	-	21	34°	0
Soft to firm clayey silt	19	0°	20 - 40	19	28°	0
Stiff to very stiff clayey silt	19	0°	60 - 100	19	30°	0
Hard clayey silt	21	34°	-	21	34°	0
Compact to very dense silty sand to sand and silt to sand and gravel	20	32°	-	20	32°	0
Firm to very stiff clayey silt till	21	0°	75-125	21	32°	0
Hard/very dense clayey silt till to sand and silt till	21	34°	-	21	34°	0

The analysis results indicate that 5.5 m and 9.0 m high embankments at the north and south approaches, respectively, with side slopes oriented no steeper than 2H:1V, would have a factor of safety greater than or equal to 1.3 against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials is carried out. Example static global stability results for both short-term (undrained) and long-term (drained) conditions are provided on Figures 1 to 4 for the north and south approaches.

6.6.2.2 Seismic Stability Analysis

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

The pseudo-static seismic slope stability analyses for a 2H:1V slope configuration at the north and south approaches indicate that the slope will have a factor of safety greater than 1.0 against deep-seated slope instability, using a peak ground acceleration of 0.06g. The results of the seismic stability analyses at the approach embankments are shown on Figures 5 and 6 for the north and south approaches, respectively. The results of the pseudo-static seismic stability analyses indicate that some shallow sloughing could occur on the slopes during seismic events. This sloughing would not, however, impair the use of the highway, and would mainly be a maintenance issue. The potential for sloughing following seismic events could be reduced by providing well-vegetated slopes, per OPSS 804 *(Seed and Cover)*.

6.6.3 Approach Embankment Settlement

The new South Canal bridges are proposed to be constructed along the existing Highway 400 alignment, but shifted approximately 36 m to the south, with widening on both the west and east sides of the existing highway embankment to accommodate additional lanes. Due to the southward shift, the north abutment will be constructed at approximately the location of the existing South Canal Road; the embankment loading will be increased at the north abutment location due to the placement of additional fill above the current abutment foreslope level. At the south abutment location, construction will require cutting into the existing embankments, and therefore there will be no or limited change in the embankment loading under the existing Highway 400 embankment at the new south abutment. Based on the design cross-sections provided by URS, the highway embankment will be widened on both the west and east sides of the existing embankment by between 8 m and 15 m (horizontal distance between existing and proposed outside shoulders at the crest), and will require placement of a maximum vertical thickness of approximately 5 m to 7 m of fill. Retaining walls are proposed to be constructed immediately adjacent to the new abutments, parallel to Highway 400.

6.6.3.1 Primary Consolidation Settlement

Primary consolidation settlement analyses for the soils below the widened approach embankments were carried out using both hand calculations and the commercially available computer program *Settle-3D* from Rocscience, using estimated elastic deformation moduli and consolidation settlement parameters as given below, based on correlations with the field and laboratory test data and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight	Elastic Modulus	P_c'	e_o	C_c	C_r
Embankment fill (existing and new)	21 kN/m^3	–	–	–	–	–
Compact to very dense sand to sand and gravel	20 kN/m^3	10 - 25 MPa	–	–	–	–
Soft to firm clayey silt	19 kN/m^3	-	100 - 175 kPa	0.64	0.16	0.026
Stiff to very stiff clayey silt	19 kN/m^3	-	175 - 250 kPa	0.64	0.13	0.022
Very stiff to hard clayey silt till/ Very dense sand and silt till	21 kN/m^3	50 – 75 MPa	–	–	–	–

Based on the settlement analyses, the primary consolidation settlement of the foundation soils under an additional 5 m to 7 m of <u>conventional earth or granular fill</u> associated with the east and west widening of the existing Highway 400 embankment, and the additional fill to be placed behind the new north abutment location, is estimated to be up to approximately 350 mm at the north approaches and up to approximately 60 mm at the south approaches. The majority of this settlement is expected to occur within the soft to firm clayey silt deposit at the north abutments and approaches.

It is estimated that the time to complete ninety per cent of the primary consolidation settlement at the north abutment and approaches will be approximately eight to nine months following placement of the fill for the embankment widening. It is estimated that less than 25 mm of primary consolidation settlement will remain approximately nine months after placement of fill for the embankment widening. The predicted post-construction settlement due to the embankment widening construction can be mitigated or reduced with preloading, surcharging and/or the use of lightweight fill; these alternatives are further discussed in Section 6.6.4.

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

6.6.3.2 Secondary (Creep) Settlement

In addition to primary consolidation within the clayey deposit at this site, secondary compression will also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after substantial dissipation of excess pore pressure under a constant stress.

The rate of secondary consolidation (creep) settlement for the clayey deposit is expected to be up to about 40 mm per log-cycle of time at the South Canal bridges site. The magnitude of creep settlement following construction will depend on the method of construction/settlement mitigation adopted and the actual time required to achieve the majority of the primary consolidation. If preloading/surcharging measures are implemented to achieve the majority of the primary consolidation settlement in advance of completion of the bridge replacement and paving, it is estimated that up to about 40 mm of creep settlement could occur over a 10-year period following completion of construction.

6.6.4 Settlement Mitigation Options

The post-construction settlement as estimated above, assuming the use of conventional earth or granular for the embankment widening and retaining wall construction, can be mitigated or reduced using the following mitigation options:

- sub-excavation of soft to firm clayey silt soils (i.e., additional subexcavation beyond the minimum requirement for subexcavation of the peat/organic soils);
- preloading of the widened approach embankment areas;

- preloading of the widening approach embankment areas in conjunction with a surcharge, to accelerate the rate of settlement over the use of preloading alone;
- use of lightweight fill such as slag, cellular concrete or expanded polystyrene (EPS) for construction of the widened portions of the embankment;
- use of wick drains (in conjunction with preloading); or
- a combination of these measures.

Other ground improvement measures such as rammed aggregate piers, deep soil mixing and dynamic compaction are not considered suitable or cost effective due to the composition of the site soils (i.e. cohesive soils), the thickness of the deposit, and the proximity to existing bridge foundations, and these are not addressed further in this report.

6.6.4.1 Subexcavation of Soft to Firm Clayey Soils

Subexcavation of the peat/organic soils that are present at the site is required, at a minimum, as outlined in Section 6.6.1.

Additional subexcavation, of the weak/soft and compressible clayey deposits below the peat/organic materials has been examined, but is not considered to be a practical option for improving the stability and controlling the total settlement of the founding soils. The soft to firm clayey silt deposit at the north abutment and approach embankment area extends to approximately Elevation 207 m, which would require excavations to a depth of approximately 13 m to 14 m below the original ground surface at this site. This would require extensive temporary protection systems along the outer edges of the existing Highway 400 northbound and southbound lanes to facilitate subexcavation beneath the toe of the existing embankments, would produce a large volume of spoil material, and would require a large volume of replacement fill.

Due to the depth of subexcavation that would be required, this option has not been considered further in this report.

6.6.4.2 Preloading

Preloading may be considered for reducing post-construction settlements of the subsoils under the proposed embankment widening and new retaining wall areas. Preloading refers to the placement of fill either up to the proposed profile grade of the highway or a portion thereof (i.e. partial preload), in one or more stages, in advance of the embankment completion and final pavement construction, in order to preconsolidate the underlying compressible soils. Preloading reduces the magnitude of long-term, post-construction settlements by promoting such settlements to occur under the fill loads in advance of final grading of the embankment.

As discussed in Section 6.6.3, it is estimated that ninety per cent of the primary consolidation settlement at the north abutment and approaches will be completed within approximately eight to nine months following placement of the fill for the embankment widening/retaining walls. Thus, after approximately eight to nine months of preloading of the embankment widening areas, less than 25 mm of primary consolidation settlement would remain below the embankment widening areas. A Non-Standard Special Provision (NSSP) has been provided in

Appendix D for inclusion in the Contract Documents to address preloading. Monitoring of the settlement during the preloading period is recommended, as discussed further in Section 6.6.5, and as addressed in the NSSP for instrumentation and monitoring, as provided in Appendix D.

The preload for the widening areas/retaining walls should be constructed up to the top of the highway granular sub-base. It is recommended that the required platform width be increased by 150 mm on each side of the existing embankment to accommodate the predicted settlement. After the preload period, it is recommended that additional sub-base fill be placed to achieve the final subgrade level prior to placement of the pavement structure.

6.6.4.3 Preloading and Surcharging

Similar to preloading, surcharging refers to the placement of embankment fill in advance of final pavement construction to reduce long-term, post-construction settlements. The difference between preloading and surcharging is the amount of fill placed and the time required for consolidation to be achieved. With surcharging, the fill is placed to the full embankment height (i.e. preloading), followed by an additional lift of fill (the surcharge) above that required to construct the final embankment geometry. The additional lift of fill applies greater stress to the underlying cohesive deposits and reduces the time for consolidation over that achieved by preloading alone, resulting in “overconsolidation” of the underlying compressible foundation soils relative to the final design embankment height. At the end of consolidation, the portion of the surcharge fill remaining above the required embankment height is removed. Surcharging would be most suitable if the construction schedule did not allow for sufficient time for the consolidation settlements to occur under preload fill loads alone (i.e., if there is not eight to nine months available in the schedule for preloading alone at the north approach embankment area).

The estimated time to complete ninety per cent of the primary consolidation settlement at the north abutment and approaches as well as the factors of safety for global stability of the embankment slopes for a range of surcharge heights are summarized below.

Surcharge Height (m)	Factor of Safety for Global Stability		Duration for 90% of Primary Consolidation Settlement
	Short-Term (Undrained)	Long-Term (Drained)	
1.0	1.5	1.6	5 to 6 months
1.5	1.4	1.5	4.5 to 5.5 months
2.0	1.3	1.4	4 to 4.5 months
2.5	1.3	1.4	3.5 to 4 months

At this site, due to space constraints, surcharging of the preload areas would be difficult.

However, if construction staging and property constraints can be accommodated, partial sub-excavation of the upper portion of the thick, soft to firm clayey silt deposit in combination with surcharging would further reduce the post-construction settlement, as the upper 2 m of the deposit has a softer consistency than the bottom portion of the deposit. As discussed further in Section 6.6.5, monitoring of the settlement during the preloading and surcharging period is recommended if this option is adopted.

6.6.4.4 Lightweight Fill

Lightweight fill, such as lightweight slag, ultra-lightweight slag, cellular concrete or expanded polystyrene (EPS) could be used for the embankment widening and behind the new retaining walls, to reduce the additional loading imposed on the underlying soils. Slag fills could also be used directly in a retained soil system (RSS) wall application. The use of lightweight fill would reduce the load applied to the foundation soils due to the lower density of the fill materials, which in turn would reduce the magnitude of post-construction settlement. The lighter fill loading would reduce the predicted magnitude of the primary consolidation settlement under the north abutment and approaches as follows:

Fill Option	Unit Weight (kN/m^3)	Predicted Primary Consolidation Settlement (mm)
Lightweight Slag	14	180 to 200
Ultra-Lightweight Slag	11	110 to 120
EPS or Cellular Concrete	0.5 to 1	5 to 10*

* Due to the sub-excavation of the fill and peat (to Elevation 217 m to 220 m, as identified in Section 6.6.1) and replacement with EPS, negligible settlements are anticipated.

For this project, based on the construction schedule as discussed in Section 6.3.3, construction of the outside portions of the bridges will have been completed in the first stage just prior to paving and opening of the widening portion to traffic for the winter shutdown period, and there is insufficient time for preloading to reduce the magnitude of settlement over the immediate approach to the north abutment. Therefore, it is recommended that EPS be used behind the north abutment to mitigate the settlement in this area. Golder has worked with AECOM's structural engineers through detail design to develop the details for the placement of the EPS blocks behind the new north abutment and in front of the existing north abutment, on top of the existing ground surface (with nominal excavation for levelling). An NSSP for supply and placement of EPS is included in Appendix D, for inclusion in the Contract Documents.

For other retaining walls or embankment widening within the limits of the immediate approach embankments, the use of slag fill is recommended. Assessment for the retaining walls and embankment widening in the area north of the South Canal bridges is presented in a separate Foundation Investigation and Design Report, and based on that assessment and for consistency, ultra-lightweight slag fill is recommended for use in the embankment widening/RSS walls on the east side of Highway 400, and lightweight slag fill is recommended for use in the embankment widening (and limited-length RSS wall) on the west side of Highway 400. NSSPs for the supply, placement and compaction of lightweight and ultra-lightweight slag fill are included in Appendix D, for inclusion in the Contract Documents, and this aspect is discussed further in Section 6.8.

6.6.4.5 Wick Drains

Where sub-excavation is not practical (i.e. due to the thickness of or depth to the compressible soil deposits), and where the time required to achieve preconsolidation is considered too long, consideration may be given to installing wick drains in conjunction with preloading and surcharging to accelerate the rate of primary consolidation. Wick drains are prefabricated geotextile drains installed vertically from ground surface into or through soft, compressible soils to increase the rate of excess porewater pressure dissipation. Typically, wick drains are installed on a 1 m to 3 m triangular grid spacing over the footprint of the embankment widening.

Wick drains have been preliminarily assessed for use at the north abutment and approach embankment area. However, it is considered that wick drains could induce settlement at the existing piles for the north pier during construction staging, thereby impacting the existing bridge foundations, and as a result this approach is not preferred at this site. In addition, based on the anticipated construction schedule, there is adequate time for preloading in the area north of the bridge, with the exception of the immediate approach embankments; this limited area immediately behind the new north abutment, on top of the existing abutment foreslope, will be addressed using EPS fill material (as discussed in Section 6.3.3).

6.6.5 Settlement Monitoring

It is recommended that monitoring be carried out for the widened portions of the Highway 400 approach embankments and retaining walls, to monitor the magnitude and rate of settlement during the preloading period. A monitoring program has been developed for the approach embankments as well as for the embankment widening and RSS wall construction beyond the limits of the immediate approaches. This is discussed further in Section 6.8.8.

6.7 Retained Soil System (RSS) Walls

Retaining walls are proposed adjacent to the new abutments/wingwalls for the replacement bridges. The proposed walls will be constructed along the outside edges of the widened northbound and southbound lanes, oriented parallel to Highway 400, and will be up to about 7 m high; a lower RSS wall will also be required in the northwest quadrant of the bridge site adjacent to South Canal Bank Road. Based on the discussion of retaining wall options presented in Section 6.2.2, retained soil system (RSS) walls are the preferred retaining wall solution for this site, beyond any pile-supported concrete walls immediately adjacent to the north abutment of the bridges, subject to considerations of settlement as discussed in Section 6.6 and as discussed further below. RSS walls should be designed for high performance and appearance in accordance with MTO’s Standard Special Provisions for the design and construction of RSS walls.

6.7.1 Founding Elevations

A typical RSS wall has front facing panels supported on compacted granular fill at a shallow depth below the ground surface in front of the wall. The facing panels and the reinforced soil mass should be founded below any existing topsoil, peat/organic soils, or unsuitable fill soils.

At the south abutment and south approach, the peat layer extends to approximately Elevation 219 m and a firm clayey silt deposit extends to approximately Elevation 218 m. At the north abutment and north approach, the peat layer extends to approximately Elevation 217 m to 218.5 m, and is underlain by a soft to firm clayey silt deposit that extends to approximately Elevation 214 m. It is not considered practical to subexcavate the soft to firm clayey soils at the north approach area, as such excavations would be approximately 7 m deep in close proximity to the existing highway and bridge structures. However, subexcavation of the peat/organic materials is required, to the elevations given in Section 6.6.1, with the subexcavated areas subsequently backfilled with OPSS 1010 Granular B Type II. The reinforced soil mass and permanent facing panels for the RSS wall system may then be founded on the granular backfill.

A minimum 0.5 m thick compacted granular pad, consisting of OPSS 1010 Granular A, should be used for levelling purposes below the permanent RSS facing panels, and should extend at least 0.5 m beyond the outside edge of both sides of the facing panels, then outward/downward at 1H:1V.

6.7.2 Geotechnical Resistance and Settlement

For the RSS facing panels bearing on compacted granular fill as described above, following subexcavation and replacement of the peat/organic materials, a factored geotechnical resistance at ULS of 150 kPa should be used for design.

For the reinforced soil mass founded following subexcavation of the peat/organic materials and replacement with Granular B Type II fill, as discussed in Sections 6.6.1 and 6.7.1, the factored geotechnical resistances at ULS given below may be used for design of the reinforced soil mass. These values assume that the reinforced soil mass acts as a unit and uses the full width of the reinforced soil mass (which can be taken as approximately 0.8 times the wall height for design purposes).

Retaining Wall	Subexcavation Elevation	Maximum Wall Height Above Finished Grade	Assumed Reinforced Width	Factored Geotechnical Resistance at ULS
North Approach	217.0 – 218.0 m	3 m 5.5 m	2.4 m 4.4 m	125 kPa 175 kPa
South Approach	219.3 – 220.0 m	7 m	5.6 m	500 kPa

As noted in Section 6.6, settlements of up to about 50 mm to 60 mm will occur for RSS walls located at the south approaches, and settlements of up to about 300 mm to 350 mm will occur for RSS walls located at the north approaches, under the planned Highway 400 widening; these settlements assume the use of conventional earth or granular fill. Based on discussions with designers and suppliers of RSS walls, it is understood that the reinforced soil mass in proprietary RSS systems is flexible and can tolerate on the order of 1 m of differential settlement over a 100 m length. However, in a typical RSS system, the facing panels cannot tolerate this magnitude of total or differential settlement and still meet aesthetic requirements.

The predicted post-construction settlement due to the embankment widening/retaining wall construction can be mitigated or reduced by preloading, surcharging and/or the use of lightweight fill as discussed in Section 6.6.4.

Beyond the mitigation measures in Section 6.6.4 and specific to RSS walls, this also included consideration of layers of EPS "sandwiched" between layers of granular fill to reduce the overall loading on the subsoils, and the use of high density cellular concrete in approximately 0.6 m thick lifts (with the anchor points for permanent wall facing panels embedded into the cellular concrete during placement of the layers). The various mitigation measures were assessed and discussed with the design team throughout detail design, and the preferred solution for RSS wall construction involves the following:

- The use of ultra-lightweight slag fill (both to achieve the required factor of safety for global stability in some sections of the RSS wall north of the bridges, and to improve settlement performance).

- The use of a two-stage RSS wall, in which the reinforced soil mass is constructed to the underside of the pavement structure, and allowed to settlement for the duration of the preloading period with settlement monitoring throughout; following the preloading period, the permanent facing panels are affixed to the front of the reinforced soil mass. With the use of slag fill for the RSS wall construction, there is some potential for increased corrosion of the steel strips, and it is recommended that this be addressed by increasing the thickness of the reinforcing strips. The Standard Special Provision for RSS walls has been modified to address the requirement for the reinforcing strips to incorporate a sacrificial thickness on this project (see Appendix D).

It is estimated that the period of time to complete ninety per cent of the primary consolidation settlement is approximately eight to nine months following placement of the fill for the embankment widening/retaining wall. It is recommended that the proposed north retaining wall areas be preloaded for a period of up to nine months, as discussed in Section 6.6.4.2. It is estimated that less than 25 mm of primary consolidation settlement will still occur beyond that time for the area below the north retaining walls, and after this period the permanent facing panels can be installed on the RSS walls. As discussed in Section 6.6.5 and 6.8, settlement monitoring is recommended during the preloading period.

Even with preloading or the use of a lighter-weight RSS wall, there is potential for differential settlement between the deep foundation-supported abutments/wingwalls and adjacent RSS walls as a result of secondary creep settlement. It is recommended that vertical slip joints be incorporated into the RSS walls at the interface with the pile-supported abutments/walls.

6.7.3 Resistance to Lateral Loads / Sliding Resistance

Resistance to lateral forces / sliding resistance between the compacted 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 native subgrade may be taken as 0.62 at the south approach area, and 0.53 at the north approach area.

6.7.4 Global Stability

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

The static global stability analyses for RSS walls were completed using the parameters outlined in Section 6.6.2, and assume that all existing topsoil, peat/organic soils and/or fill materials are removed prior to constructing the RSS walls, and replaced with Granular B Type II fill materials.

The results of the static global stability analyses indicate that a minimum factor of safety of 1.5 is achieved for both short-term (undrained) and long-term (drained) conditions for RSS walls constructed of conventional granular adjacent to the north and south abutments, as shown on Figures 7 through 10. The global stability for RSS walls constructed of lighter weight slag fill materials will be higher than that for conventional granular fill materials.

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

6.8 Construction Considerations

6.8.1 Removal of the Existing Bridges

The replacement of the South Canal bridges will require removal of the existing bridge structures, realignment of the existing South Canal southward, and realignment of the existing South Canal Road southward along the canal (in conjunction with an approximately 2 m to 3 m high berm to separate the local road from the canal). The upper soils at the site consist of firm to very stiff clayey silt, which will be susceptible to disturbance and softening as a result of construction activities or traffic.

In particular, it is recommended that foundation removals be limited to removal of the existing pile caps only and cutting off the existing piles at the underside of the pile caps, with no extraction of existing timber or steel piles. It is recommended that AECOM's NSSP for removals (amending OPSS 510) incorporate the following statement:

> *"The work for the existing bridge foundation removal shall be limited to removal of the pile caps including cutting the piles to the underside of the pile caps and the complete removal of the pile caps, where required and as specified in the Contract Documents, with no extraction of existing timber or steel piles."*

6.8.2 Subexcavation of Peat/Organic Materials

Based on the borehole information, layers of peat and/or organic soils were encountered below the fill and/or surficial clayey silt in several of the boreholes within the approach embankment widening areas. The base of the peat/organic layer was encountered between approximately Elevation 217 m and 220 m. These organic materials should be stripped from the plan limits of the approach embankment widening/retaining wall areas prior to fill placement for the widened embankments.

Staged subexcavation, in strips of limited width in "wet conditions" (i.e., without protection systems and dewatering), will be required to maintain the stability of the excavation in the area north of South Canal, to protect the Highway 400 embankment and the local roads. As discussed in Section 6.6.1, an Operational

Constraint is provided in Appendix D (consistent with the Operational Constraint for the embankment widening and RSS wall construction to the north of the bridges) to address the subexcavation requirements, including the following:

- Removal of the peat/organic soils and the overlying fill materials within the approach embankment or wall footprint is to be carried out in short "strip" sections perpendicular to the Highway 400 and local road alignments, with the base of the excavation (as measured parallel to the toe of the Highway 400 embankment or local road) not wider than 3 m.
- Temporary excavation side slopes or back slopes through the peat/organic soils and overlying fill materials shall be no steeper than 1 horizontal to 1 vertical (1H:1V).
- Excavation and backfilling operations are to be carried out simultaneously in a manner that the excavation is not left open for more than the 3 m "strip" width at any given time.

6.8.3 Excavation and Groundwater Control

Excavation for the new pile caps and the removal of peat/organic material for the embankment widening and retaining wall construction will extend to a maximum depth of about 4 m, through the existing fill and peat/organic soils and into the firm to very stiff clayey silt deposit. If space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill is classified as Type 3 soil, while the existing peat is classified as Type 4 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) through Type 3 soils and 3H:1V through Type 4 soils.

It is anticipated that the excavations to remove the pile caps for the existing bridge abutments and piers, and to construct the new centre pier pile cap, will extend below the groundwater table at the site, within predominantly cohesive soils. Some of these excavations will be in relatively close proximity to the existing and/or realigned South Canal, depending on staging. It is recommended that an NSSP be included in the Contract Documents to amend OPSS 902, to alert the contractor to the groundwater control requirements associated with the excavations for the bridge foundation elements (see Appendix D).

6.8.4 Temporary Protection Systems

Temporary protection systems will be required along the edges of the existing Highway 400 northbound and southbound lanes to facilitate the staged removal of the existing bridges and the construction of the new bridges. Additionally, it is anticipated that temporary protection systems may be required along the north and south side of the South Canal and South Canal Road to facilitate the removal of the existing pier pile caps (if and where required).

The temporary protection 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 existing adjacent structures or utilities can tolerate this magnitude of deformation. If excavations must be completed for removals in close proximity to the existing or new foundations, it is recommended that such protection systems meet Performance Level 1b as specified in OPSS 539.

To handle removal of existing pile caps, construction of new pile caps, and any subexcavation of peat/organic soils, as well as potentially subexcavation of soft to firm clayey soils within proposed RSS wall areas, the protection systems are required for an estimated excavation depth of approximately 4 m relative to the original (present) ground surface at the site, or about 10 m below the existing Highway 400 grade. It is considered that either a driven, interlocking sheet pile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site, based on the subsurface soil and groundwater conditions. An interlocking sheet pile system would contribute to both ground and, where applicable, groundwater control – it would provide for control of seepage of groundwater from lenses/interlayers of non-cohesive soil within the clayey silt deposit. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards.

The sheet piles or soldier piles would have to be driven or socketted to sufficient depth to provide the necessary passive resistance for the retained soil height, including any surcharge loads behind the protection system within at least a 1H:1V zone relative to the base of the excavation. Lateral support to the sheet piles or 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.

6.8.5 Mitigating Artesian Groundwater Pressures – Granular Filter Blankets

If end-bearing piles or caissons are adopted for support of the replacement bridges, the piles/caissons will penetrate the stratum with artesian groundwater pressures. There is potential for upward migration of fine soil particles along the pile shafts, with resulting potential for settlement of the piles. Specialized construction techniques will be required, including the use of a granular filter/drainage blanket beneath the new pile caps, both during pile driving and in the permanent condition. In addition, it is recommended that the filter/drainage blanket be extended beyond the new pile caps to cover the full extent of the existing abutment and pier foundation areas following removal of the existing pile caps and cut-off of the existing piles.

The filter blanket should consist of a minimum 0.5 m thick layer of concrete fine aggregate meeting the gradation requirements of OPSS 1010 Granular A. The granular fill should extend a minimum of 1 m horizontally beyond each of the piles. Appropriate drainage from under the pile cap should be provided for the granular filter blanket by using a 100 mm diameter perforated subdrain as per OPSS 405 *(Pipe Subdrains)* wrapped in knitted sock geotextile and draining to a temporary ditch or sump during construction. The geotextile should consist of non-woven, Class 1 geotextile with filtration opening size (FOS) of 75 µm to 115 µm in accordance with OPSS 1860. A schematic diagram of a filter blanket constructed below the pile caps is provided on Figure 11, and this detail should be incorporated into the Contract Drawings; in addition, an NSSP addressing the granular filter blanket is provided in Appendix D for inclusion in the Contract Documents.

Alternatively, grouting may be an option to mitigate the upward flow of water along the pile shafts and potential loss of fine soil particles. However, it is considered that such an operation will be onerous given the potential for all sides of the piles (i.e., flanges and both sides of the web) to need grouting to properly seal the seepage pathway.

6.8.6 Pile Driving

As discussed in Section 6.3.1, vibration monitoring is recommended on the existing Highway 400 bridges during driving of steel H-piles at this site; vibration monitoring is also recommended in conjunction with soldier pile or sheet pile installation and/or removal, associated with temporary protection systems. A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. An NSSP has been provided in Appendix D, for inclusion in the Contract Documents.

In addition, an amendment to OPSS 903 is recommended to alert the contractor to the potential presence of cobbles and boulders in the glacially-derived soils at the site, and to specify pile dynamic analyzer (PDA) testing on a minimum of 10 per cent of piles at each foundation element for both stages of the northbound and southbound bridges.

6.8.7 Preloading

As discussed in Section 6.6.3, it is estimated that up to approximately 350 mm of settlement will occur under the north approach embankment, and up to approximately 60 mm of settlement will occur under the south approach embankment, assuming the use of conventional earth and/or granular fill. Where lightweight or ultra-lightweight slag fill is used for the north approach widening, it is estimated that up to about 100 mm to 125 mm of settlement will occur. It is further estimated that the time to complete ninety per cent of the primary consolidation settlement at the north abutment and approaches will be approximately eight to nine months following placement of the fill for the embankment widening. An operational constraint has been developed for inclusion in the Contract Documents to address the timing requirements associated with the preloading of the approach embankment widening locations, including timing for placement of the permanent facing panels for the two-stage RSS wall.

6.8.8 Settlement Monitoring

As discussed in Section 6.6.4, it is recommended that settlement monitoring be carried out for the widened portions of the Highway 400 approach embankments to monitor the magnitude and rate of settlement during any preloading period.

A monitoring program has been developed for the approach embankments as well as for the embankment widening and RSS wall construction beyond the limits of the immediate approaches, consisting of the following:

- Settlement plates and settlement pins, installed at the base of the fill platform and top of fill, respectively.
- Settlement profilers and shape accel arrays, installed at selected locations to supplement the information from the settlement plates and pins.
- Vibrating wire and standpipe piezometers, to monitoring groundwater levels and pore water pressures within and outside the widening area.
- Inclinometers, to monitoring lateral deformation of the embankment widening areas.

Instrumentation and monitoring plans and an NSSP for settlement monitoring are included in Appendix E, for inclusion in the Contract Documents. The related Foundation Monitoring Plan for the Contract Administrator Assignment has been provided under separate cover.

6.8.9 Use of EPS and Lightweight Slag Fill for Embankment Widening

An NSSP for the supply and placement of EPS for this project has been provided in Appendix D, for inclusion in the Contract Documents.

Lightweight or ultra lightweight slag fill, where adopted for the embankment widening and two-stage RSS wall construction, respectively, will require special placement and compaction procedures to prevent overcrushing and overcompaction. MTO's Non-Standard Special Provision (NSSP) for the supply and placement of ultra-lightweight slag fill should be included in the Contract Documents; this NSSP is included in Appendix D.

In addition, an amendment to SSP 599S22 is recommended to address the requirements for thicker steel reinforcing strips for the RSS wall application, where slag fill will be used; this amendment is provided in Appendix D.

6.8.10 Erosion/Scour Protection

The soils adjacent to the realigned South Canal may be susceptible to erosion and scour under the design flood/flow velocities. The requirements for design of erosion/scour protection should be assessed by the hydraulic design engineer. As a minimum, it is recommended that erosion protection (e.g. rip-rap or granular sheeting) be provided on the canal banks adjacent to the centre pier and, if applicable, the south abutments, to protect the foundations/pile caps from being exposed. The rip-rap should be consistent with the standard R-10 classification or granular sheeting classification in accordance with OPSS 1004 (Aggregates) but should be approved by the hydraulic design engineer.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Nikol Kochmanová, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder, with technical input from Mr. Murty Devata, P.Eng., a specialist foundations consultant with Golder. Mr. Jorge Costa, P.Eng., a Principal with Golder and Designated MTO Foundations Contact, conducted an independent review of this report.

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Ontario Provincial Standard Specifications (OPSS)

OPSS 405	Construction Specification for Pipe Subdrains
OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 804	Construction Specification for Seed and Cover
OPSS 903	Construction Specification for Deep Foundations
OPSS 1002	Material Specification for Aggregates – Concrete
OPSS 1004	Material Specification for Aggregates – Miscellaneous
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS 1860	Material Specification for Geotextiles

Ontario Provincial Standard Drawings (OPSD)

OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment
OPSD 208.010	Benching of Earth Slopes
OPSD 3001.100	Foundation, Piles, Steel Tube Piles, Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario

TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings	• Not feasible for support of the structures due to the presence of relatively weak (firm) clay at shallow depths		• Likely very large footings required given the low geotechnical resistances available • Need to subexcavate weak soil deposits • Potential settlement issues not able to be mitigated to acceptable levels	• Conventional excavation and construction techniques	• Estimated cost is approximately $600/m^3$ for construction of shallow foundations, excluding deeper excavation and protection system at north abutment, and cofferdam at the piers
Steel H-piles or steel pipe piles driven to/terminating within the till deposit (friction piles)	• Feasible for support of abutments, wingwalls and center piers • Feasible for support of retaining wall	• Steel H-piles allow for integral abutment configuration, and pipe piles for semi-integral abutment configuration • Does not breach the artesian groundwater strata • Only minor groundwater seepage anticipated in pile cap excavations, so pumping from filtered sumps will provide adequate groundwater control	• Lower axial resistance than for end-bearing piles • Risk of encountering or penetrating into the higher groundwater pressure zone due to irregular surface of/depth to artesian layer • Temporary excavation support may still be required to facilitate removal of the existing abutments	• Conventional construction methods for H-pile or steel pipe (tube) pile foundations	• Estimated cost is approximately $250/m length for pile installation and $600/m^3$ for pile cap construction; the cost may be higher to account for use of temporary liners

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles or steel pipe (tube) piles driven to refusal within the till deposit (end-bearing piles)	• Feasible for support of abutments, wingwalls and center piers • Feasible for support of retaining wall	• Steel H-piles allow for integral abutment configuration, and pipe piles for semi-integral abutment configuration • Higher axial resistance than for friction piles	• This option breaches into the artesian groundwater strata • Potential upward migration of fine soil particles along the pile shafts • Potential negative impacts on the aquifer • Potential requirements for permitting this application through Ontario Ministry of the Environment • Temporary excavation support may still be required to facilitate removal of the existing abutments • Slightly greater risk than for friction piles if cobbles and/or boulders are encountered during driving; this could result in piles “hanging up” and lower geotechnical resistances	• Conventional construction methods for H-pile or steel pipe (tube) pile foundations	• Estimated cost is approximately \$250/m length for pile installation and \$600/m^3 for pile cap construction; the cost may be higher to account for use of temporary liners • Higher cost for longer piles likely compensated for when compared to shorter friction piles, as more friction piles will be required to achieve load-carrying capacity

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Caissons founded within the till deposit above the artesian groundwater strata	• Feasible but not practical for support of abutments, wingwalls and center piers • Feasible but not practical for support of retaining wall	• Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles	• High groundwater levels create potential for loss of ground or base heave • Presence of relatively weak (firm) clay deposit creates potential for construction problems associated with soil squeeze and the need for long permanent liners • Temporary or permanent liners would be required plus special measures such as use of drilling mud or head of water inside casing, and tremie placement of concrete; likely not possible to inspect caisson base • Precludes use of integral abutments	• Conventional construction methods for caisson foundations; temporary or permanent liners required for ground and groundwater control • Risk of loosening or heaving of founding soils at base of caissons	• Estimated cost is approximately \$1000/m length for caisson installation and \$600/m^3 for pile cap construction; this cost expected to be higher to account for temporary and permanent liners
Caissons founded within the till deposit below the artesian groundwater strata	• Feasible but not practical for support of abutments, wingwalls and center piers • Feasible but not practical for support of retaining wall	• Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles • Allows for “sealing” of drillhole if concrete is tremied into the caisson and the liner is removed progressively, allowing concrete to spread against the soil wall	• High groundwater levels create potential for loss of ground or base heave • Presence of relatively weak (firm) clay deposit creates potential for construction problems associated with soil squeeze and the need for long permanent liners • Temporary or permanent liners would be required plus special measures such as use of drilling mud or head of water inside casing, and tremie placement of concrete; likely not possible to inspect caisson base • Precludes use of integral abutments	• Conventional construction methods for caisson foundations; temporary or permanent liners required for ground and groundwater control • Risk of loosening or heaving of founding soils at base of caissons	• Estimated cost is approximately \$1000/m length for caisson installation and \$600/m^3 for pile cap construction; this cost expected to be higher to account for temporary and permanent liners

TABLE 2 – COMPARISON OF RETAINING WALL TYPES AND FOUNDATION ALTERNATIVES

Wall Type and Foundation Option	Feasibility	Advantages	Disadvantages	Constructability/ Risks	Estimated Costs
Concrete retaining walls on deep foundations	• Feasible	• Potentially reduced excavation, protection system and backfill requirements compared to RSS wall	• Friction piles would require controlled installation to ensure they don't penetrate into artesian groundwater pressure zone	• Conventional excavation and construction techniques • Relatively long construction time compared to most wall alternatives	• Higher cost relative to RSS wall
Retained soil system (RRS) walls	• Feasible	• Tolerant of post construction settlements – particularly with the use of a two-stage RSS wall system to address preloading period • Lower cost alternative than pile-supported concrete walls	• Potentially larger amount of excavation required to install reinforcing strips; temporary protection systems expected to be required • Requires subexcavation of peat/organic soils and replacement with Granular B Type II backfill • Use of ultra-lightweight or lightweight slag backfill required, with specialized construction procedures; consideration of design of reinforcing strips to accommodate potential corrosion	• Conventional excavation and construction techniques, albeit with special provisions for subexcavation and backfilling in strips of limited width to remove peat/organic soils prior to construction of reinforced soil mass	• Lower cost than concrete retaining wall
Soldier pile and concrete panel walls	• Not considered feasible		• Most advantageous in "top-down" construction applications, i.e. as part of a cut-widening, rather than for an embankment widening • Likely more time-consuming than other wall types due to steps involved (pre-augering for socket holes, placing soldier piles, placing backfill in lifts, installing concrete panels, installing, pre-stressing and testing tie-backs)	• More specialized equipment and skilled labour required • Construction costs and time may escalate if cobbles and boulders are encountered in soldier pile installation	• Comparable costs to concrete retaining wall, but higher than RSS wall

Wall Type and Foundation Option	Feasibility	Advantages	Disadvantages	Constructability/ Risks	Estimated Costs
Concrete retaining walls on shallow foundations	• Not feasible		• Subexcavation of organic deposits required, with special construction considerations (excavating in limited strips, etc.) • Even with subexcavation, excessive total and differential settlement in underlying compressible clayey deposit	• Conventional construction but inadequate founding conditions at shallow depth and excessive total and differential settlement	• Higher cost relative to RSS wall; must also consider subexcavation costs

CONT No. 2015–2004
GWP No. 2025–13–00

SOUTH CANAL BRIDGES

HIGHWAY 400 WIDENING

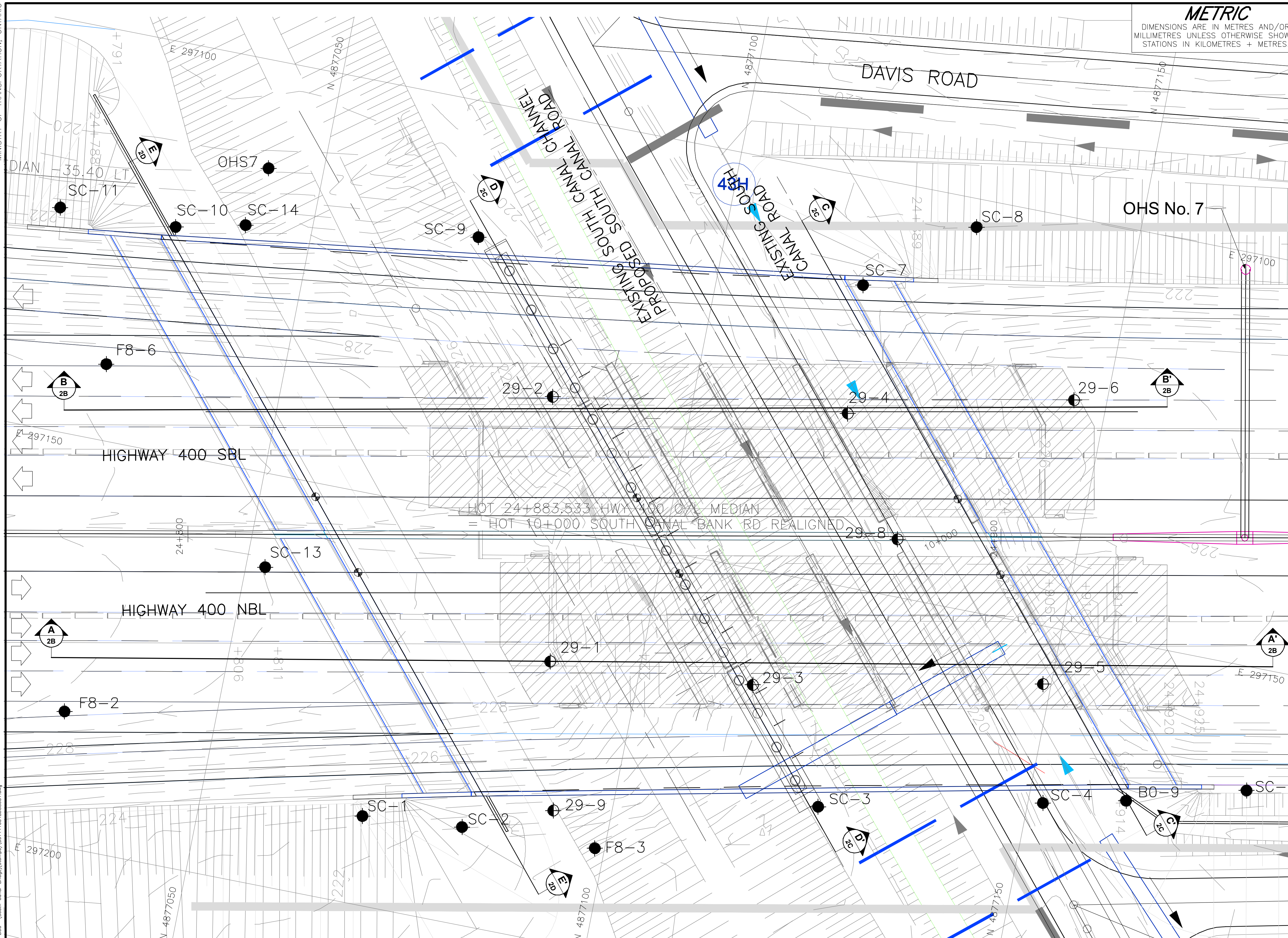
BOREHOLE LOCATIONS

SHEET

Golder Associates

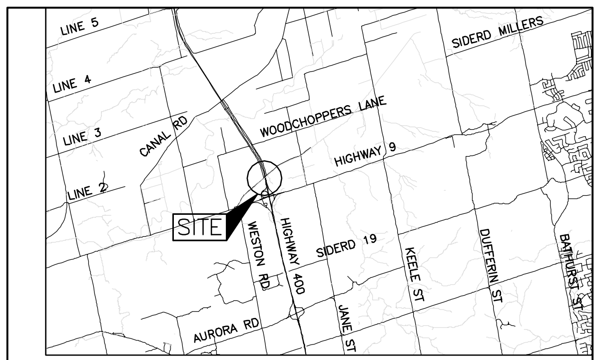
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

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



PLAN

SCALE
5 0 5 10 m



KEY PLAN

SCALE
2 0 2 km

LEGEND

- Borehole – Current Investigation by Golder
- Borehole – Previous Investigation (Geocres No. 31D–029)

BOREHOLE CO–ORDINATES			
No.	ELEVATION	NORTHING	EASTING
29–1	227.4	4877089.0	297166.0
29–2	223.9	4877083.0	297134.0
29–3	220.6	4877114.0	297164.0
29–4	221.1	4877119.0	297129.0
29–5	221.1	4877149.0	297157.0
29–6	225.8	4877146.0	297122.0
29–8	221.1	4877128.0	297143.0
29–9	221.1	4877092.9	297183.9
BO–9	221.0	4877161.8	297169.1
F8–2	229.2	4877031.6	297183.6
F8–3	221.0	4877098.8	297187.5
F8–6	229.1	4877028.4	297140.7
OHS7	220.4	4877043.3	297113.2
SC–1	223.0	4877070.0	297189.1
SC–2	222.0	4877082.3	297188.1
SC–3	220.1	4877124.8	297177.2
SC–4	220.8	4877151.8	297171.4
SC–5	221.1	4877176.1	297165.0
SC–7	220.7	4877117.8	297113.1
SC–8	220.5	4877130.1	297103.5
SC–9	221.0	4877070.2	297116.5
SC–10	222.1	4877033.5	297122.5
SC–11	221.8	4877019.1	297122.9
SC–13	229.0	4877052.4	297161.4
SC–14	222.0	4877041.9	297120.6

NOTES

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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 plan and General Arrangement provided in digital format by URS Canada Inc., (Drawing Files "Hwy400_plan.dwg" and "01_GA_July 10 2012.dwg") received November 13, 2013 and September 26, 2012.

LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA Jan. 13, 2015 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Jan. 13, 2015 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D–556

HWY. 400	PROJECT NO. 09–1111–0018		DIST. CENTRAL
SUBM'D. NK	CHKD. LCC	DATE: Mar 20, 2013	SITE:
DRAWN: CD/JFC	CHKD. LCC	APPD. JMAC	DWG. 1

PLOT DATE: January 13, 2015
FILENAME: T:\Projects\2009\09-1111-0018 (URS, York Region)\--DOC-- (South Canal-Bridge)(Stamps)\091111001800001.dwg

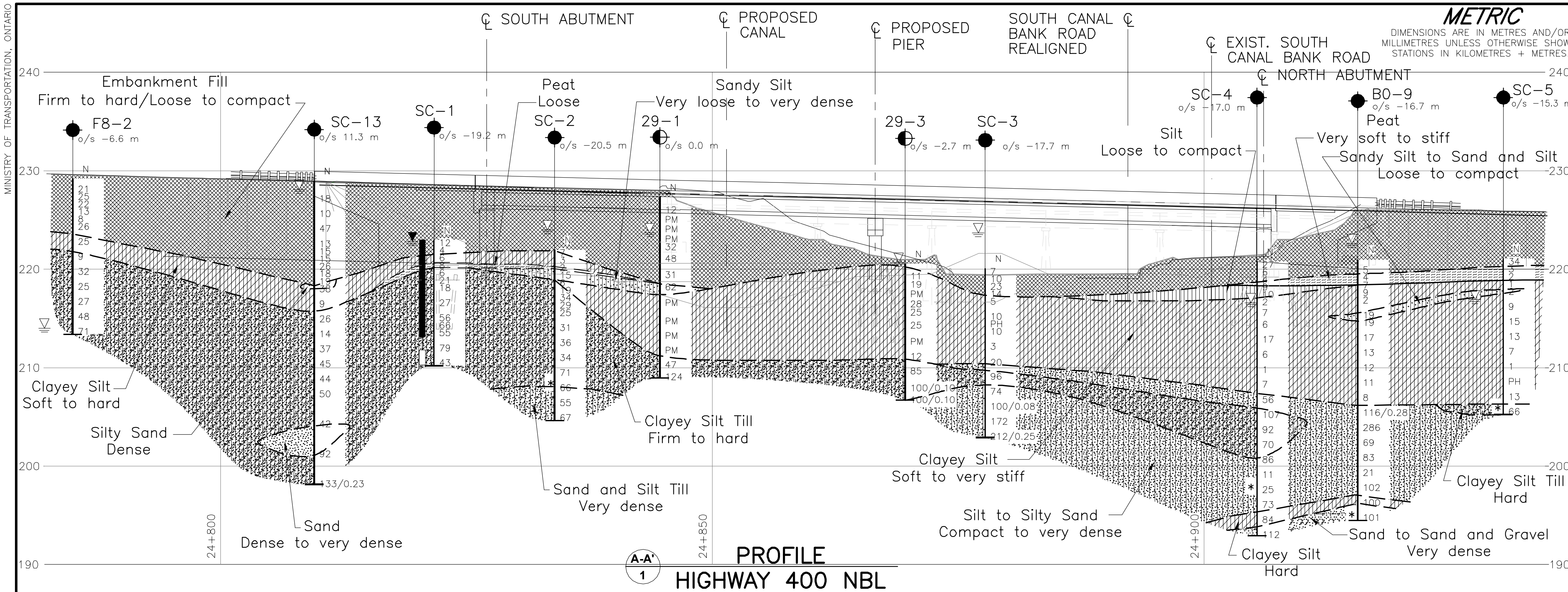
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2015–2004
GWP No. 2025–13–00

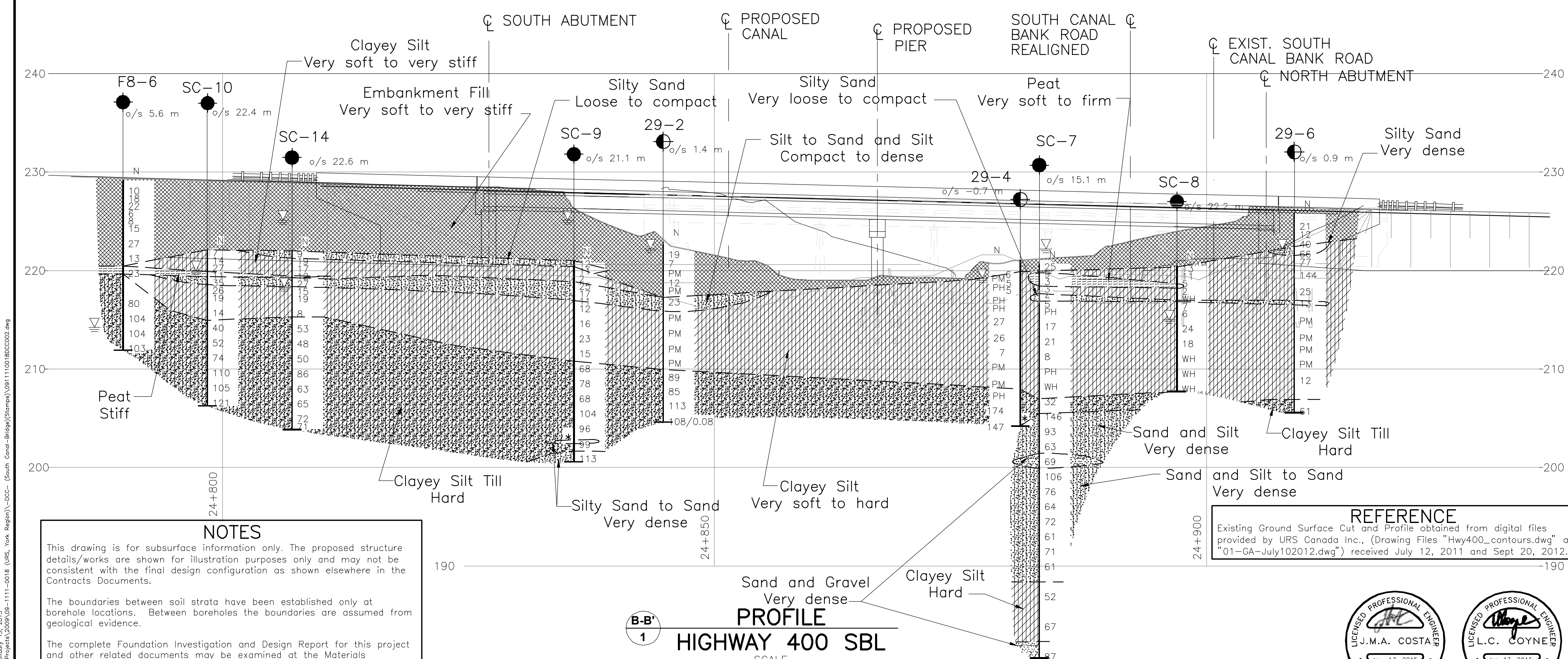
SOUTH CANAL BRIDGES
HIGHWAY 400 WIDENING
SOIL STRATA

SHEET

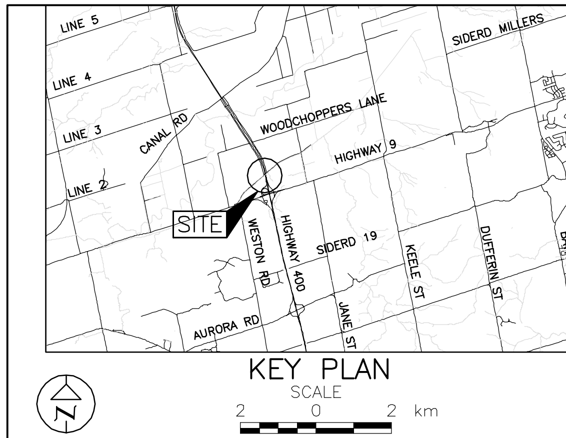
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



A-A'
1
PROFILE
HIGHWAY 400 NBL
SCALE 5 0 5 10 m



B-B'
1
PROFILE
HIGHWAY 400 SBL
SCALE 5 0 5 10 m



KEY PLAN
SCALE
2 0 2 km

LEGEND

- Borehole – Current Investigation by Golder
- Borehole – Previous Investigation (Geocres No. 31D–029)
- 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 June 12, 2012
- WL upon completion of drilling
- * Approximate depth at which artesian groundwater pressure encountered

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
29–1	227.4	4877089.0	297166.0
29–2	223.9	4877083.0	297134.0
29–3	220.6	4877114.0	297164.0
29–4	221.1	4877119.0	297129.0
29–6	225.8	4877146.0	297122.0
BO–9	221.0	4877161.8	297169.1
F8–2	229.2	4877031.6	297183.6
F8–6	229.1	4877028.4	297140.7
SC–1	223.0	4877070.0	297189.1
SC–2	222.0	4877082.3	297188.1
SC–3	220.1	4877124.8	297177.2
SC–4	220.8	4877151.8	297171.4
SC–5	221.1	4877176.1	297165.0
SC–7	220.7	4877117.8	297113.1
SC–8	220.5	4877130.1	297103.5
SC–9	221.0	4877070.2	297116.5
SC–10	222.1	4877033.5	297122.5
SC–13	229.0	4877052.4	297161.4
SC–14	222.0	4877041.9	297120.6

NOTES

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REFERENCE

Existing Ground Surface Cut and Profile obtained from digital files provided by URS Canada Inc., (Drawing Files "Hwy400_contours.dwg" and "01-GA-July102012.dwg") received July 12, 2011 and Sept 20, 2012.

LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA Jan. 13, 2015 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Jan. 13, 2015 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D–556

HWY. 400	PROJECT NO. 09–1111–0018		DIST. CENTRAL
SUBM'D. NK	CHKD. LCC	DATE: Mar 20, 2013	SITE:
DRAWN: LL/JFC	CHKD. LCC	APPD. JMAC	DWG. 2

MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: January 13, 2015

MINISTRY OF TRANSPORTATION, ONTARIO

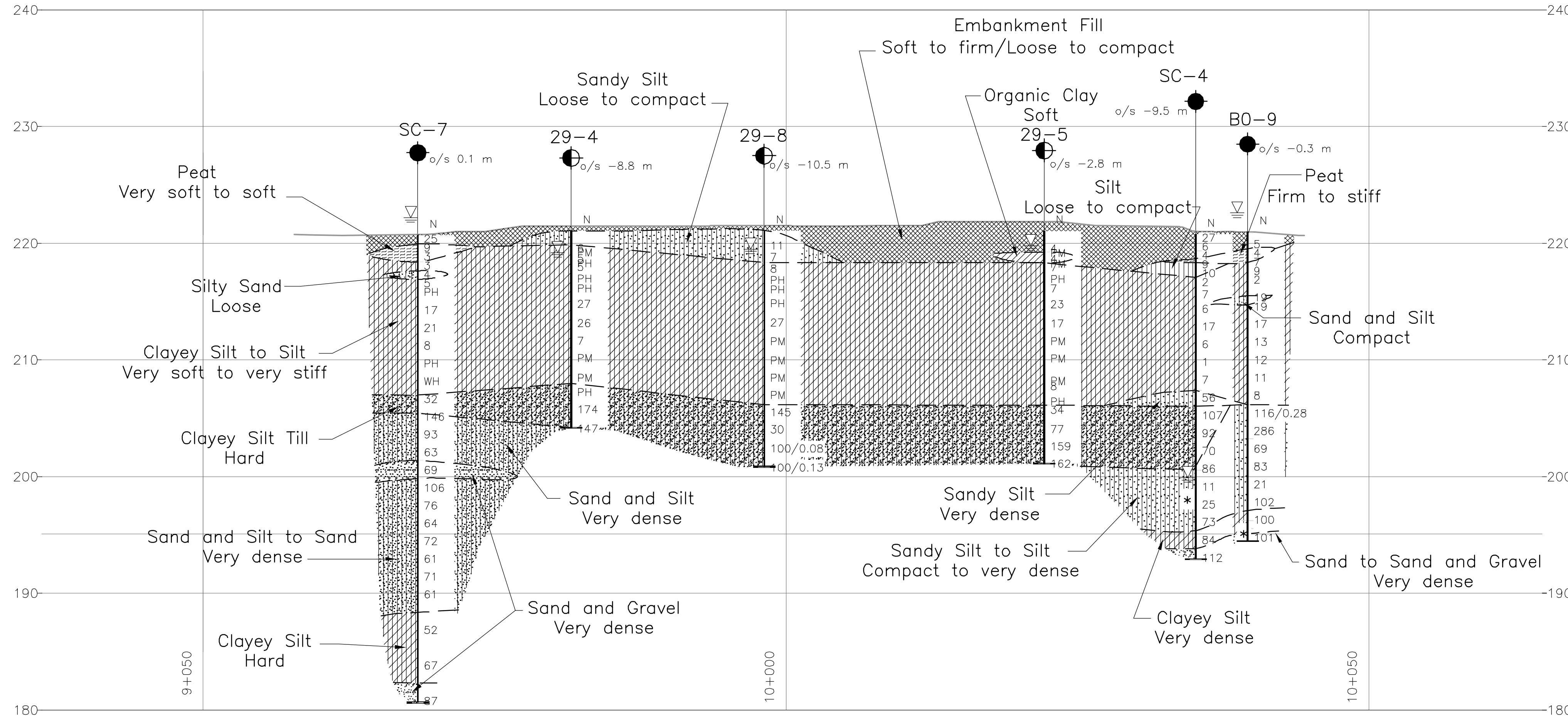
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2015–2004
GWP No. 2025–13–00

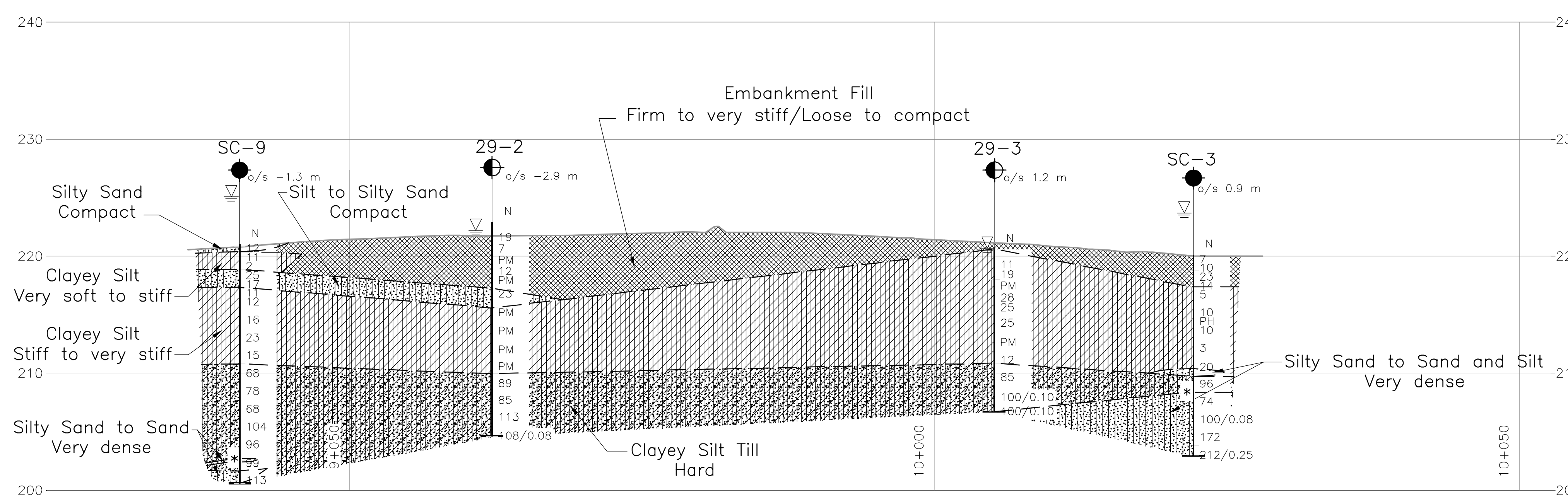
SOUTH CANAL BRIDGES
HIGHWAY 400 WIDENING
SOIL STRATA

SHEET

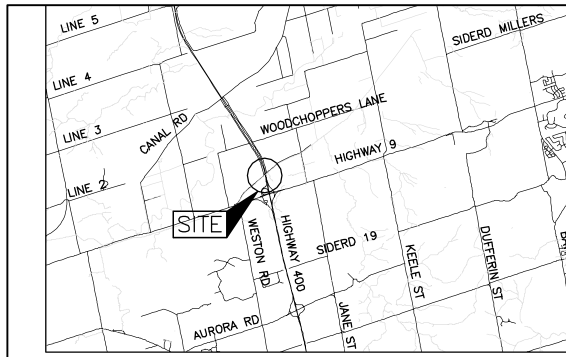
Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



C-C' / 1
CROSS–SECTION NORTH ABUTMENT
SCALE
5 0 5 10 m



D-D' / 1
CROSS–SECTION CENTRE PIER
SCALE
5 0 5 10 m



KEY PLAN
SCALE
2 0 2 km

LEGEND

- Borehole – Current Investigation by Golder
- Borehole – Previous Investigation (Geocres No. 31D–029)
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- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
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BOREHOLE CO–ORDINATES

No.	ELEVATION	NORTHING	EASTING
29–2	223.9	4877083.0	297134.0
29–3	220.6	4877114.0	297164.0
29–4	221.1	4877119.0	297129.0
29–5	221.1	4877149.0	297157.0
29–8	221.1	4877128.0	297143.0
B0–9	221.0	4877161.8	297169.1
SC–3	220.1	4877124.8	297177.2
SC–4	220.8	4877151.8	297171.4
SC–7	220.7	4877117.8	297113.1
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LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA Jan. 13, 2015 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Jan. 13, 2015 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31D–556

HWY. 400		PROJECT NO. 09–1111–0018	DIST. CENTRAL
SUBM'D. NK	CHKD. LCC	DATE: Mar. 20, 2013	SITE:
DRAWN: LL/JFC	CHKD. LCC	APPD. JMAC	DWG. 3

PLOT DATE: January 13, 2015
FILENAME: T:\Projects\2009\09-1111-0018 (URS, York Region)_DOC-(South Canal-Bridge)(Stamps)\09111100180C0003.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

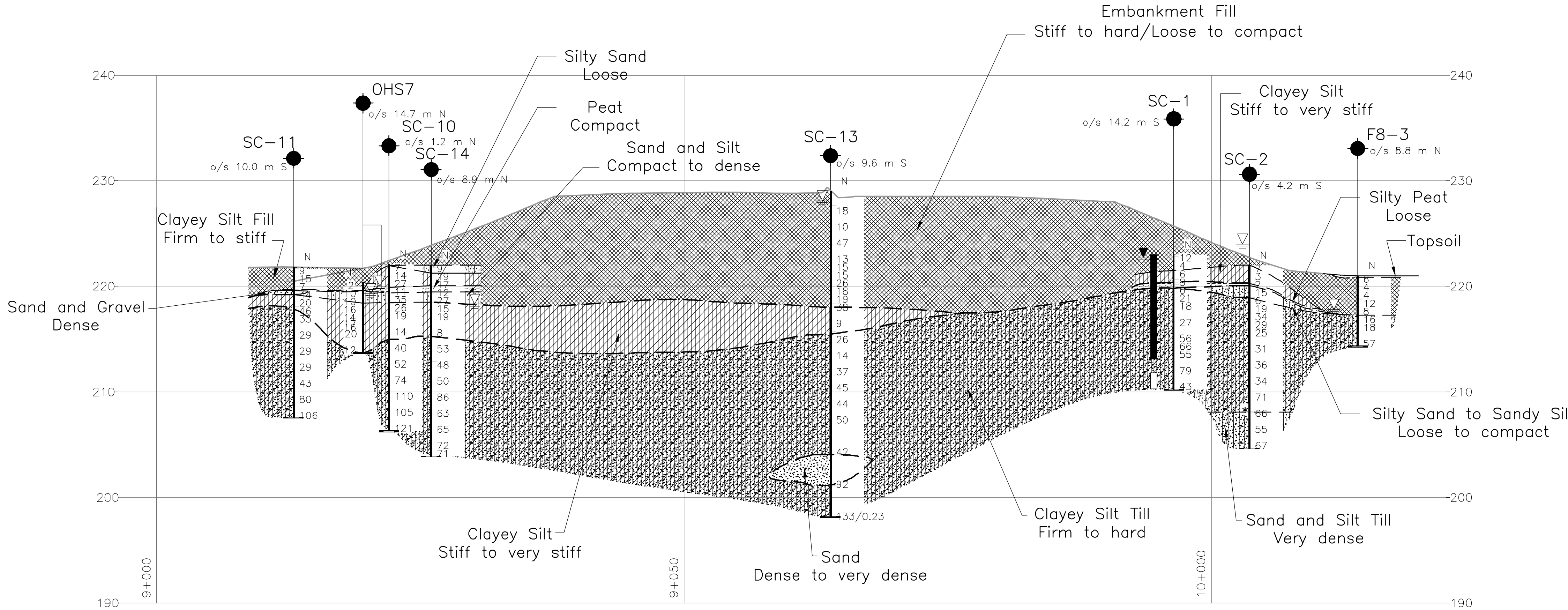
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2015–2004
GWP No. 2025–13–00

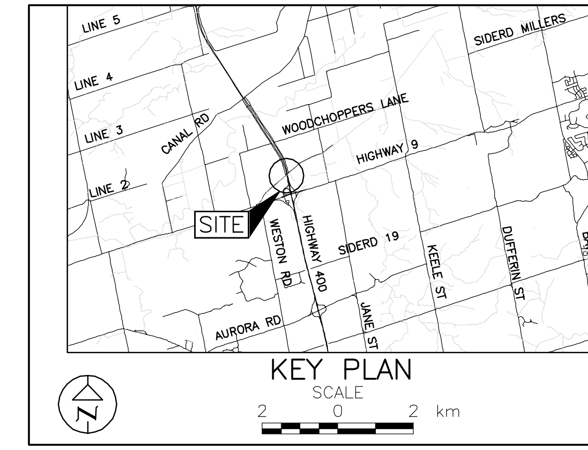
SOUTH CANAL BRIDGES
HIGHWAY 400 WIDENING
SOIL STRATA

SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



E-E' 1 CROSS SECTION SOUTH ABUTMENT
SCALE
5 0 5 10 m



KEY PLAN
SCALE
2 0 2 km

LEGEND

- Borehole – Current Investigation by Golder
- Borehole – Previous Investigation (Geocres No. 31D–029)
- Seal
- Piezometer
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- WL in piezometer, measured on June 12, 2012
- WL upon completion of drilling
- * Approximate depth at which Artesian Groundwater pressure measured

BOREHOLE CO-ORDINATES

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SC–1	223.0	4877070.0	297189.1
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SC–10	222.1	4877033.5	297122.5
SC–11	221.8	4877019.1	297122.9
SC–13	229.0	4877052.4	297161.4
SC–14	222.0	4877041.9	297120.6

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REFERENCE

Base plans provided in digital format by URS Canada Inc., (Drawing File "Hwy400_bgd.dwg" and "01–GA–July102012.dwg") received October 25, 2011 and July 10, 2012.

LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA Jan. 13, 2015 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Jan. 13, 2015 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

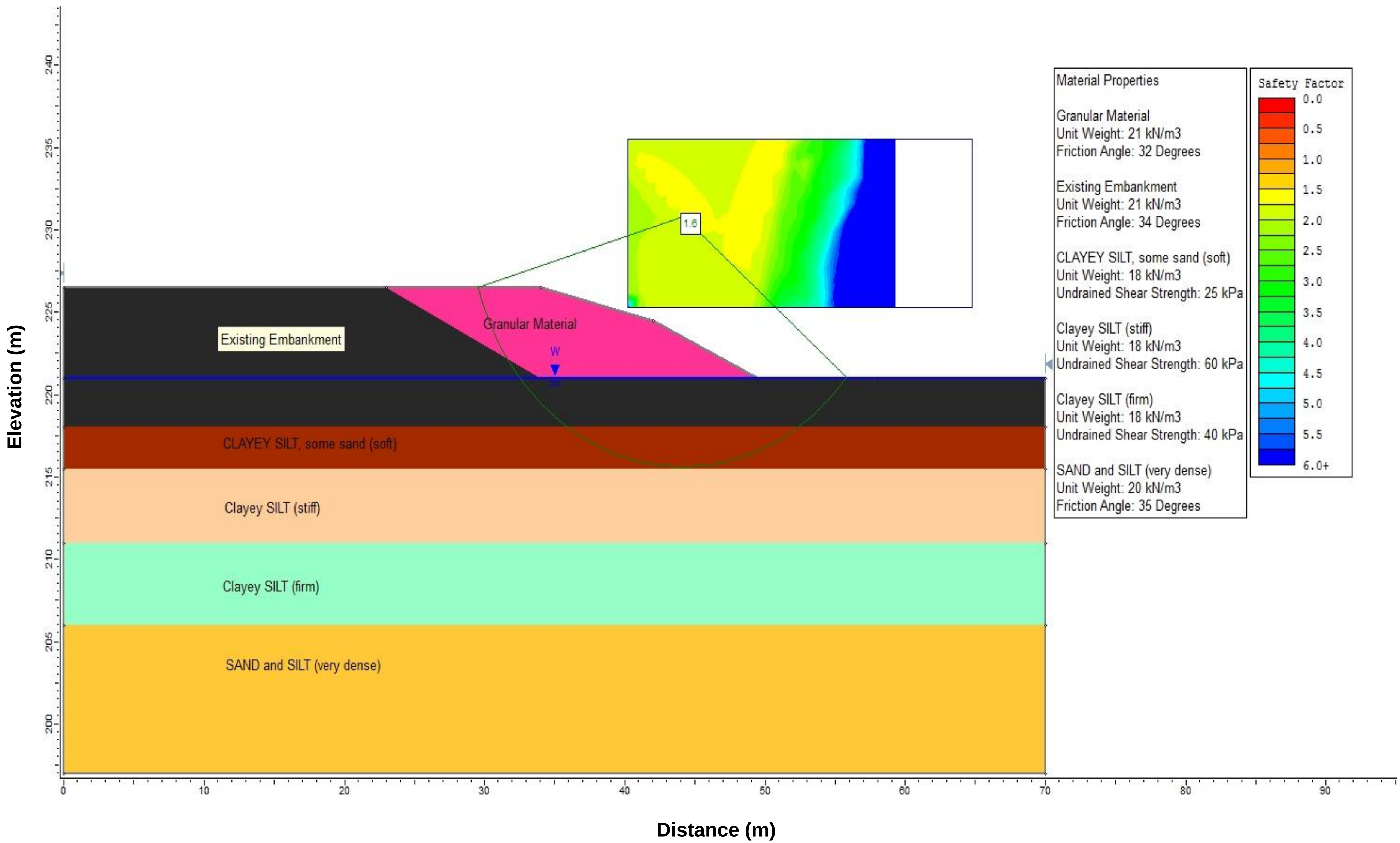
Geocres No. 31D–556

HWY. 400		PROJECT NO. 09–1111–0018	DIST. CENTRAL
SUBM'D. NK	CHKD. LCC	DATE: Mar. 20, 2013	SITE:
DRAWN: LL/JFC	CHKD. LCC	APPD. JMAC	DWG. 4

PLOT DATE: January 13, 2015
FILENAME: T:\Projects\2009\09-1111-0018 (URS, York Region)\-DOC- (South Canal-Bridge)(Stamps)\0911110018DC0004.dwg

Static Global Stability – North Approach Embankment Short-term (Undrained) Conditions

Figure 1



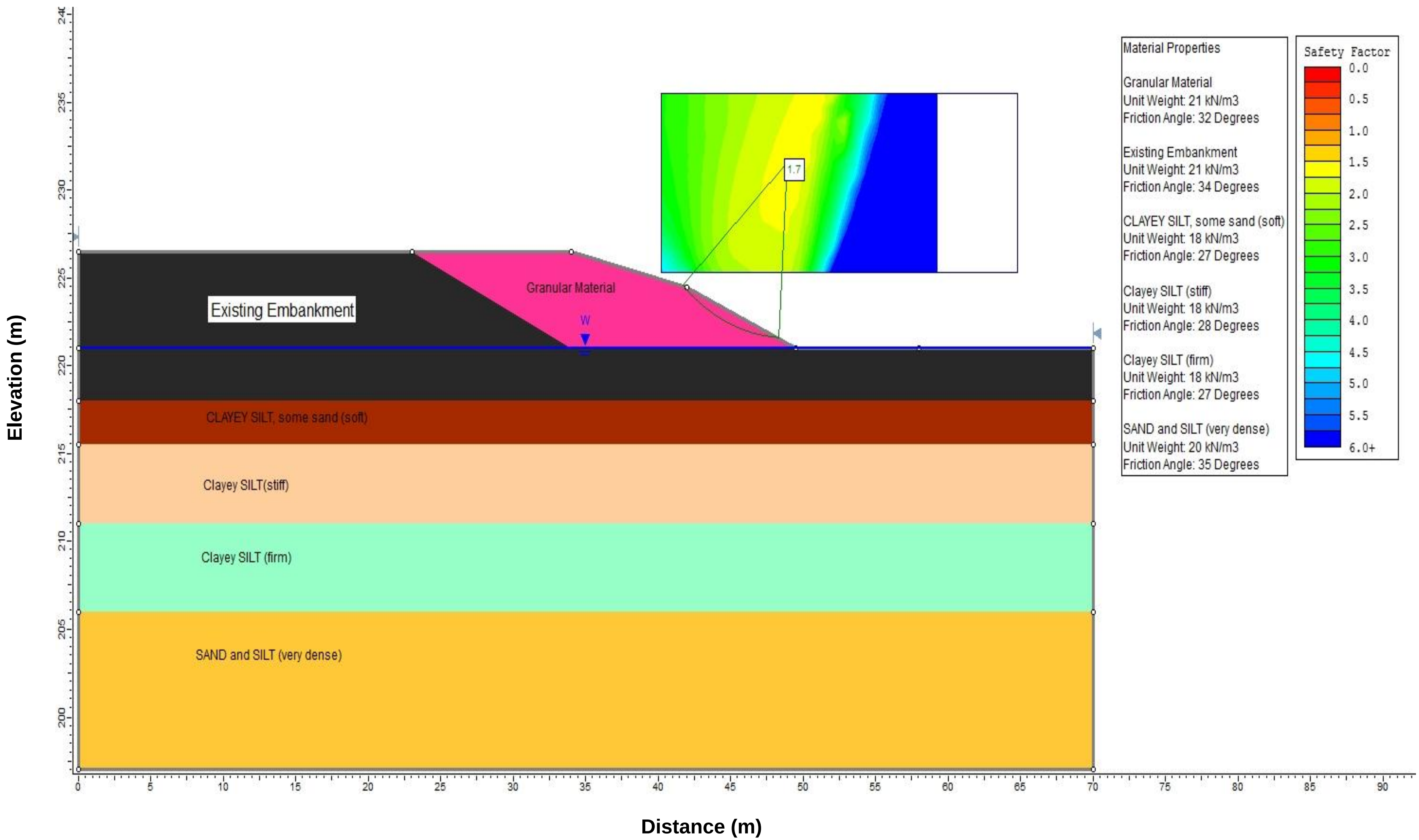
Date: October 2012
Project No: 09-1111-0018

Analysis By: AV **Reviewed By:** LCC

Golder Associates

Static Global Stability – North Approach Embankment Long-term (Drained) Conditions

Figure 2



Date: October 2012

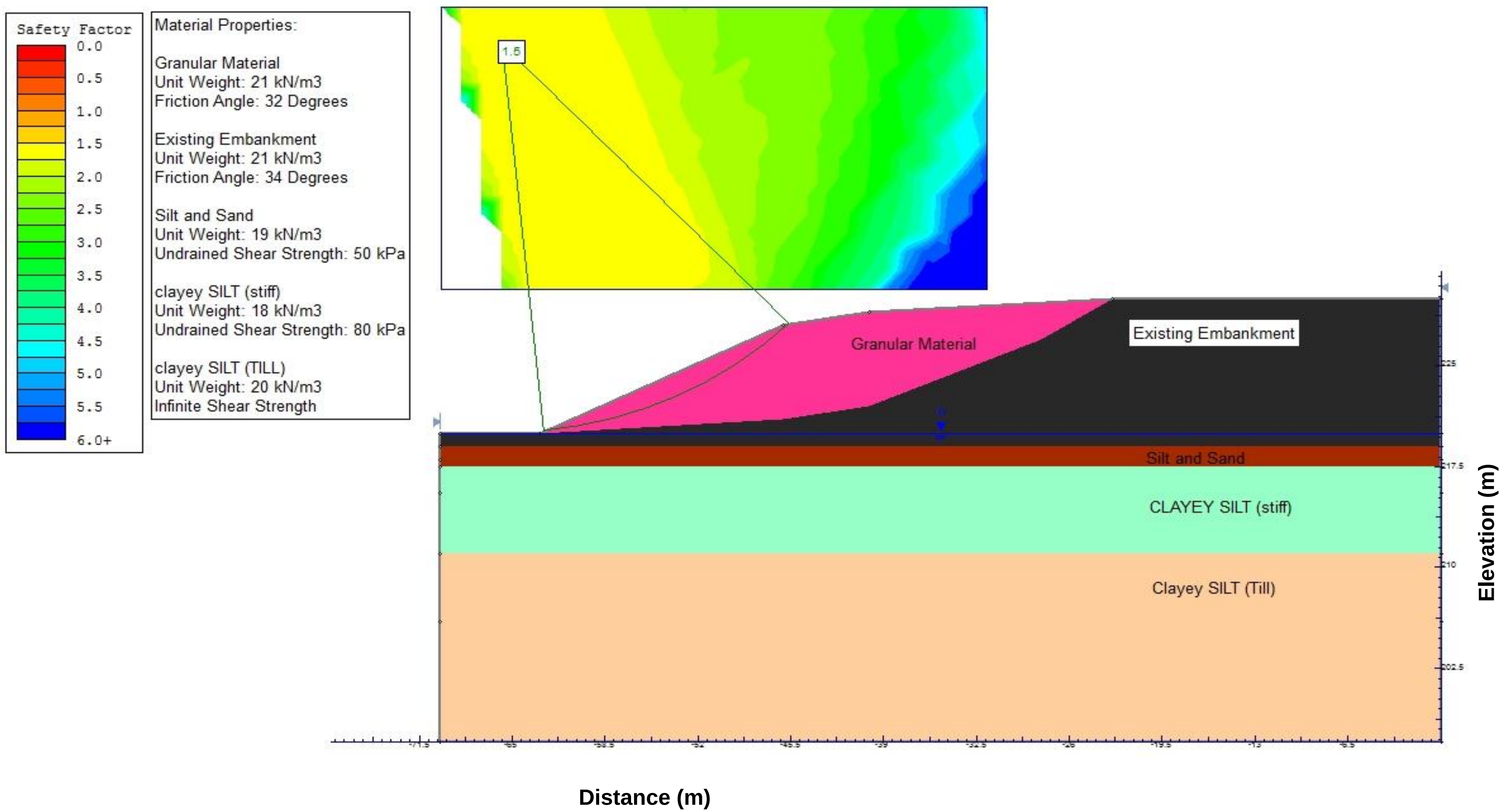
Project No: 09-1111-0018

Analysis By: AV **Reviewed By:** LCC

Golder Associates

Static Global Stability – South Approach Embankment Short-term (Undrained) Conditions

Figure 3



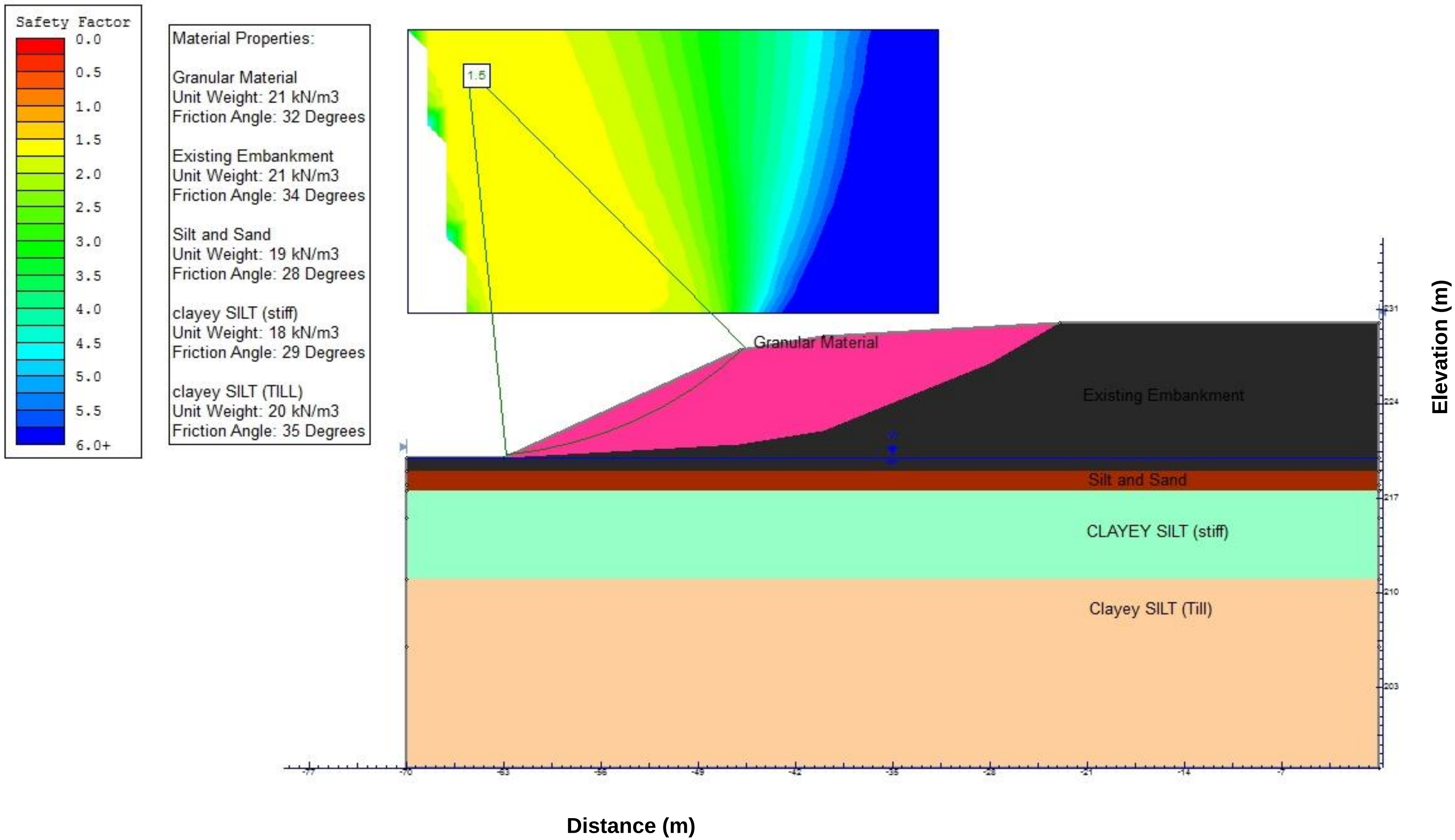
Date: October 2012
Project No: 09-1111-0018

Analysis By: AV **Reviewed By:** LCC

Golder Associates

Static Global Stability – South Approach Embankment Long-term (Drained) Conditions

Figure 4



Date: October 2012

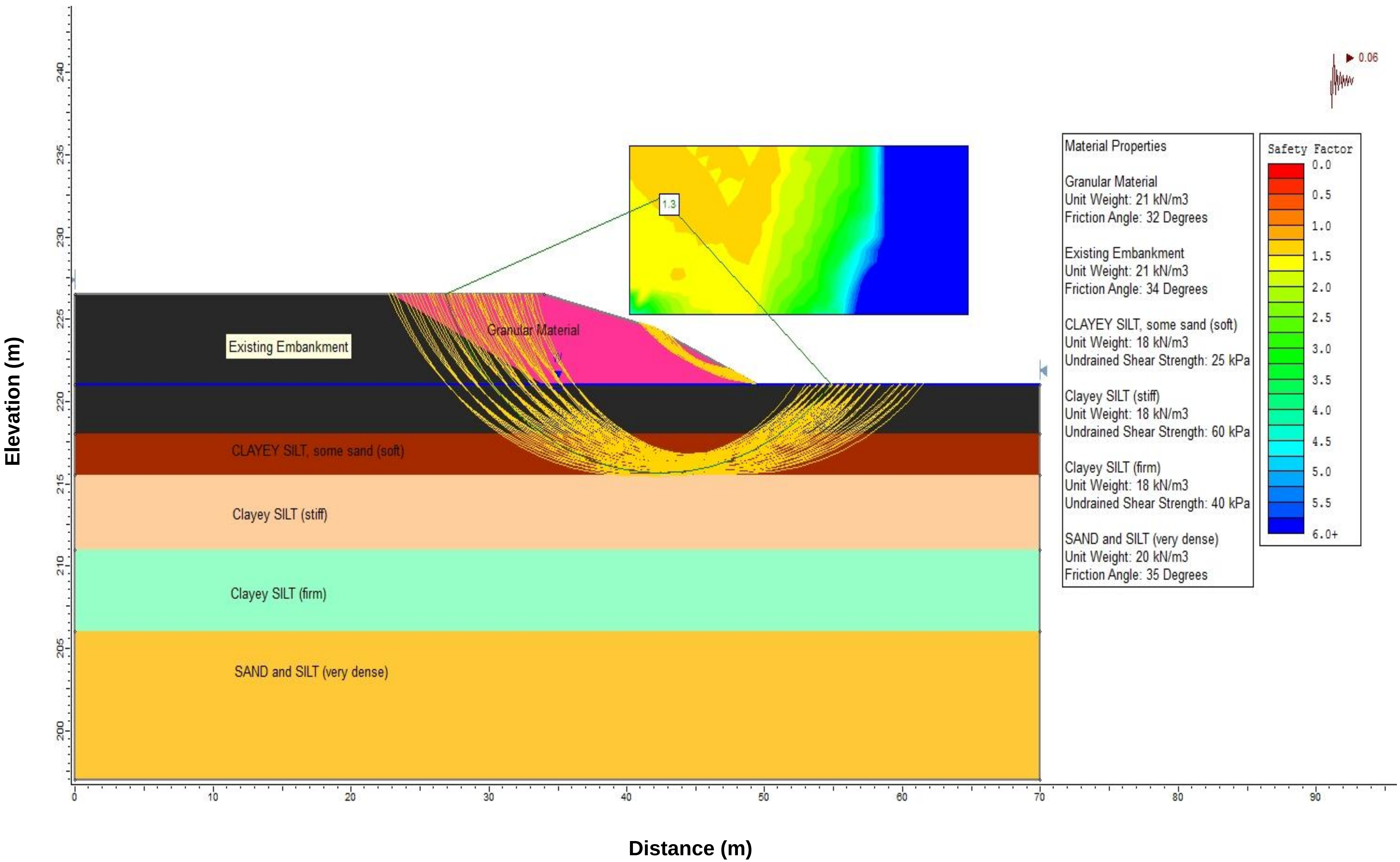
Project No: 09-1111-0018

Analysis By: AV **Reviewed By:** LCC

Golder Associates

Seismic Stability – North Approach Embankment

Figure 5



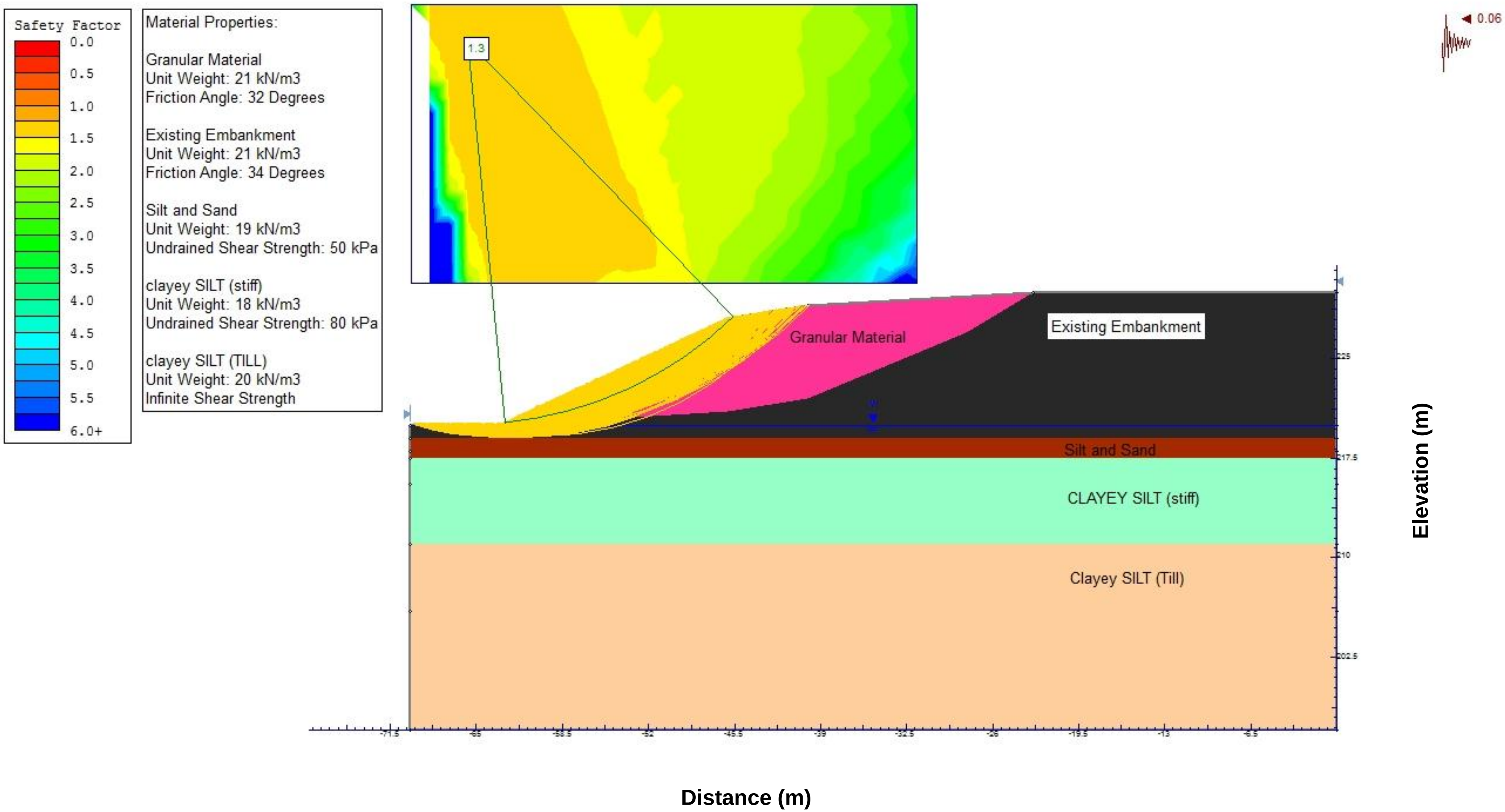
Date: October 2012
Project No: 09-1111-0018

Analysis By: AV **Reviewed By:** LCC

Golder Associates

Seismic Stability – South Approach Embankment

Figure 6

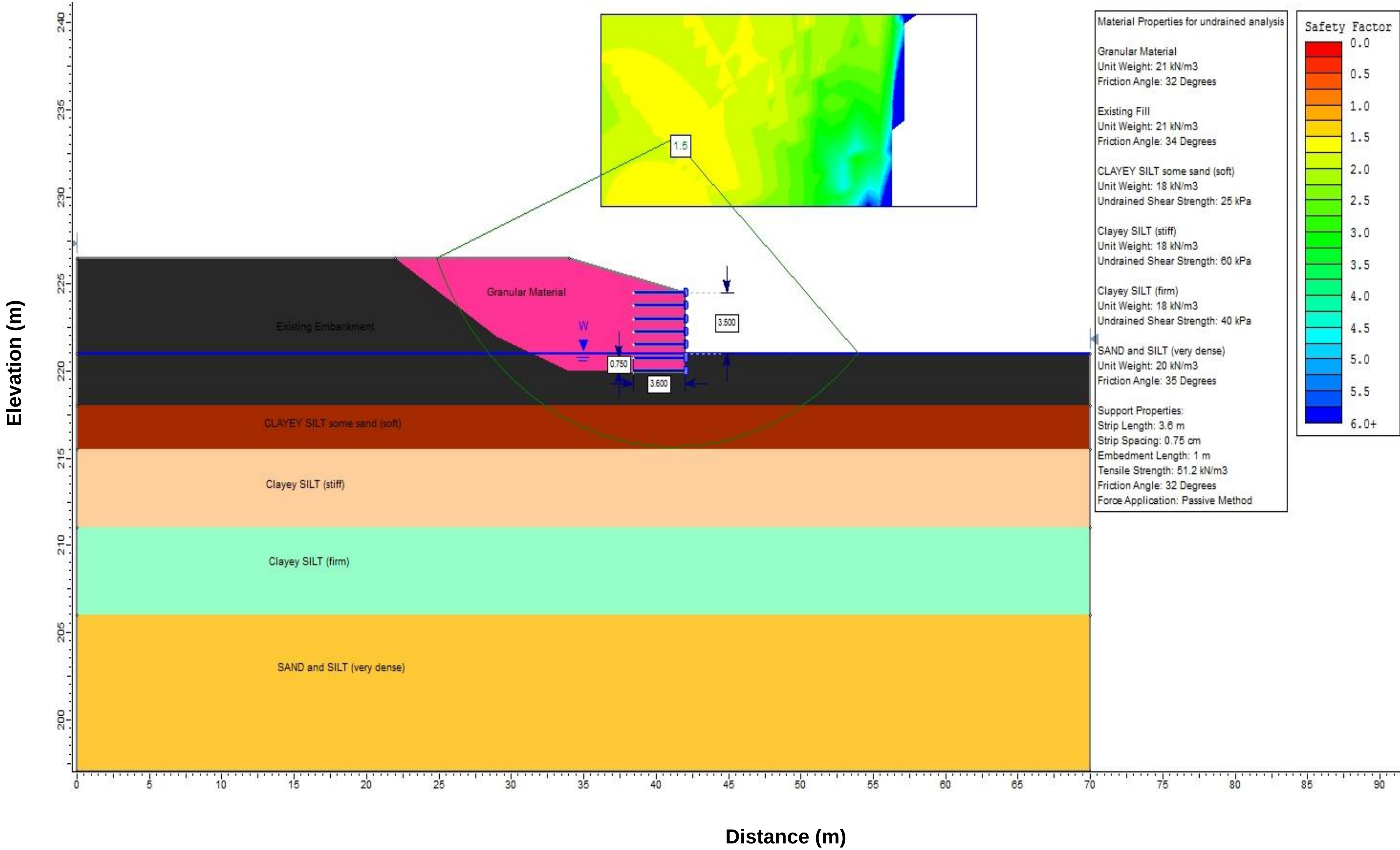


Date: October 2012
Project No: 09-1111-0018

Analysis By: AV **Reviewed By:** LCC

Static Global Stability – RSS Walls at North Approach Short-Term (Undrained) Conditions

Figure 7

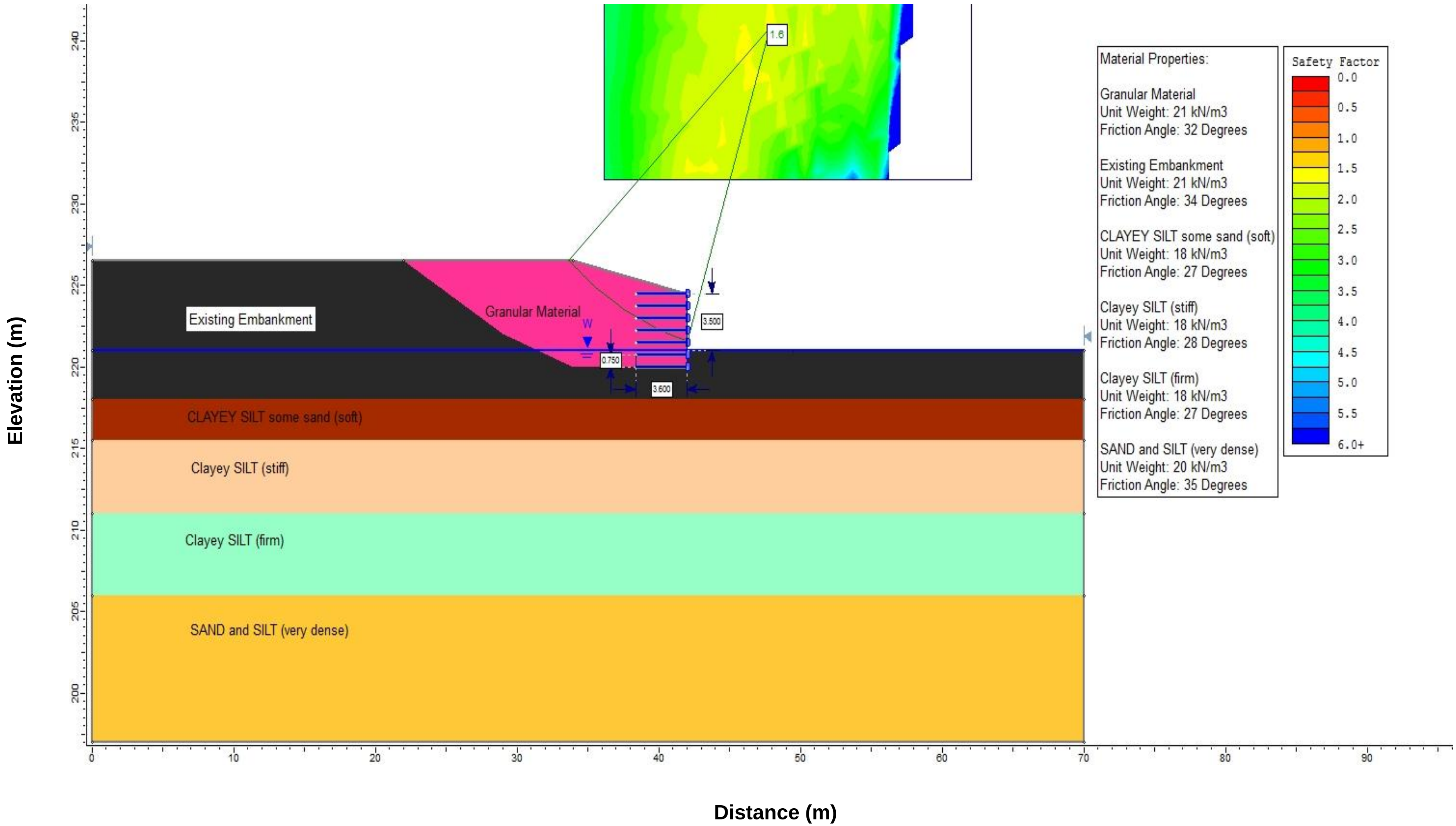


Date: October 2012
Project No: 09-1111-0018

Analysis By: AV **Reviewed By:** LCC

Static Global Stability – RSS Walls at North Approach Long-Term (Drained) Conditions

Figure 8



Date: October 2012

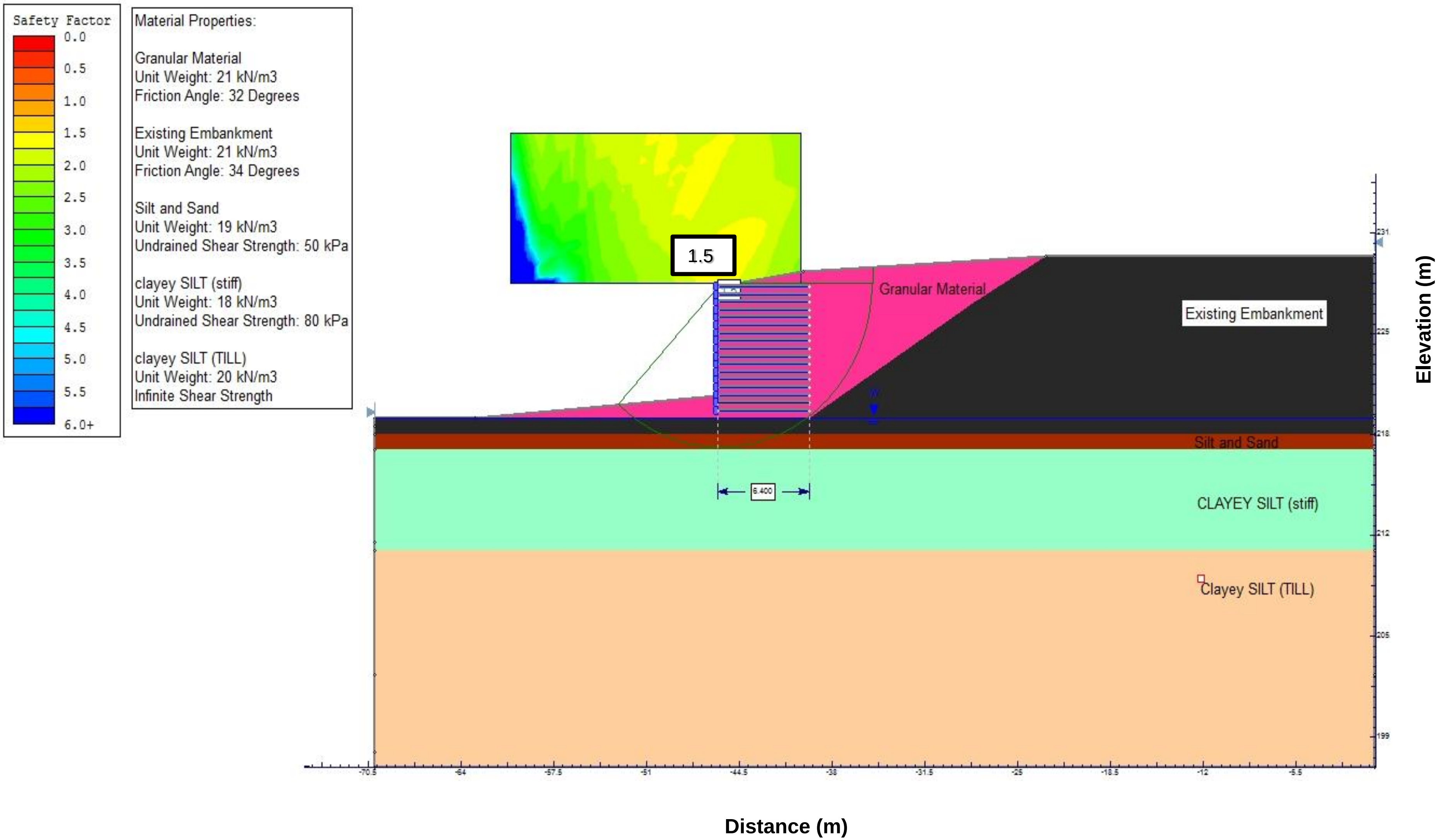
Project No: 09-1111-0018

Analysis By: AV **Reviewed By:** LCC

Golder Associates

Static Global Stability – RSS at South Approach Short-Term (Undrained) Conditions

Figure 9



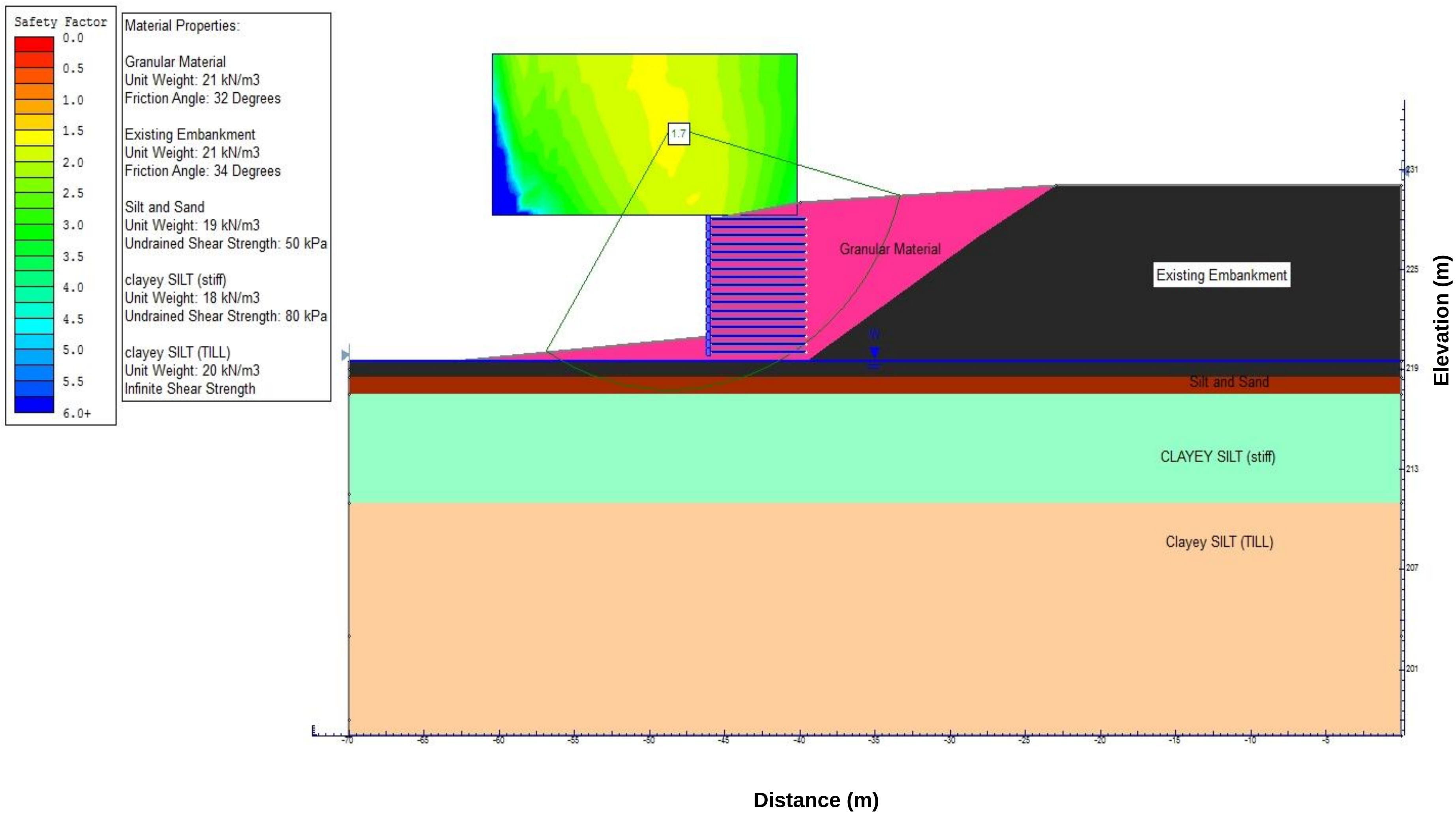
Date: October 2012
Project No: 09-1111-0018

Analysis By: AV **Reviewed By:** LCC

Golder Associates

Static Global Stability – RSS at South Approach
Long-Term (Drained) Conditions

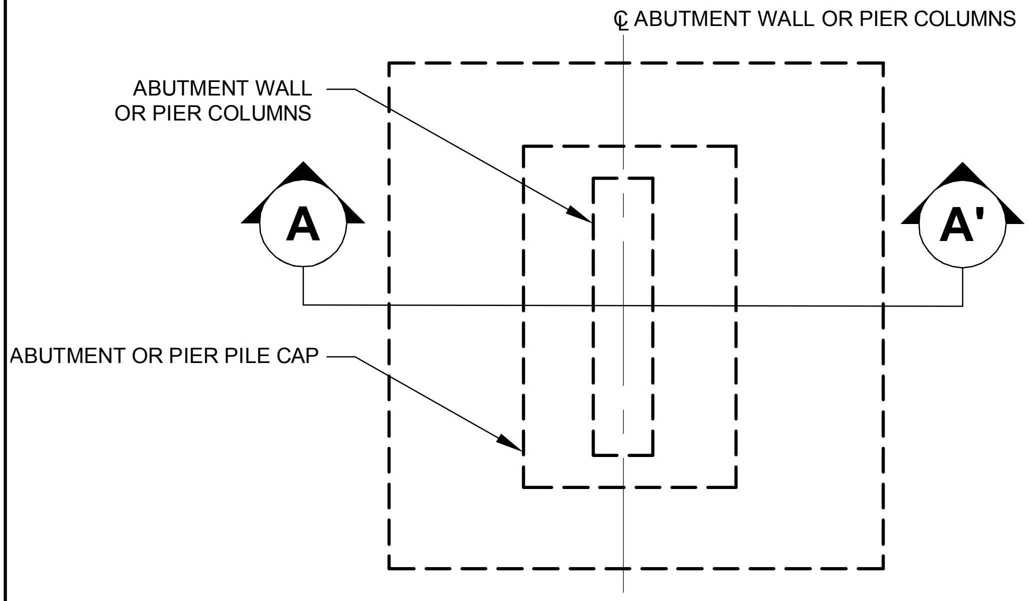
Figure 10



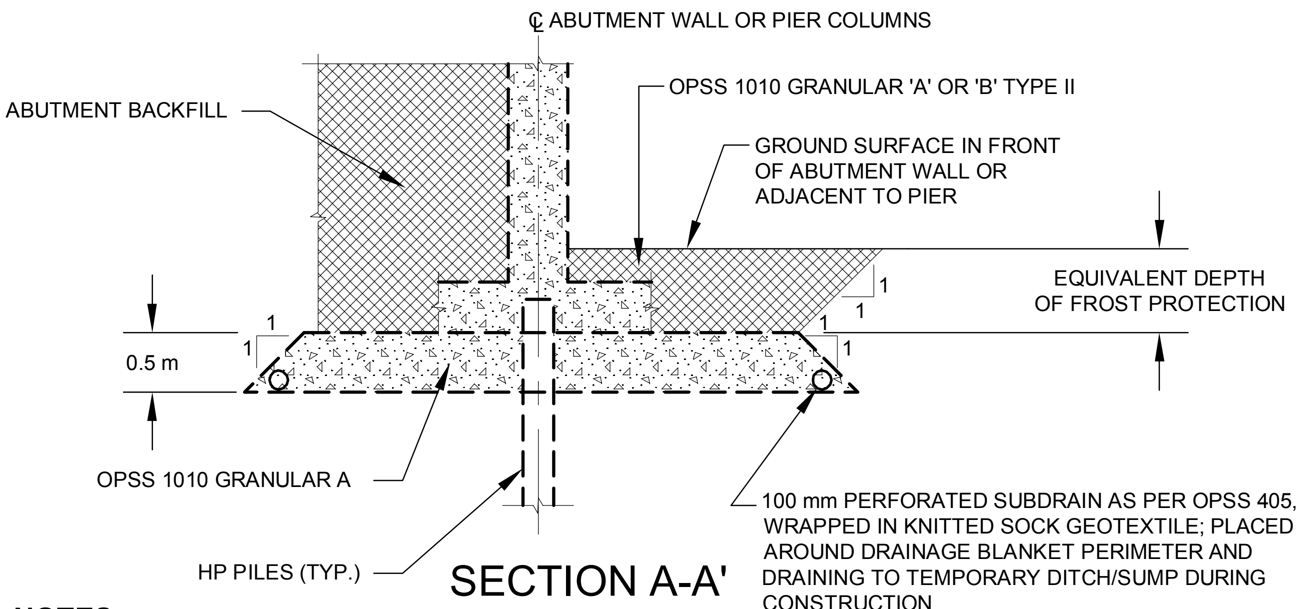
Date: October 2012
Project No: 09-1111-0018

Analysis By: AV **Reviewed By:** LCC

Golder Associates



PLAN



SECTION A-A'

NOTES:

1. THE DRAINAGE BLANKETS SHOULD BE IN PLACE PRIOR TO PILE DRIVING, AND SHOULD EXTEND OVER THE LIMITS OF THE NEW PILE CAPS, AS WELL AS OVER THE FULL EXTENT OF THE EXISTING ABUTMENT AND PIER FOUNDATION AREAS FOLLOWING REMOVAL OF EXISTING PILE CAPS AND CUT-OFF THE EXISTING PILES.

2. IF BLANKET IS DISTURBED DURING PILE DRIVING, THE BLANKET SHOULD BE RESTORED TO THE DETAILS SHOWN ON THIS FIGURE AFTER THE COMPLETION OF THE PILE DRIVING.

3. DRAINAGE BLANKET SHOULD EXTEND A MINIMUM OF 1 m HORIZONTALLY BEYOND EACH OF THE PILES.

NOT TO SCALE

PROJECT

HIGHWAY 400 WIDENING
SOUTH CANAL BRIDGES
GWP 2835-02-00

TITLE

GRANULAR FILTER BLANKET DETAILS

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		09-1111-0018	FILE No. 0911110018JA011.dwg	
DESIGN			SCALE AS SHOWN	REV.
CAD	JFC	Jul. 14, 2015	FIGURE No.	
CHECK	TVA	Jul. 14, 2015		
REVIEW	LCC	Jul. 14, 2015	11	

APPENDIX A

Borehole Records from Current Investigation

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
FoS	factor of safety

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
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ 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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 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) Non-Cohesive (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

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

Foundation Design

Golder Associates

RECORD OF BOREHOLE No B0-9

SHEET 1 OF 2

METRIC

PROJECT 09-1111-0018
G.W.P. 2835-02-00
LOCATION N 4877161.8 ;E 297169.1
ORIGINATED BY TZ
DIST Central HWY 400
BOREHOLE TYPE 108 mm Inside Diameter Hollow Stem Augers
COMPILED BY NK
DATUM Geodetic
DATE November 14-15, 2011
CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH / DCPT	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
221.0	GROUND SURFACE											
0.0 / 0.1	TOPSOIL											
	CLAYEY SILT, trace gravel, trace sand, containing organics; Firm; Brown; Moist	1	SS	5	220							
219.5 / 1.5	PEAT (Amorphous), containing rootlets and decomposed wood fragments, containing clayey silt seams; Firm; Black; Moist	2	SS	4	219							
		3	SS	7								
218.4 / 2.6	CLAYEY SILT, trace to some sand; Firm to stiff; Grey; Moist	4	SS	9	218							
		5	SS	2	217				0	18	72	10
					216	+ 3; + 3						
215.5 / 5.5	Silty SAND, some gravel, trace clay; Compact; Grey; Wet	6	SS	19	215							
214.7 / 6.3	CLAYEY SILT, trace sand; Stiff to very stiff; Grey; Wet	7A / 7B	SS	19					19	47	29	5
					214							
		8	SS	17	213							
					212							
		9	SS	13	211							
		10	SS	12	210				0	2	71	27
					209							
		11	SS	11	208							
		12	SS	8	207	+ 3						
206.2 / 14.8												

Continued Next Page

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

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RECORD OF BOREHOLE No B0-9

SHEET 2 OF 2

METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00

LOCATION N 4877161.8 ;E 297169.1

ORIGINATED BY TZ

DIST Central HWY 400

BOREHOLE TYPE 108 mm Inside Diameter Hollow Stem Augers

COMPILED BY NK

DATUM Geodetic

DATE November 14-15, 2011

CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT, some clay Very dense Grey Moist	13	SS	116/0.28		0 0 87 13
					205	
		14	SS	286	204	
203.2 17.8	SAND and SILT, trace to some clay Compact to very dense Grey Wet				203	
		15	SS	69		0 32 56 12
					202	
		16	SS	83	201	
					200	
		17	SS	21		
					199	
		18	SS	102	198	
197.1 23.9	CLAYEY SILT with sand, some gravel Hard Grey Wet				197	
		19A	SS	100		16 25 44 15
196.2 24.8	SAND, trace gravel, trace silt Very dense Grey Wet	19B			196	
195.5 25.5	SAND and GRAVEL Very dense Grey Wet				195	
		20	SS	101		
194.5 26.5	END OF BOREHOLE					

NOTES:

1. Artesian conditions encountered below a depth of 25.9 m (Elev. 195.1 m).

2. Water level measured inside casing at 1.6 m above ground surface (Elev. 222.6 m) on completion of drilling.

3. Borehole abandoned using cement grout with 3 m of bentonite placed above grout immediately below ground surface.

Column heads not used in the table: GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

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

Foundation Design

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RECORD OF BOREHOLE No F8-2

SHEET 1 OF 2

METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00

LOCATION N 4877031.6 ;E 297183.6

ORIGINATED BY AM

DIST Central HWY 400

BOREHOLE TYPE D-90 Track Mount, 108 mm Inside Diameter Hollow Stem Augers

COMPILED BY CS

DATUM Geodetic

DATE April 1, 2011

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
229.2	GROUND SURFACE						
0.0	ASPHALT				229		
228.9 / 0.3	Silty sand and gravel (FILL)						
228.4 / 0.8	Silty sand, trace clay, trace gravel (FILL) Compact Grey Moist	1	SS	21	228		
227.7 / 1.5	Clayey silt, some sand, trace to some gravel (FILL) Stiff to very stiff Brown and grey Moist	2	SS	25	227		
		3	SS	22			
		4	SS	13	226		
		5	SS	8	225		3 37 49 11
		6	SS	26	224		
223.6 / 5.6	CLAYEY SILT, some sand, trace gravel, slightly organic Very stiff Brown and grey Moist	7	SS	25	223		O.C. = 1.8 3 17 61 19
222.0 / 7.2	CLAYEY SILT, trace to some sand, trace gravel (TILL) Stiff to hard Brown Moist	8	SS	9	222, 221		
		9	SS	32	220		6 27 47 20
		10	SS	25	219, 218		
		11	SS	27	217		
		12	SS	48	216, 215		

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/m^3

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No F8-2

SHEET 2 OF 2

METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00

LOCATION N 4877031.6 ;E 297183.6

ORIGINATED BY AM

DIST Central HWY 400

BOREHOLE TYPE D-90 Track Mount, 108 mm Inside Diameter Hollow Stem Augers

COMPILED BY CS

DATUM Geodetic

DATE April 1, 2011

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ 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 ---										
	Becoming wet at a depth of 14.8 m					▽	214				
213.4	Augers grinding at a depth of 15.2 m		13	SS	71				○		
15.8	END OF BOREHOLE										
	NOTE:										
	1. Water level in open borehole at a depth of 15.2 m below ground surface (Elev. 214.0 m) upon completion of drilling.										

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

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

Golder Associates

RECORD OF BOREHOLE No F8-3

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0018
G.W.P. 2835-02-00 LOCATION N 4877098.8 ;E 297187.5 ORIGINATED BY AM
DIST Central HWY 400 BOREHOLE TYPE D-25 Track Mount, 108 mm Inside Diameter Hollow Stem Augers COMPILED BY TT
DATUM Geodetic DATE January 18, 2011 CHECKED BY SMM

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	GR	SA	SI	CL
221.0	GROUND SURFACE									
0.0	TOPSOIL									
0.2	Clayey silt, trace to some sand, trace gravel, slightly organic, rootlets and wood fragments (FILL) Firm Brown Moist	1	SS	6						
		2	SS	4						
		3	SS	4		plotted: w_P, w, w_L				
218.8 / 2.2	Clayey silt, trace sand (FILL) Stiff Brown Wet Grey clayey silt seams between depths of 2.7 m and 2.8 m	4	SS	12		plotted: w	0	3	69	28
217.7		5	SS	8	∇ (Elev. 217.8 m)					
217.3 / 3.7	CLAYEY SILT, trace sand, containing rootlets Stiff Grey Moist									
	CLAYEY SILT with sand (TILL) Very stiff to hard Grey Moist	6	SS	16		plotted: w				
		7	SS	18		plotted: w_P, w, w_L	0	26	53	21
214.3		8	SS	57						
6.7	END OF BOREHOLE									

NOTE:

1. Water level in open borehole at a depth of 3.2 m below ground surface (Elev. 217.8 m) upon completion of drilling.

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (220, 219, 218, 217, 216, 215); 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

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RECORD OF BOREHOLE No F8-6

SHEET 1 OF 2

METRIC

PROJECT 09-1111-0018			
G.W.P. 2835-02-00	LOCATION N 4877028.4 ;E 297140.7		ORIGINATED BY AM
DIST Central	HWY 400	BOREHOLE TYPE D-90 Truck Mount, 108 mm Inside Diameter Hollow Stem Auger	COMPILED BY CS
DATUM Geodetic		DATE March 31, 2011	CHECKED BY SMM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
229.1	GROUND SURFACE				
0.0	ASPHALT				
0.1	Silty sand and gravel (FILL)				
228.3 / 0.8	Clayey silt, some sand, trace gravel (FILL) Stiff Brown Moist	1	SS	10	
227.6 / 1.5	Sand and gravel, some silt, trace clay (FILL) Compact Brown Moist	2	SS	18	
		3	SS	22	40 45 11 4
226.1 / 3.0	Clayey silt, trace sand, trace gravel (FILL) Firm to very stiff Brown to grey Moist Augers grinding and spoon bouncing at a depth of 3.3 m	4	SS	6	
		5	SS	8	
		6	SS	15	1 17 57 25
		7	SS	27	
	Silt and sand interlayers at a depth of 7.9 m	8	SS	13	
220.4 / 8.7	Organic Sandy SILT, some clay, trace gravel Stiff Brown to black Moist	9A	SS	23	O.C. = 7.0 %
219.6 / 9.5	Sandy SILT to Silty SAND, trace to some clay, trace gravel (TILL) Compact to very dense Grey Moist	9B			10 28 46 16
	Augers grinding and spoon bouncing at a depth of 10.7 m	~~10~~	~~SS~~		
		11	SS	80	0 25 48 27
		12	SS	104	

ELEVATION SCALE: 229, 228, 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

Continued Next Page

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

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RECORD OF BOREHOLE No F8-6

SHEET 2 OF 2

METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00

LOCATION N 4877028.4 ;E 297140.7

ORIGINATED BY AM

DIST Central HWY 400

BOREHOLE TYPE D-90 Truck Mount, 108 mm Inside Diameter Hollow Stem Auger

COMPILED BY CS

DATUM Geodetic

DATE March 31, 2011

CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ 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 ---										
	Sandy SILT to Silty SAND, trace to some clay, trace gravel (TILL) Compact to very dense Grey Moist		13	SS	104		214		○		
							213				
211.9 17.2			14	SS	103		212		○		
	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 14.9 m below ground surface (Elev. 214.2 m) upon completion of drilling.										

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

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RECORD OF BOREHOLE No OHS7

SHEET 1 OF 1 — METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00 — LOCATION N 4877043.3 ;E 297113.2 — ORIGINATED BY TT

DIST Central HWY 400 — BOREHOLE TYPE Geoprobe, 108 mm Outside Diameter Solid Stem Augers — COMPILED BY CS

DATUM Geodetic — DATE April 6, 2011 — CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
220.4	GROUND SURFACE					
0.0	Clayey silt, trace sand, trace gravel, containing rootlets and wood fragments (FILL) Soft Grey and black Moist	1	SS	2		
219.6 0.9	PEAT/TOPSOIL					
	Sandy SILT, trace clay Compact Grey Wet	2	SS	10	∇	
		3A / 3B	SS	10		
218.4 2.0	CLAYEY SILT, trace sand, containing zones of oxidation to a depth of 5.6 m Stiff Grey Moist, becoming wet below 5.6 m	4	SS	16		0 0 63 37
		5	SS	14		
		6	SS	17		
		7	SS	16		
		8	SS	20		
	becoming wet at a depth of 5.6 m					
		9	SS	12		0 2 63 35
213.7 6.7	END OF BOREHOLE					

Elevation scale: 220, 219, 218, 217, 216, 215, 214

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

61

NOTES:

1. Water level in open borehole at a depth of 1.1 m below ground surface (Elev. 219.3 m) upon completion of drilling.

2. Borehole caved at a depth of 2.4 m below ground surface (Elev. 218.0 m) upon completion of drilling.

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

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RECORD OF BOREHOLE No SC-1

SHEET 1 OF 2

METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4877070.0 ;E 297189.1

BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing

DATE June 8 and 11, 2012

ORIGINATED BY OS

COMPILED BY NK

CHECKED BY LCC

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
223.0	GROUND SURFACE
0.0	TOPSOIL
0.2	Silty sand, some gravel, trace clay, containing organic matter and rootlets (FILL) Loose to compact Brown with oxidation staining Moist
221.5 1.5	CLAYEY SILT with sand, trace gravel, containing organic matter Firm Grey and black Moist
220.7	PEAT, containing silt Loose Dark brown to black Moist
220.3 2.8	SILTY SAND, trace clay, trace gravel, containing wood fragments and organic matter Loose Grey and black Moist
	CLAYEY SILT with sand, trace to some gravel (TILL) Firm to hard Grey Moist
	Cobbles inferred from split spoon bouncing at 7.6 m and 8.4 m
210.2 12.8	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	12	
2	SS	4	
3	SS	6	
4	SS	8	
5	SS	6	
6	SS	21	
7	SS	18	7 25 47 21
8	SS	27	
9	SS	56	
10	SS	66	
11	SS	55	
12	SS	79	
13	SS	43	1 27 49 23

GROUND WATER CONDITIONS

ELEVATION SCALE: 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

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

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RECORD OF BOREHOLE No SC-1

SHEET 2 OF 2

METRIC

PROJECT 09-1111-0018			
G.W.P. 2835-02-00	LOCATION N 4877070.0 ;E 297189.1	ORIGINATED BY OS
DIST Central HWY 400	BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing	COMPILED BY NK
DATUM Geodetic	DATE June 8 and 11, 2012	CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE NOTES: 1. Water level in open borehole measured at a depth of 2.8 m (Elev. 220.2 m) on completion of drilling. 2. Water level in piezometer measured at a depth of 0.3 m (Elev. 222.7 m) on June 12, 2012.										

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

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RECORD OF BOREHOLE No SC-2

SHEET 1 OF 2

METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00

LOCATION N 4877082.3 ;E 297188.1

ORIGINATED BY OS

DIST Central HWY 400

BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing

COMPILED BY NK

DATUM Geodetic

DATE June 6-8, 2012

CHECKED BY LCC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
222.0	GROUND SURFACE				
0.0	TOPSOIL				
0.1	CLAYEY SILT with sand, trace to some gravel, containing wet silty sand lenses Soft to firm Brown to grey below 0.7 m Moist	1	SS	7	
		2	SS	3	
220.3 1.9	PEAT, containing silt Loose Dark brown to black Moist	3	SS	2	
	Sandy SILT, some clay, containing wood fragments and organic matter Very loose to compact Grey Moist to wet	4	SS	15	0 25 60 15
218.9 3.1	CLAYEY SILT with sand, trace to some gravel (TILL) Firm to hard Grey Moist	5	SS	6	
		6	SS	19	9 25 46 20
		7	SS	34	
		8	SS	29	
		9	SS	25	
		10	SS	31	
		11	SS	36	
		12	SS	34	
		13	SS	71	
208.1 13.9	SAND and SILT, trace clay, trace gravel (TILL) Very dense Grey Wet	14	SS	66	

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

Continued Next Page

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

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RECORD OF BOREHOLE No SC-2

SHEET 2 OF 2 **METRIC**

PROJECT 09-1111-0018
G.W.P. 2835-02-00 LOCATION N 4877082.3 ;E 297188.1 ORIGINATED BY OS
DIST Central HWY 400 BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing COMPILED BY NK
DATUM Geodetic DATE June 6-8, 2012 CHECKED BY LCC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND and SILT, trace clay, trace gravel (TILL) Very dense Grey Wet		15	SS	55				○		2 61 30 7
							206				
			16	SS	67		205				
204.6 17.4	END OF BOREHOLE										

NOTES:

1. Artesian conditions observed at a depth of 13.7 m (Elev. 208.3 m) during drilling operations.

2. Water level measured inside casing at 2.0 m above ground surface (Elev. 224.0 m) on completion of drilling.

3. Borehole abandoned using cement grout.

GTA-MTO 001 T:\PROJECTS\2009\09-1111-0018 (URS, YORK REGION)\LOG\0911110018.GPJ GAL-GTA.GDT 01/13/15 SIB

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

RECORD OF BOREHOLE No SC-3

SHEET 1 OF 2 **METRIC**

PROJECT 09-1111-0018			
G.W.P. 2835-02-00	LOCATION N 4877124.8 ;E 297177.2	ORIGINATED BY OS	
DIST Central	HWY 400	BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing	COMPILED BY NK
DATUM Geodetic	DATE May 23-25, 2012	CHECKED BY LCC	

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
220.1	GROUND SURFACE
0.0	TOPSOIL
0.2	Clayey silt with sand to some sand, trace to some gravel, containing rootlets, wood fragments and organics (FILL) Firm to stiff Brown to grey Moist to wet
218.8 1.3	Silty sand, containing wood fragments and organics (FILL) Compact Grey to black Moist
217.4 2.7	CLAYEY SILT, trace to some sand, trace gravel Soft to stiff Grey Moist to wet
210.4 9.7	Silty SAND Grey Moist
209.7 10.4	CLAYEY SILT with sand, some gravel (TILL) Hard Grey Moist
208.4 11.7	SAND and SILT, trace clay Very dense Grey Moist

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7		
2A / 2B	SS	10		
3	SS	23		
4	SS	14		
5	SS	5		
6	SS	10		
S1	TO	PH	SG = 2.77 C	
7	SS	10		
8	SS	3		
9	SS	20		
10	SS	96		15 39 31 15
11	SS	74		0 50 43 7
12	SS	100/0.08		

GROUND WATER CONDITIONS

ELEVATION SCALE: 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208, 207, 206

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values plotted: 2, >96, 2, 2, >96

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

WATER CONTENT (%): 10 20 30

Continued Next Page

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

RECORD OF BOREHOLE No SC-3

SHEET 2 OF 2

METRIC

PROJECT 09-1111-0018
G.W.P. 2835-02-00
LOCATION N 4877124.8 ;E 297177.2
ORIGINATED BY OS
DIST Central HWY 400
BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing
COMPILED BY NK
DATUM Geodetic
DATE May 23-25, 2012
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	SAND and SILT, trace clay Very dense Grey Moist	13	SS	172	205	~29	0	52	39	9
203.8 16.3	Silty SAND, containing silt seams Very dense Grey Moist	14	SS	212/0.25	204					
202.9 17.2	END OF BOREHOLE				203					

NOTES:

1. Artesian groundwater conditions were encountered within the cohesionless soil below a depth of 11.7 m (Elev. 208.4 m).

2. Artesian groundwater level was measured at 3.6 m above ground surface (Elev. 223.7 m) on May 25, 2012, after completion of drilling.

3. Borehole abandoned using cement grout.

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

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

Golder Associates

RECORD OF BOREHOLE No SC-4

SHEET 1 OF 2 **METRIC**

PROJECT 09-1111-0018
G.W.P. 2835-02-00
LOCATION N 4877151.8 ;E 297171.4
ORIGINATED BY TT
DIST Central HWY 400
BOREHOLE TYPE 108 mm Inside Diameter Hollow Stem Augers
COMPILED BY NK
DATUM Geodetic
DATE November 17, 18 and 21, 2011
CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa (Field vane; numbers = sensitivity)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
220.8	GROUND SURFACE					
0.0	Sand and silt to sandy silt, trace clay, trace gravel, containing organics (FILL) Loose to compact Brown, grey and black Moist	1	SS	27		
		2	SS	6		
219.3 1.5	Clayey silt, trace sand, trace gravel, containing organics and silty sand zones (FILL) Firm Grey Moist	3	SS	4		
218.7	Peat (Fibrous becoming amorphous at 2.2 m) Black Moist					
218.4 2.4	SILT, some sand, some clay, containing rootlets, organics and zones of clayey silt Loose to compact Grey Moist	4	SS	9		
		5	SS	10		0 19 69 12
217.1 3.7	CLAYEY SILT, some sand, trace gravel, containing silty sand layers to a depth of 7.2 m Firm to very stiff Grey Moist	6	SS	2		
					>115	
		7	SS	7		
					>115	
		8	SS	6		4 26 54 16
					3, 3	
		9	SS	17		
		10	SS	6		
					4, 3	
		11	SS	1		
					3, 2	
		12	SS	7		8 19 53 20
					4	
207.4 13.4	Sandy SILT, trace clay, trace gravel, containing gravelly sand layers Very dense Grey Wet	13	SS	56		
206.0 14.8						

Continued Next Page

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

Elevation scale: 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208, 207, 206

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

Foundation Design

RECORD OF BOREHOLE No SC-4

SHEET 2 OF 2 — METRIC

PROJECT 09-1111-0018
G.W.P. 2835-02-00
LOCATION N 4877151.8 ;E 297171.4
ORIGINATED BY TT
DIST Central HWY 400
BOREHOLE TYPE 108 mm Inside Diameter Hollow Stem Augers
COMPILED BY NK
DATUM Geodetic
DATE November 17, 18 and 21, 2011
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	SAND and SILT, trace to some clay, trace gravel (TILL) Very dense Grey Moist	14	SS	107				
		15	SS	92	2	32	56	10
		16	SS	70				
200.6		17	SS	86				
20.2	Sandy SILT to SILT, trace sand, trace clay Compact to very dense Grey Moist	18	SS	11	0	28	70	2
		19	SS	25				
195.3		20	SS	73	0	1	89	10
25.5	CLAYEY SILT, trace sand, containing silty sand seams Hard Grey Moist	21	SS	84				
193.8								
27.0	SAND and GRAVEL, trace to some silt, trace clay Very dense Grey Wet	22	SS	112				
192.9								
27.9	END OF BOREHOLE							

NOTE:

1. Artesian conditions encountered at a depth of 22.9 m (Elev. 197.9 m).

2. Borehole abandoned using cement grout.

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

RECORD OF BOREHOLE No SC-5

SHEET 1 OF 2

METRIC

PROJECT 09-1111-0018			
G.W.P. 2835-02-00	LOCATION N 4877176.1 ;E 297165.0		ORIGINATED BY TT
DIST Central	HWY 400	BOREHOLE TYPE 108 mm Inside Diameter Hollow Stem Augers	COMPILED BY NK
DATUM Geodetic		DATE November 15, 2011	CHECKED BY LCC

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
221.1	GROUND SURFACE
0.0	ASPHALT
	Sand and silt, trace gravel (FILL) Dense Brown Moist
220.4 / 0.7	Clayey silt with sand, trace gravel (FILL) Hard Grey Moist
	PEAT, some silt Soft Black Moist
218.9 / 2.2	CLAYEY SILT, some sand Very soft to soft Grey Moist
218.3	Sandy SILT, trace clay Grey Wet
217.9 / 3.2	CLAYEY SILT, trace to some sand, trace gravel, containing sand seams and interlayers at a depth of 13.7 m Firm to very stiff Grey Moist
206.3 / 14.8	

SAMPLES

NUMBER	TYPE	"N" VALUES	ELEVATION (approx.)	WATER CONTENT / REMARKS
1	SS	34	221	
2	SS	3	220	
3	SS	2	219	282.5
4	SS	1		
5	SS	2	218	
6	SS	9	216	GR 1, SA 7, SI 66, CL 26
7	SS	15	215	
8	SS	13	213	
9	SS	7	212	
10	SS	1	210	
T1	TO	PH	209	Unit weight 20.1 Oed
11	SS	13	207	GR 4, SA 29, SI 39, CL 28

Ground water level: approx. elev. 217.0

SHEAR STRENGTH kPa (FIELD VANE) — sensitivity values

Approx. elevation	Shear strength plot / sensitivity
217.0	2
216.7	2
215.5	6
215.2	2
214.0	> 163
212.4	2
212.1	3
210.9	3
210.6	3
209.4	2
209.1	3
208.0	3
207.7	2
206.3	>120

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

Continued Next Page

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

RECORD OF BOREHOLE No SC-5

SHEET 2 OF 2 — METRIC

PROJECT 09-1111-0018			
G.W.P. 2835-02-00	LOCATION N 4877176.1 ;E 297165.0	ORIGINATED BY TT	
DIST Central HWY 400	BOREHOLE TYPE 108 mm Inside Diameter Hollow Stem Augers	COMPILED BY NK	
DATUM Geodetic	DATE November 15, 2011	CHECKED BY LCC	

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ 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		12	SS	66		206				
205.3 15.9	END OF BOREHOLE										

NOTES:

1. Blowing sands encountered at a depth of 15.2 m (Elev. 205.9 m)

2. Water level in open borehole at a depth of 4.2 m (Elev. 216.9 m) on completion of drilling.

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No SC-7

SHEET 1 OF 3 **METRIC**

PROJECT 09-1111-0018

G.W.P. 2835-02-00 LOCATION N 4877117.8 ;E 297113.1 ORIGINATED BY SB/TT

DIST Central HWY 400 BOREHOLE TYPE 108 mm Inside Diameter Hollow Stem Augers COMPILED BY NK

DATUM Geodetic DATE November 7, 2011 CHECKED BY LCC

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
220.7	GROUND SURFACE
0.0	Clayey silt, trace sand, trace gravel, containing roots and organic matter (FILL) Very stiff Brown Moist
219.9 0.8	PEAT (Fibrous), trace clay Very soft to stiff Black and brown Moist
218.4 2.3	CLAYEY SILT, trace sand and peat Soft Grey Moist
217.6 3.1	Silty SAND, trace clay Loose Grey Moist
216.9 3.8	CLAYEY SILT to SILT, trace to some clay, trace to some sand, trace gravel Firm to very stiff Grey Moist
207.0 13.7	CLAYEY SILT, trace sand, trace gravel (TILL) Hard Grey Moist

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS	25					
2	SS	9					
3	SS	3					
4	SS	3					
5	SS	4					
6	SS	5		0	6	80	14
7	TO	PH					
8	SS	17					
9	SS	21					
10	SS	8					
11	TO	PH	20.6 Oed				
12	SS	WH		2	20	57	21
13	SS	32					

ELEVATION SCALE: 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208, 207, 206

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values: 3, 2, 3, 2, 2, 2

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

132.3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GROUND WATER CONDITIONS

Continued Next Page

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

Foundation Design

Golder Associates

RECORD OF BOREHOLE No SC-7 SHEET 2 OF 3 **METRIC**

PROJECT 09-1111-0018

G.W.P. 2835-02-00 | LOCATION N 4877117.8 ;E 297113.1 | ORIGINATED BY SB/TT

DIST Central HWY 400 | BOREHOLE TYPE 108 mm Inside Diameter Hollow Stem Augers | COMPILED BY NK

DATUM Geodetic | DATE November 7, 2011 | CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
205.5 / 15.2	SAND and SILT, trace to some gravel, trace clay; Very dense; Grey; Moist to wet; Blowing sand encountered at a depth of 15.8 m	14	SS	146	1	38	52	9
		15	SS	93				
		16	SS	63				
201.4 / 19.3	SAND and GRAVEL, trace silt and clay; Very dense; Grey; Wet	17	SS	69	33	59	6	2
199.9 / 20.8	SAND and SILT to SAND, trace to some silt, trace gravel and clay; Very dense; Grey; Wet	18	SS	106				
		19	SS	76				
		20	SS	64				
		21	SS	72	0	85	10	5
		22	SS	61				
		23	SS	71				

Elevation scale: 205, 204, 203, 202, 201, 200, 199, 198, 197, 196, 195, 194, 193, 192, 191

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 (%)

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No SC-7

SHEET 3 OF 3

METRIC

PROJECT 09-1111-0018			
G.W.P. 2835-02-00	LOCATION N 4877117.8 ;E 297113.1	ORIGINATED BY SB/TT	
DIST Central HWY 400	BOREHOLE TYPE 108 mm Inside Diameter Hollow Stem Augers	COMPILED BY NK	
DATUM Geodetic	DATE November 7, 2011	CHECKED BY LCC	

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

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	SAND and SILT to SAND, trace to some silt, trace gravel and clay Very dense Grey Wet	24	SS	61	190				
					189				
188.4 32.3	CLAYEY SILT, trace sand, trace gravel, containing silt seams Hard Grey Moist				188				
		25	SS	52	187	1	1	65	33
					186				
					185				
		26	SS	67	184				
					183				
182.3 38.4	SAND and GRAVEL, some silt, trace clay Very dense Grey Wet				182				
		27	SS	87	181	47	39	13	1
180.6 40.1	END OF BOREHOLE								

NOTES:

1. Blowing sands and artesian conditions encountered below a depth of 15.7 m (Elev. 205.0m).

2. Tricone and wash boring used below a depth of 15.2 m (Elev. 205.5 m) due to artesian conditions in the sand layer.

3. Artesian pressure up to 1.5 m above ground surface (Elev. 222.2 m) noted during removal of hollow stem augers.

4. Borehole caved at a depth of 36.6 m (Elev. 184.1 m) on completion of drilling.

5. Borehole abandoned using cement grout, with 3 m of bentonite placed above the grout immediately below ground surface.

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

Golder Associates

RECORD OF BOREHOLE No SC-8

SHEET 1 OF 1 **METRIC**

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

LOCATION N 4877130.1 ;E 297103.5
BOREHOLE TYPE 108 mm Inside Diameter Hollow Stem Augers
DATE November 7, 2011

ORIGINATED BY SB
COMPILED BY NK
CHECKED BY LCC

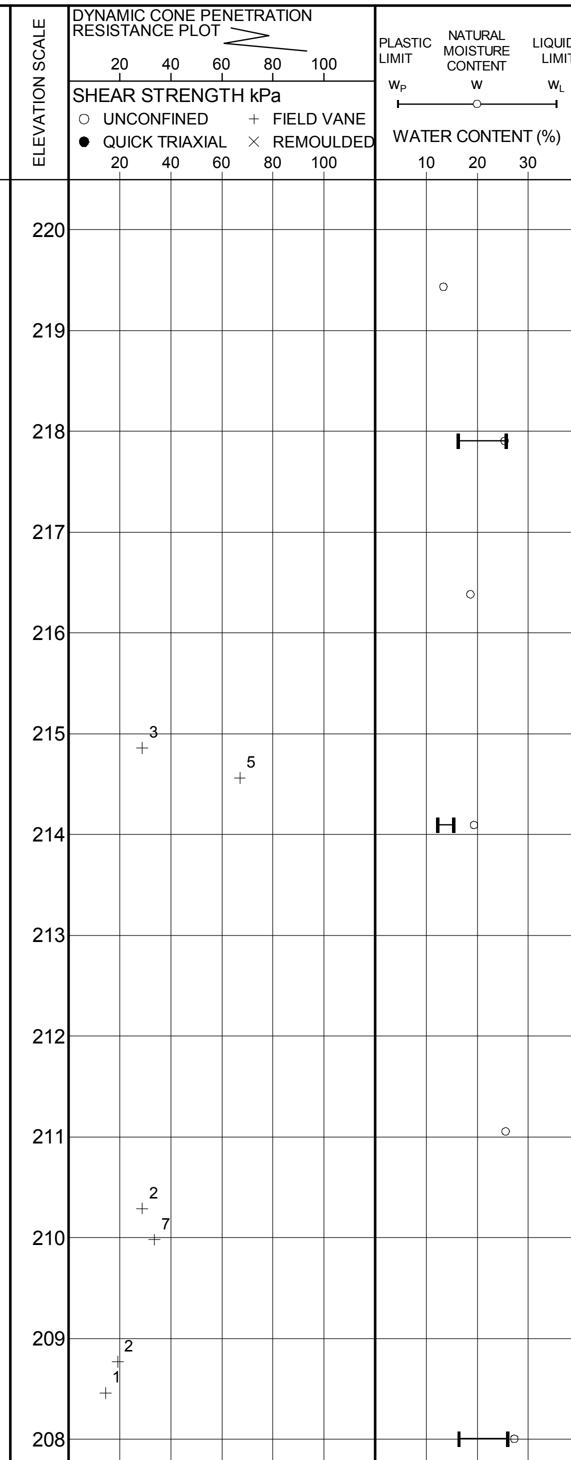
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
220.5	GROUND SURFACE
0.0	TOPSOIL
0.2	CLAYEY SILT, trace sand, trace gravel, containing organic matter Stiff Brown and grey Moist
218.7 1.8	PEAT (Fibrous) Firm Black Moist
218.2 2.3	CLAYEY SILT, trace sand Very soft Grey Moist
217.0	Silty SAND Very loose Grey Moist
3.7	CLAYEY SILT to SILT, some clay, trace to some sand Soft to very stiff Grey Moist
207.7 12.8	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 5.5 m (Elev. 215.0 m) on completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	13	
2	SS	11	
3	SS	5	
4	SS	2	0 1 85 14
5	SS	WH	
6	SS	7	
7	SS	6	
8	SS	24	0 15 71 14
9	SS	18	
10	SS	WH	
11	SS	WH	
12	SS	WH	0 2 68 30

GROUND WATER CONDITIONS



UNIT WEIGHT γ kN/m^3

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

Golder Associates

RECORD OF BOREHOLE No SC-9

SHEET 1 OF 2

METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00

LOCATION N 4877070.2 ;E 297116.5

ORIGINATED BY OS

DIST Central HWY 400

BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing

COMPILED BY NK

DATUM Geodetic

DATE May 15 and 16, 2012

CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
221.0	GROUND SURFACE							
0.0	TOPSOIL							
0.2	Silty SAND, some gravel, trace clay, containing rootlets and organic matter Compact Brown Moist	1	SS	12				
220.4 / 0.6	CLAYEY SILT, some sand, some gravel, contains rootlets and organic matter, containing peat at a depth of 1.0 m Soft to stiff Grey Moist	2	SS	11	220			
		3	SS	2	219			
218.9 / 2.1	SILT, some sand, trace to some clay Compact Grey Wet	4	SS	25				0 14 74 12
		5	SS	17	218			
217.3 / 3.7	CLAYEY SILT, trace sand, trace gravel Stiff to very stiff Grey Moist to wet	6	SS	11	217			
		7	SS	12	216			
						>96 (field vane)		
		8	SS	16	215			0 0 57 43
					214			
		9	SS	23	213			
					212			
		10	SS	15				
210.8 / 10.2	CLAYEY SILT with to some sand, trace gravel (TILL) Hard Grey Moist				211			
		11	SS	68	210			
					209			
		12	SS	78	208			
		13	SS	68	207			2 25 48 25

Continued Next Page

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

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RECORD OF BOREHOLE No SC-9

SHEET 2 OF 2 **METRIC**

PROJECT 09-1111-0018
G.W.P. 2835-02-00
LOCATION N 4877070.2 ;E 297116.5
ORIGINATED BY OS
DIST Central HWY 400
BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing
COMPILED BY NK
DATUM Geodetic
DATE May 15 and 16, 2012
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	CLAYEY SILT with to some sand, trace gravel (TILL) Hard Grey Moist	14	SS	104	205					
		15	SS	96	204	~10				
202.7 18.5	Silty SAND Grey Wet	16	SS	99	203					
201.6 19.4	CLAYEY SILT with sand, some gravel (TILL) Hard Grey Wet				202					
200.6 20.4	SAND, some silt, trace gravel, trace clay Very dense Grey Wet	17	SS	113	201	~14	5	77	14	4

END OF BOREHOLE

NOTES:

1. Artesian groundwater conditions were encountered within the cohesionless soil below a depth of 18.3 m (Elev. 202.7 m) during drilling operations

2. Artesian groundwater level was measured at 4.1 m above ground surface (Elev. 225.1 m) on May 16, 2012.

3. Borehole abandoned using cement grout.

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

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

Golder Associates

RECORD OF BOREHOLE No SC-10

SHEET 1 OF 2 — METRIC

PROJECT 09-1111-0018			
G.W.P. 2835-02-00	LOCATION N 4877033.5 ;E 297122.5	ORIGINATED BY OS	
DIST Central HWY 400	BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing	COMPILED BY NK	
DATUM Geodetic	DATE May 14,2012	CHECKED BY LCC	

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
222.1	GROUND SURFACE
0.0	TOPSOIL
0.1	CLAYEY SILT with to some sand, some gravel, containing rootlets and organic matter/wood fragments Firm to very stiff Brown to grey Moist
219.9 2.2	PEAT (Fibrous) Stiff Black Moist
219.4 2.7	Gravelly SAND and SILT, trace clay, containing clayey silt seams Compact to dense Grey Moist
218.4 3.7	CLAYEY SILT, trace to some gravel, trace to some sand Stiff to very stiff Grey Moist
214.9 7.2	CLAYEY SILT with to some sand, trace gravel (TILL) Hard Grey Moist

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7	
2	SS	14	
3	SS	27	
4	SS	11	
5	SS	35	25 40 30 5
6	SS	26	
7	SS	19	0 2 64 34
8	SS	14	
9	SS	40	
10	SS	52	
11	SS	74	1 25 47 27
12	SS	110	
13	SS	105	

GROUND WATER CONDITIONS: ∇ (at approx. elev. 220.1)

ELEVATION SCALE: 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208

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

Golder Associates

RECORD OF BOREHOLE No SC-10

SHEET 2 OF 2 — METRIC

PROJECT 09-1111-0018			
G.W.P. 2835-02-00	LOCATION N 4877033.5 ;E 297122.5		ORIGINATED BY OS
DIST Central	HWY 400	BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing	COMPILED BY NK
DATUM Geodetic		DATE May 14,2012	CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ 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 ---										
206.2	CLAYEY SILT with to some sand, trace gravel (TILL) Hard Grey Moist		14	SS	121		207				
15.9	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 2.1 m (Elev. 220.0 m) on completion of drilling.										

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

Golder Associates

RECORD OF BOREHOLE No SC-11

SHEET 1 OF 2

METRIC

PROJECT	09-1111-0018		
G.W.P.	2835-02-00	LOCATION	N 4877019.1 ;E 297122.9
DIST	Central	HWY	400
BOREHOLE TYPE	D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing		
DATUM	Geodetic	DATE	May 11, 2012
ORIGINATED BY	OS	COMPILED BY	NK
CHECKED BY	LCC		

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
221.8	GROUND SURFACE
0.0	TOPSOIL
0.1	Sand and gravel, trace clay, some silt, containing rootlets and organic matter (FILL) Loose to compact Dark brown to brown Moist, becoming wet at a depth of 0.3 m
220.4 / 1.4	Clayey silt with sand (FILL) Firm Brown Moist
219.7	PEAT
219.2 / 2.6	SAND and GRAVEL, containing wood fragments Dense Grey Moist
218.1 / 3.7	CLAYEY SILT, trace sand, trace gravel Very stiff Grey Moist
	CLAYEY SILT some to with sand, trace to some gravel (TILL) Very stiff to hard Grey Moist
207.5 / 14.3	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	9	
2	SS	15	49 33 13 5
3	SS	7	
4	SS	44	
5	SS	20	
6	SS	46	
7	SS	33	
8	SS	29	19 21 43 17
9	SS	29	
10	SS	29	
11	SS	43	
12	SS	80	
13	SS	106	0 28 47 25

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (221 to 208); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

Continued Next Page

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

Golder Associates

Foundation Design

PROJECT 09-1111-0018	RECORD OF BOREHOLE No SC-11	SHEET 2 OF 2	METRIC
G.W.P. 2835-02-00	LOCATION N 4877019.1 ;E 297122.9		ORIGINATED BY OS
DIST Central HWY 400	BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing		COMPILED BY NK
DATUM Geodetic	DATE May 11, 2012		CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE NOTE: 1. Borehole dry on completion of drilling.										

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

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RECORD OF BOREHOLE No SC-13

SHEET 1 OF 3 **METRIC**

PROJECT 09-1111-0018

G.W.P. 2835-02-00

DIST Central HWY 400

DATUM Geodetic

LOCATION N 4877052.4 ;E 297161.4

BOREHOLE TYPE D-90 Track Mount, 89 mm Outside Diameter Tricone Wash Boring, N Casing

DATE June 5 and 6, 2012

ORIGINATED BY TWB

COMPILED BY NK

CHECKED BY LCC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
229.0	GROUND SURFACE						
0.0	Sand and gravel, some silt, trace clay (FILL) Brown Moist				▽	228	
227.6 1.4	Clayey silt, some sand, some gravel (FILL) Stiff to hard Brown to grey Moist	1	SS	18		227	
		2	SS	10		226	
		3	SS	47		225, 224	
		4	SS	13		223	15 17 46 22
		5	SS	15		222	
		6	SS	15		221	
		7	SS	26		220	
		8	SS	18			
		9	SS	19		219	
218.3	Silty SAND, trace clay, trace gravel Dense Grey Wet	10	SS	38		218	
218.0 11.0	CLAYEY SILT, some sand Hard to stiff Grey, containing oxidized staining Moist	11	SS	9		217	0 12 58 30
215.7 13.3	CLAYEY SILT with to some sand, trace gravel (TILL) Stiff to hard Grey Moist Stiff below 14.78 m	12	SS	26		216, 215	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No SC-13

SHEET 2 OF 3 **METRIC**

PROJECT 09-1111-0018
G.W.P. 2835-02-00
LOCATION N 4877052.4 ;E 297161.4
ORIGINATED BY TWB
DIST Central HWY 400
BOREHOLE TYPE D-90 Track Mount, 89 mm Outside Diameter Tricone Wash Boring, N Casing
COMPILED BY NK
DATUM Geodetic
DATE June 5 and 6, 2012
CHECKED BY LCC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	CLAYEY SILT with to some sand, trace gravel (TILL) Stiff to hard Grey Moist	13	SS	14						
					213					
		14	SS	37	212	w ≈ 10, Wp–WL ≈ 11–20	2	25	50	23
					211					
		15	SS	45						
					210					
		16	SS	44	209	w ≈ 10				
					208					
		17	SS	50		w ≈ 10, Wp–WL ≈ 11–20				
					207					
					206					
					205					
204.1		18	SS	42						
24.9	SAND, trace to some silt, trace gravel, trace clay Very dense Grey Wet				204					
					203					
					202					
201.1		19	SS	92		w ≈ 11	4	84	9	3
27.9	CLAYEY SILT with to some sand to Sandy SILT, trace to some clay, trace gravel (TILL) Very dense Grey Moist				201					
					200					

Continued Next Page

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

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RECORD OF BOREHOLE No SC-13

SHEET 3 OF 3

METRIC

PROJECT 09-1111-0018
G.W.P. 2835-02-00
LOCATION N 4877052.4 ;E 297161.4
ORIGINATED BY TWB
DIST Central HWY 400
BOREHOLE TYPE D-90 Track Mount, 89 mm Outside Diameter Tricone Wash Boring, N Casing
COMPILED BY NK
DATUM Geodetic
DATE June 5 and 6, 2012
CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ 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 ---										
198.1			20	SS	133/0.23						16 24 48 12
30.9	END OF BOREHOLE NOTES: 1. Water level in open borehole (inside casing) at a depth of 1.0 m (Elev. 228.0 m) on June 6, 2012. 2. Borehole abandoned using cement grout.										

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

RECORD OF BOREHOLE No SC-14

SHEET 1 OF 2 **METRIC**

PROJECT 09-1111-0018
G.W.P. 2835-02-00 LOCATION N 4877041.9 ;E 297120.6 ORIGINATED BY OS
DIST Central HWY 400 BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing COMPILED BY NK
DATUM Geodetic DATE May 22, 2012 CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
222.0	GROUND SURFACE				
0.0	TOPSOIL				
0.2	Silty SAND, some gravel, trace clay, containing rootlets, and organic matter	1	SS	9	
221.2 / 0.8	Loose Brown to light brown Moist; CLAYEY SILT with sand, some gravel, containing organic matter Very stiff Grey Moist	2	SS	19	10 41 37 12
		3	SS	17	
219.5 / 2.7	PEAT, containing wood fragments Stiff Black Moist	4	SS	12	
218.5 / 3.5	SAND and SILT, trace clay, trace gravel Compact Brown Moist; CLAYEY SILT, trace sand Stiff to very stiff Light grey to grey, containing oxidized stains Moist to wet	5	SS	27	5 32 57 6
		6	SS	15	
		7	SS	19	
		8	SS	8	
215.3 / 6.7	CLAYEY SILT with to some sand, trace gravel (TILL) Hard Firm Moist				
		9	SS	53	
		10	SS	48	
		11	SS	50	
		12	SS	86	
		13	SS	63	1 26 47 26

Elevation scale: 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208

Water content annotation: 143.8

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

Continued Next Page

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

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RECORD OF BOREHOLE No SC-14

SHEET 2 OF 2

METRIC

PROJECT 09-1111-0018

G.W.P. 2835-02-00

LOCATION N 4877041.9 ;E 297120.6

ORIGINATED BY OS

DIST Central HWY 400

BOREHOLE TYPE D-25 Track Mount, 76 mm Wash Rotary Boring, NW Casing

COMPILED BY NK

DATUM Geodetic

DATE May 22, 2012

CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ 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 with to some sand, trace gravel (TILL) Hard Firm Moist		14	SS	65						
							206				
			15	SS	72		205				
203.9			16	SS	71		204				
18.1	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 3.7 m (Elev. 218.3 m) on completion of drilling.										

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

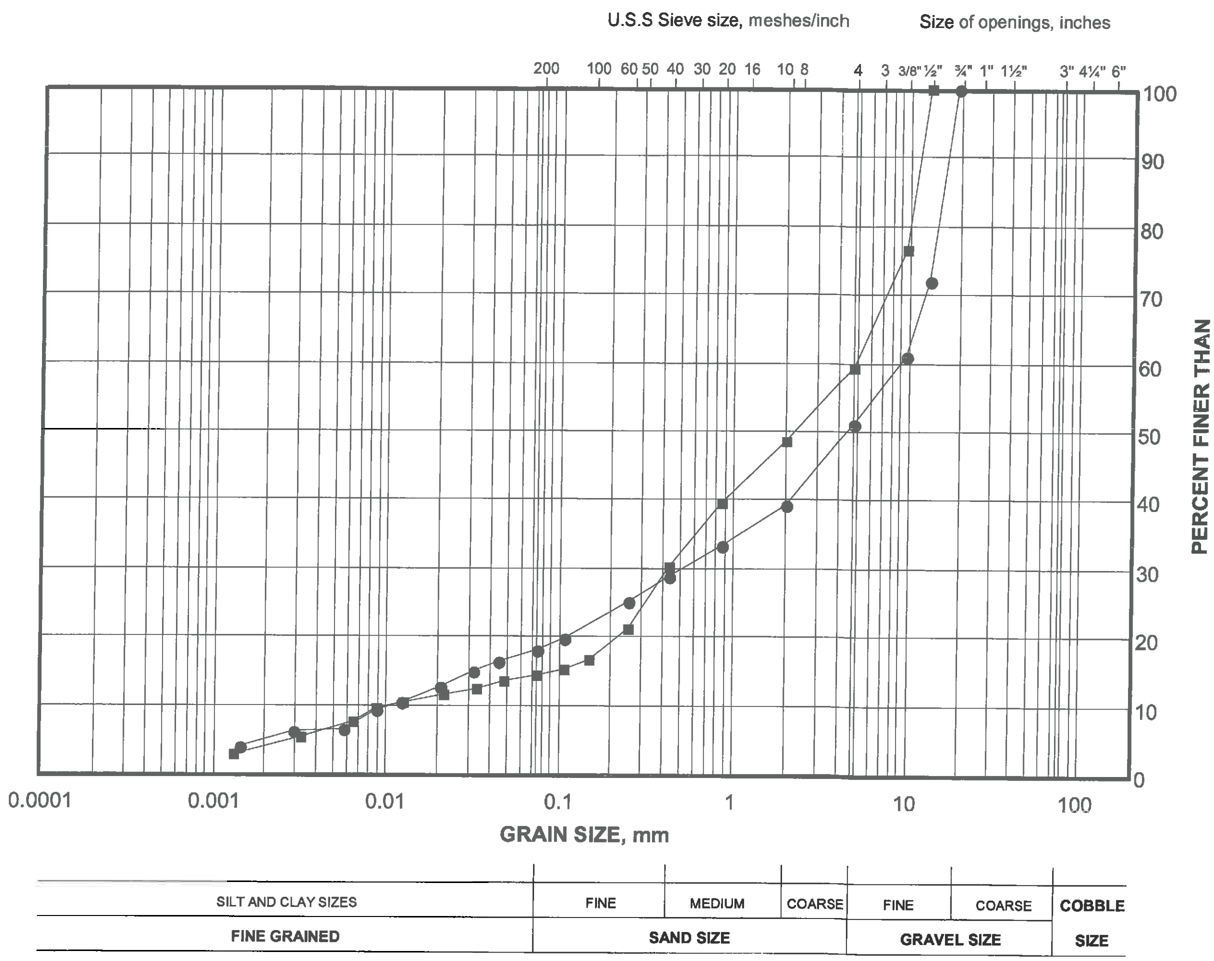
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel Fill

FIGURE B1A



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SC-11	2	220.7
■	F8-6	3	226.6

Project Number: 09-1111-0018

Checked By: [signature]

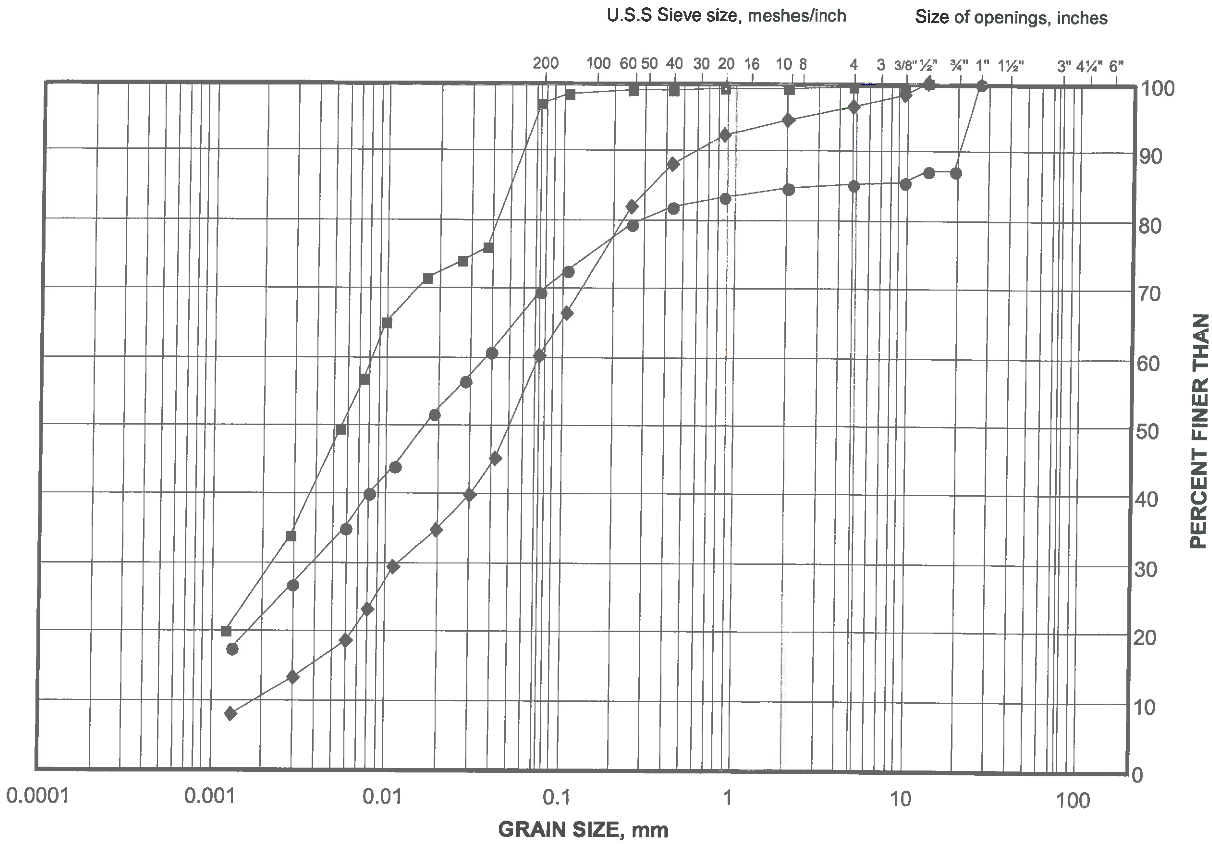
Golder Associates

Date: 15-Mar-13

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Fill

FIGURE B1B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SC-13	4	222.6
■	F8-3	4	218.4
◆	F8-2	5	225.1

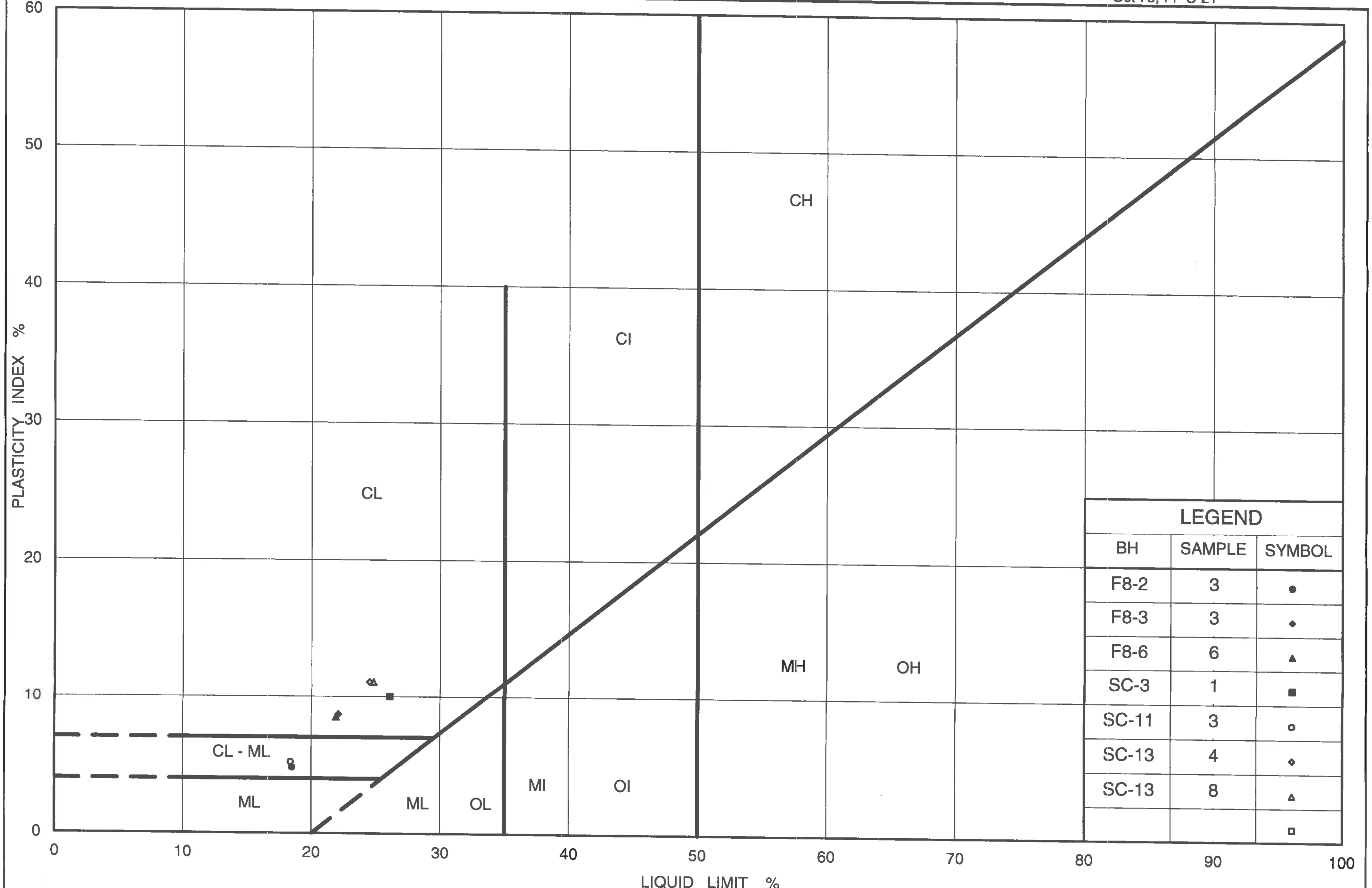
Project Number: 09-1111-0018

Checked By: [signature]

Golder Associates

Date: 15-Mar-13

Oct 75, FF-S-21



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PLASTICITY CHART
Clayey Silt Fill

Figure No. B2

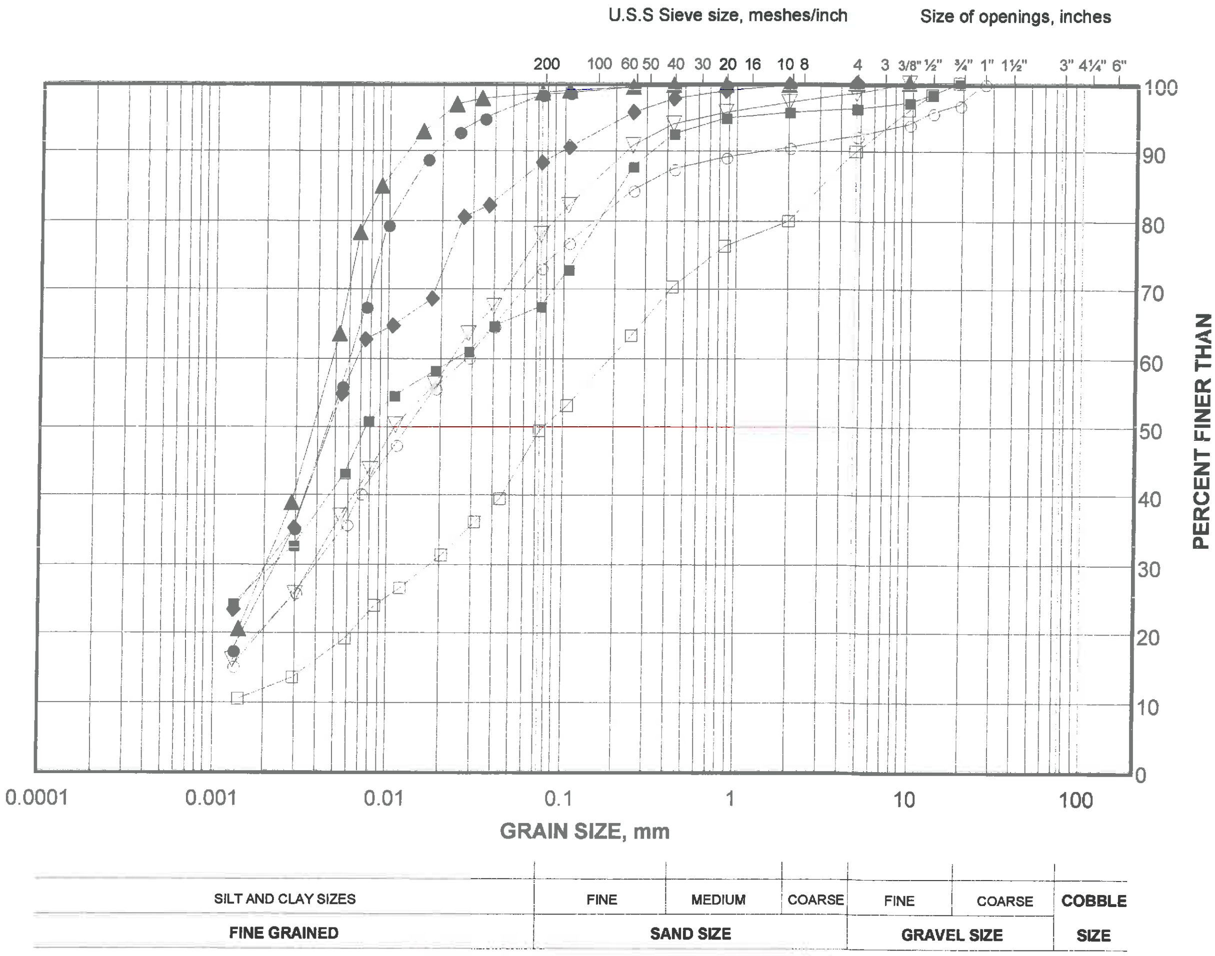
Project No. 09-1111-0018

Checked By:

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt (Upper Deposit)

FIGURE B3A



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BO-9	10	210.0
■	SC-5	11	207.1
◆	SC-13	11	216.5
▲	SC-8	12	208.0
▽	SC-7	12	208.2
○	SC-4	12	208.3
□	SC-14	2	220.9

Project Number: 09-1111-0018

Checked By: [signature]

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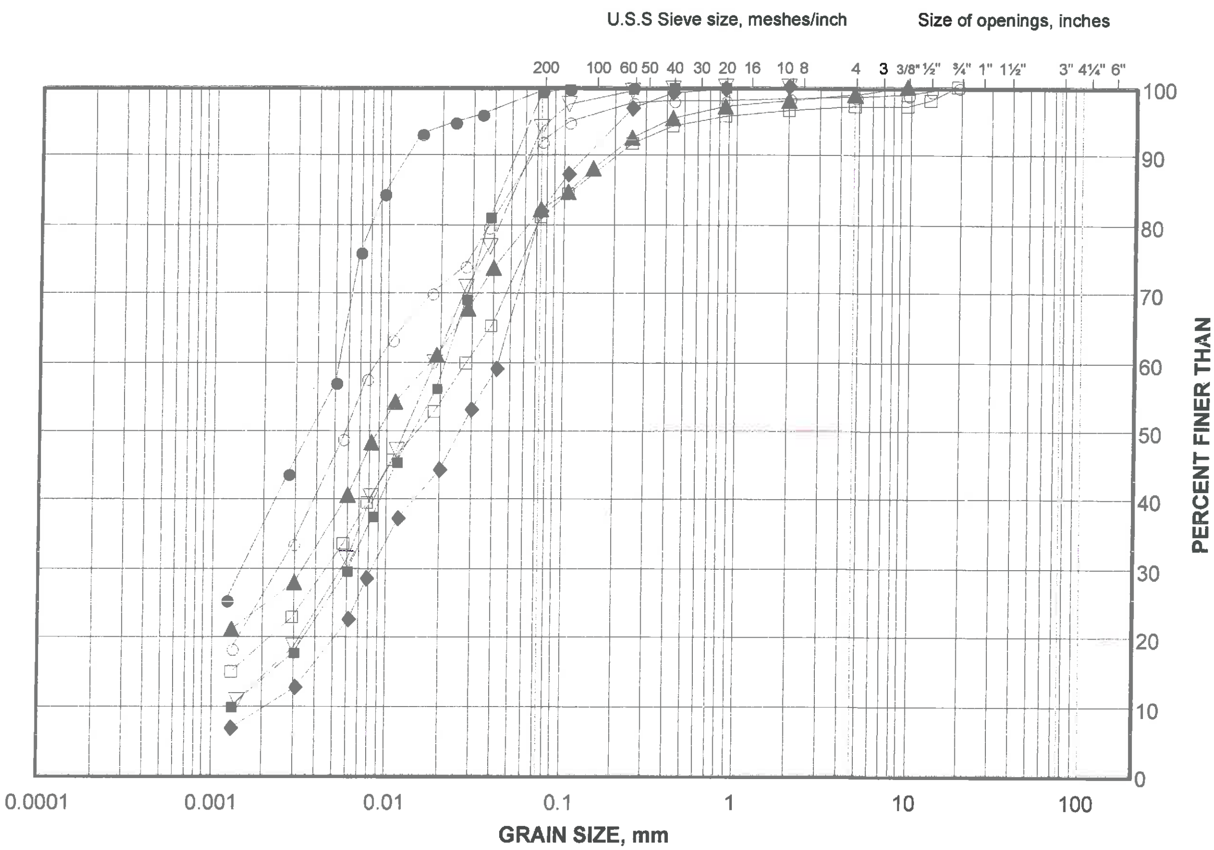
Date: 08-Mar-13

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt (Upper Deposit)

FIGURE B3B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	OHS-7	4	217.8
■	SC-8	4	217.9
◆	BO-9	5	216.9
▲	F8-6	6	224.2
▽	SC-7	6	216.6
○	SC-5	6	216.2
□	F8-2	7	222.8

Project Number: 09-1111-0018

Checked By: [signature]

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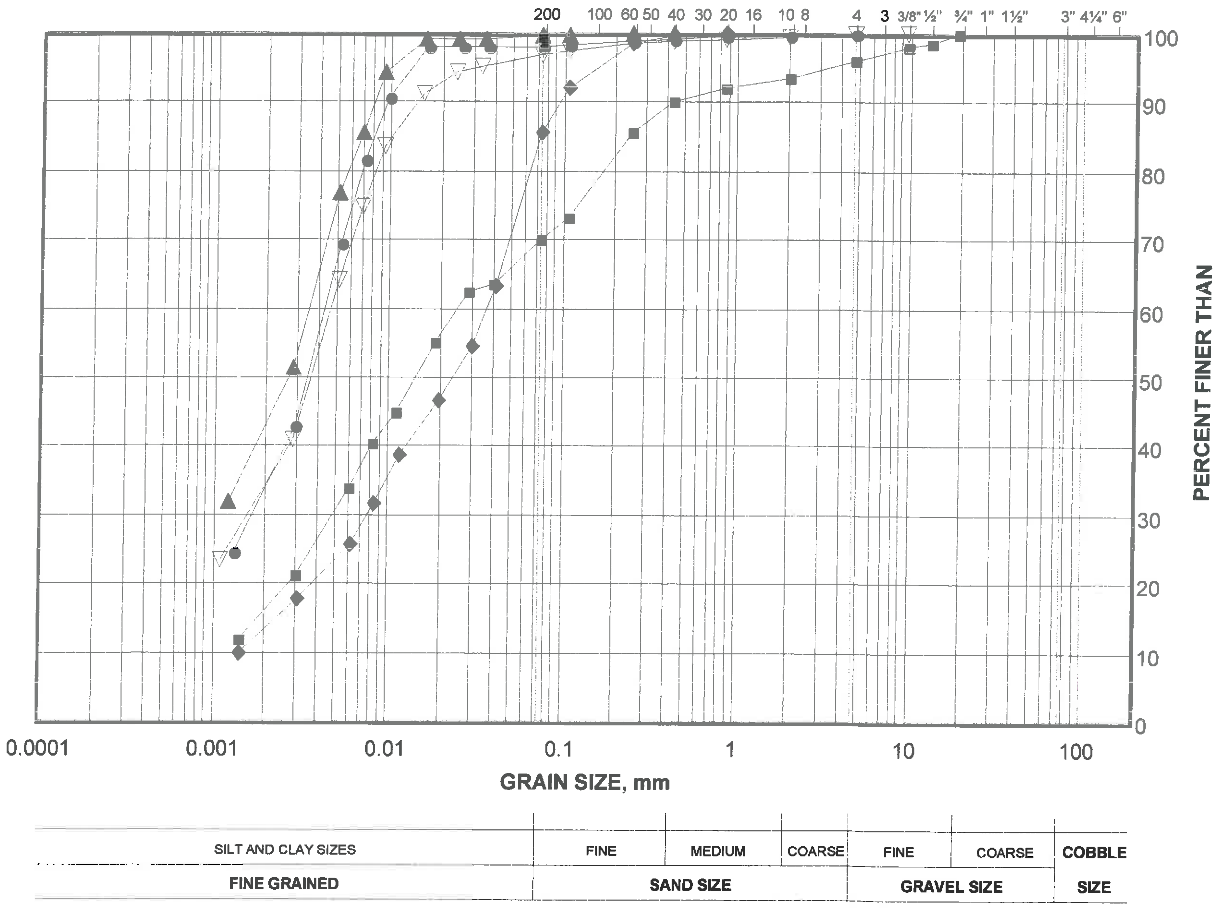
Date: 08-Mar-13

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt (Upper Deposit)

FIGURE B3C



LEGEND

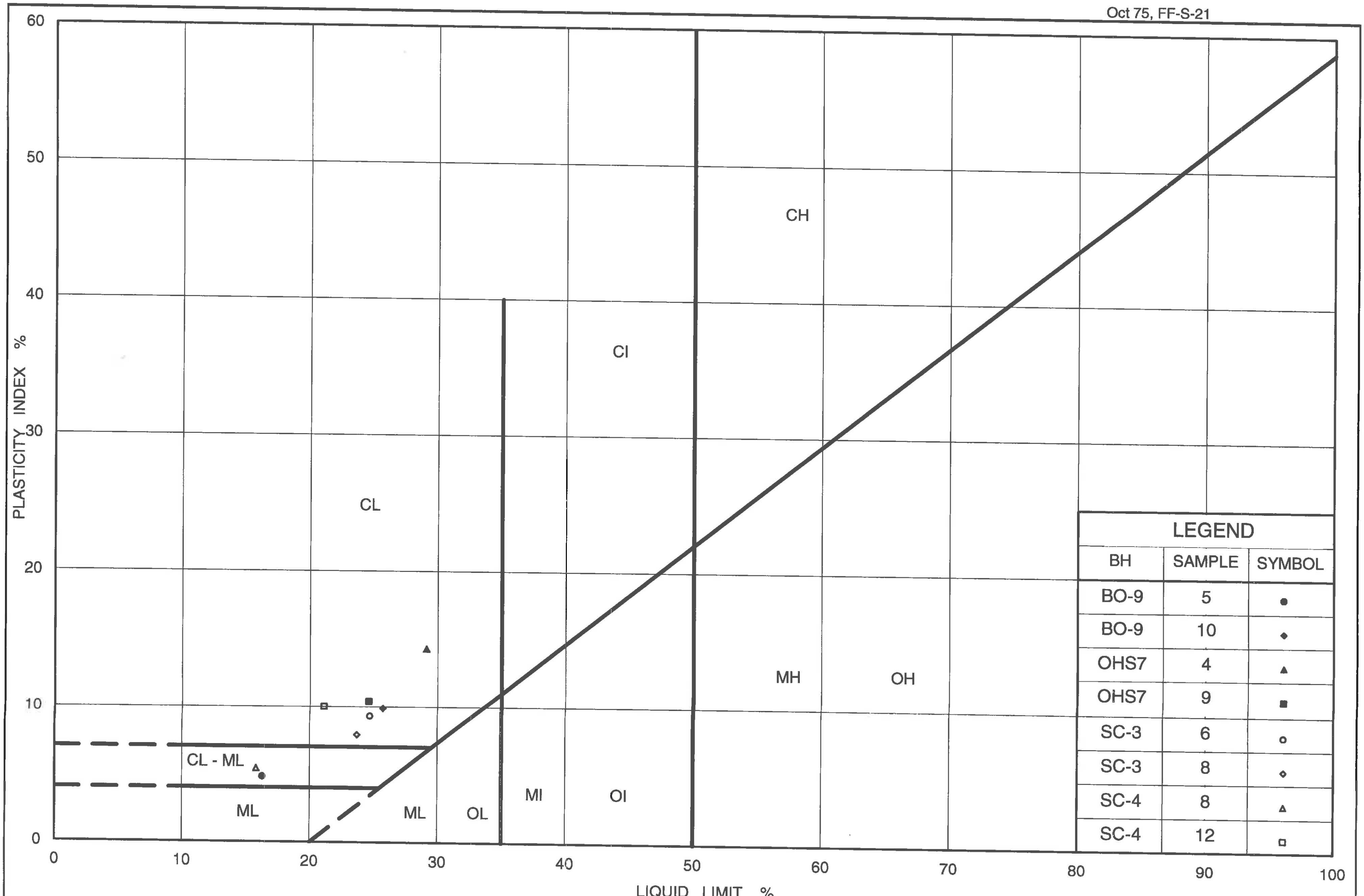
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SC-10	7	217.2
■	SC-4	8	214.4
◆	SC-8	8	214.1
▲	SC-9	8	214.6
▽	OHS-7	9	214.0

Project Number: 10-1111-0120

Checked By: Moza

Golder Associates

Date: 08-Mar-13



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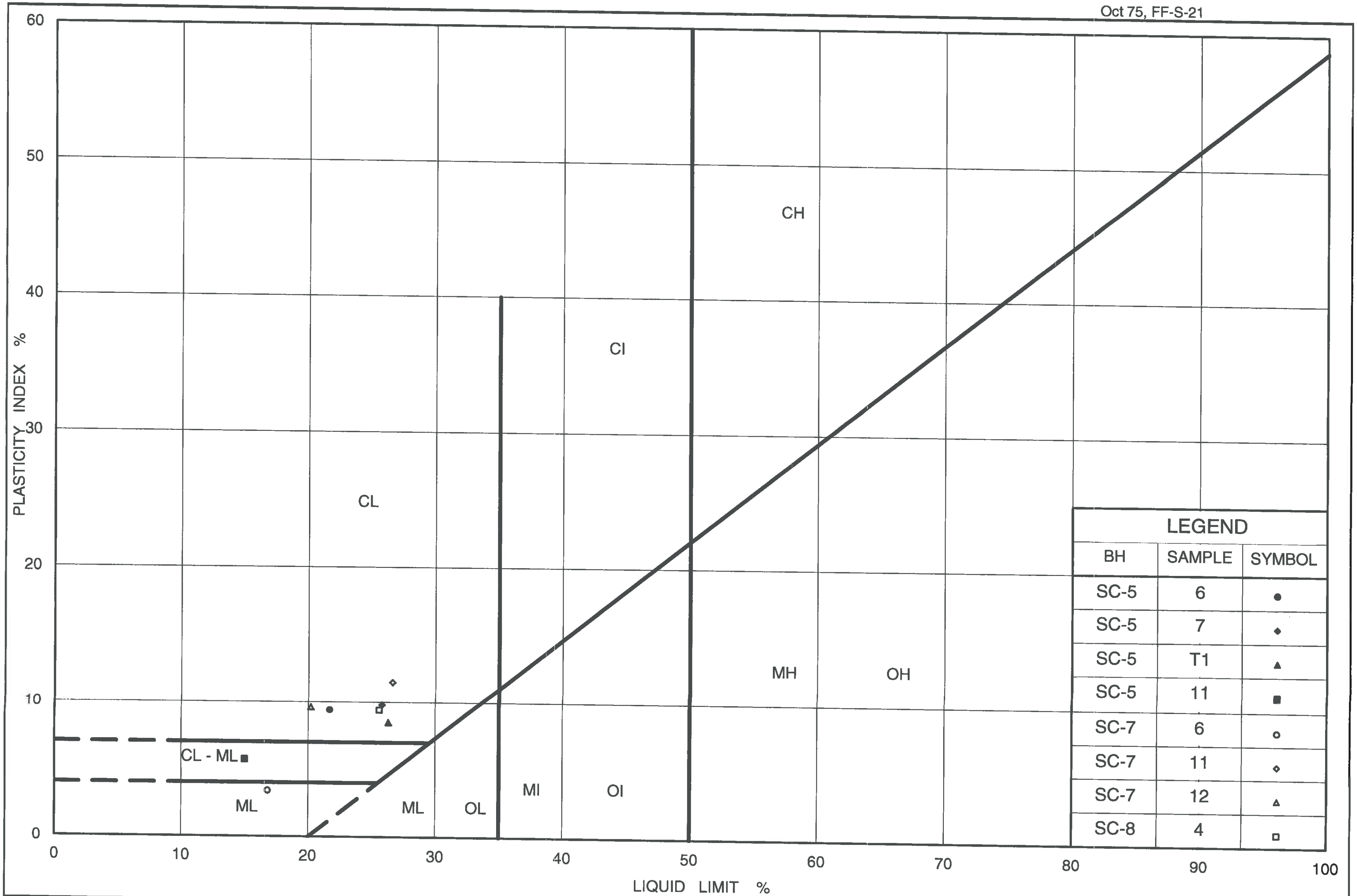
PLASTICITY CHART
Clayey Silt (Upper Deposit)

Figure No. B4A

Project No. 09-1111-0018

Checked By:



LEGEND		
BH	SAMPLE	SYMBOL
SC-5	6	●
SC-5	7	◆
SC-5	T1	▲
SC-5	11	■
SC-7	6	○
SC-7	11	◇
SC-7	12	△
SC-8	4	□

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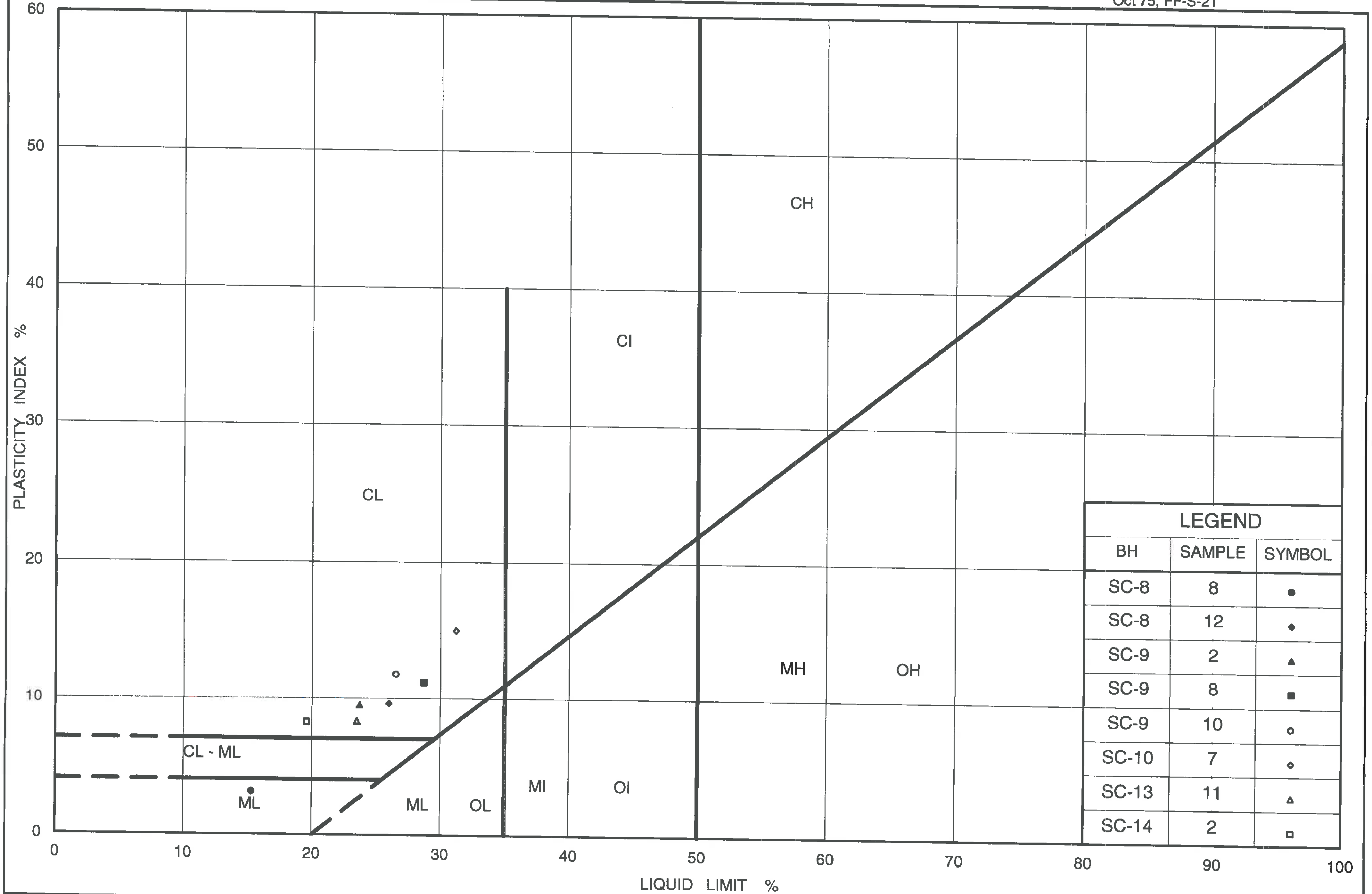
PLASTICITY CHART
Clayey Silt (Upper Deposit)

Figure No. B4B

Project No. 09-1111-0018

Checked By:

Oct 75, FF-S-21



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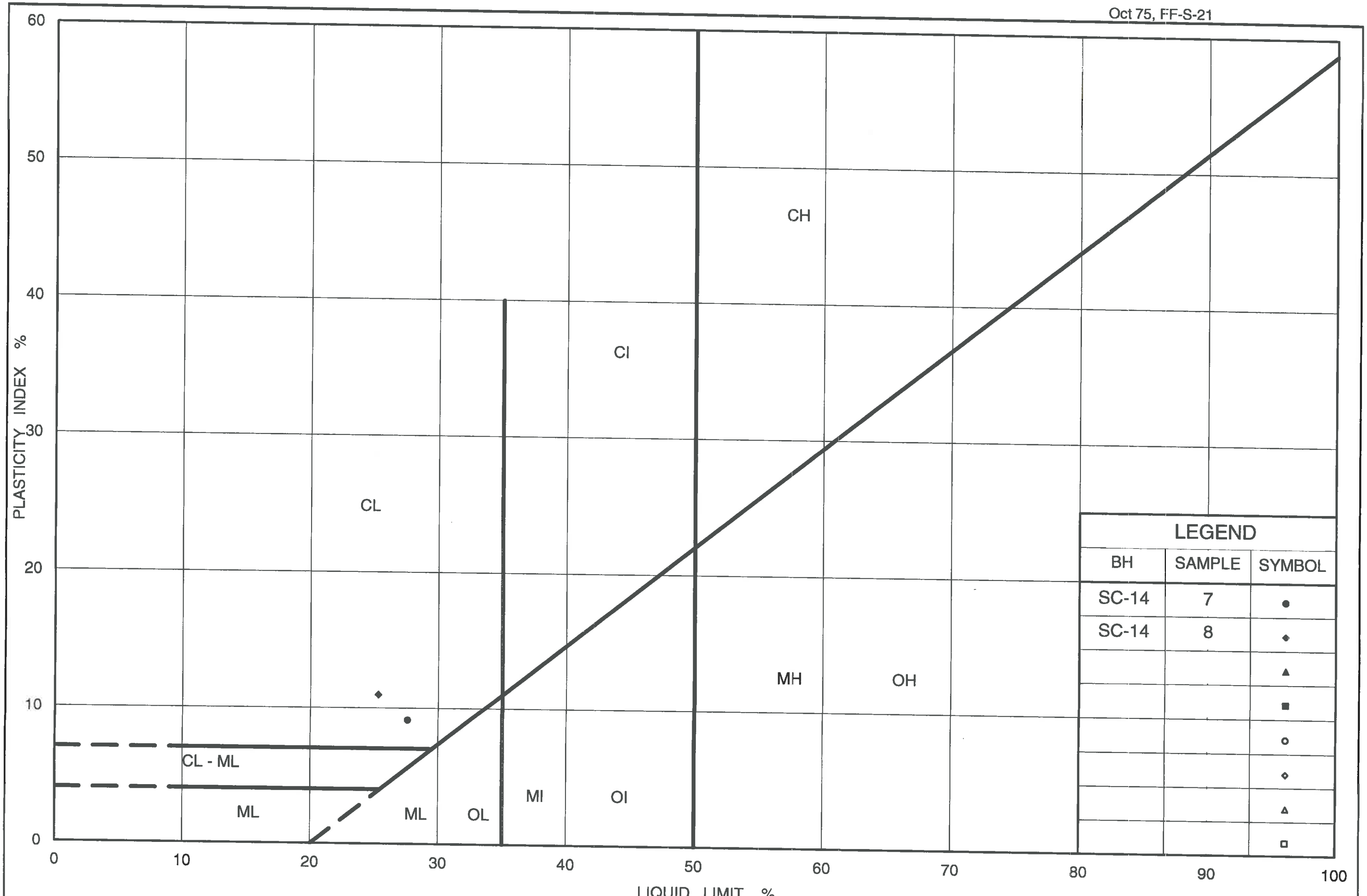
PLASTICITY CHART
Clayey Silt (Upper Deposit)

Figure No. B4C

Project No. 09-1111-0018

Checked By: [signature]



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PLASTICITY CHART
Clayey Silt (Upper Deposit)

Figure No. B4D

Project No. 09-1111-0018

Checked By:

CONSOLIDATION TEST RESULTS

Clayey Silt (Upper Deposit)

FIGURE B5A

SAMPLE IDENTIFICATION

Project Number	09-1111-0018	Sample Number	S1
Borehole Number	SC-3	Sample Depth, m	5.34-5.79

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	12		
Date Started	06/20/2012		
Date Completed	07/05/2012		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.55	Unit Weight, kN/m^3	20.74
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	16.56
Area, cm^2	31.58	Specific Gravity, measured	2.77
Volume, cm^3	80.46	Solids Height, cm	1.553
Water Content, %	25.28	Volume of Solids, cm^3	49.04
Wet Mass, g	170.18	Volume of Voids, cm^3	31.42
Dry Mass, g	135.84	Degree of Saturation, %	109.3

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.548	0.641	2.548				
5.97	2.524	0.625	2.536	1411	9.66E-04	1.60E-03	1.52E-07
10.77	2.516	0.620	2.520	1782	7.55E-04	6.46E-04	4.78E-08
20.51	2.502	0.611	2.509	1156	1.15E-03	5.40E-04	6.11E-08
39.99	2.486	0.601	2.494	454	2.90E-03	3.30E-04	9.41E-08
78.32	2.463	0.586	2.474	265	4.90E-03	2.38E-04	1.14E-07
156.28	2.436	0.569	2.449	252	5.05E-03	1.35E-04	6.67E-08
311.94	2.400	0.545	2.418	217	5.71E-03	9.18E-05	5.14E-08
622.06	2.356	0.517	2.378	228	5.26E-03	5.49E-05	2.83E-08
1241.34	2.306	0.485	2.331	240	4.80E-03	3.18E-05	1.50E-08
2481.97	2.249	0.448	2.277	104	1.06E-02	1.81E-05	1.87E-08
1241.34	2.253	0.451	2.251				
311.94	2.275	0.465	2.264				
78.32	2.301	0.482	2.288				
20.51	2.324	0.496	2.313				
6.04	2.343	0.509	2.334				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

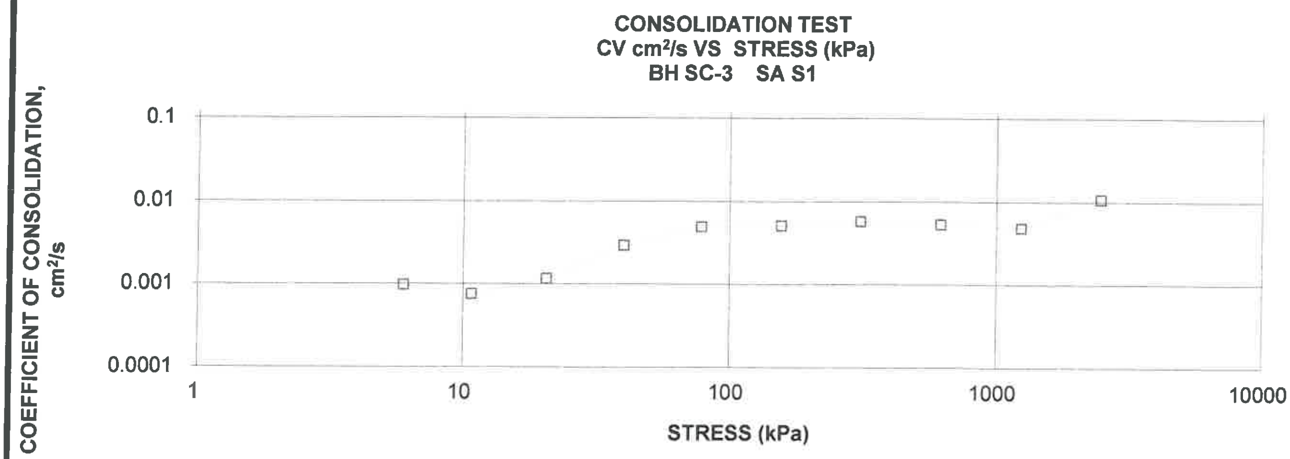
Sample Height, cm	2.34	Unit Weight, kN/m^3	21.73
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	18.00
Area, cm^2	31.58	Specific Gravity, measured	2.77
Volume, cm^3	74.00	Solids Height, cm	1.553
Water Content, %	20.69	Volume of Solids, cm^3	49.04
Wet Mass, g	163.94	Volume of Voids, cm^3	24.96
Dry Mass, g	135.84		

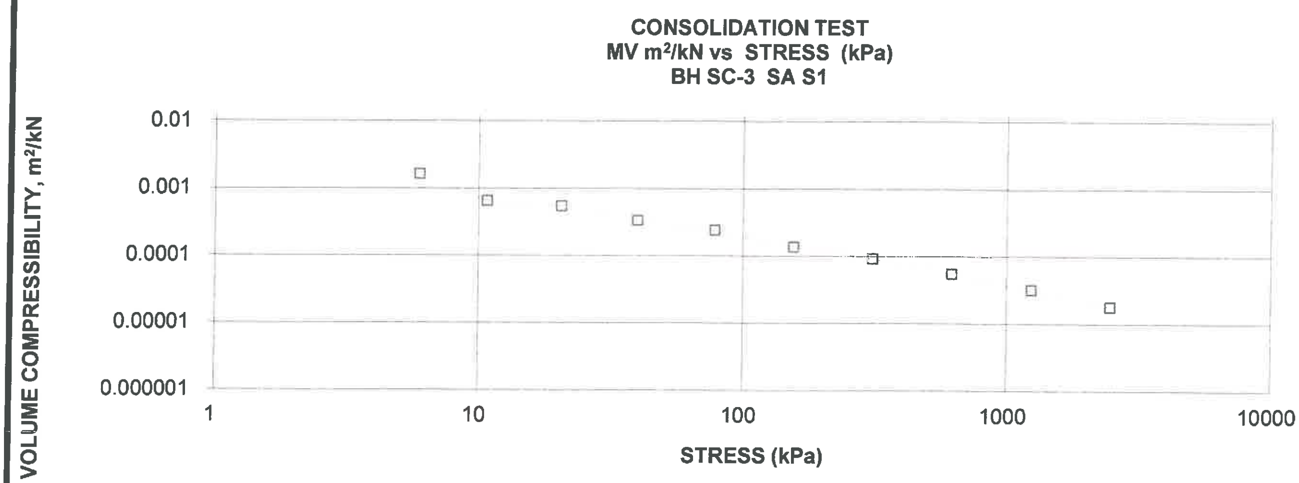
Prepared By: LH | **Golder Associates** | Checked By: [signature]

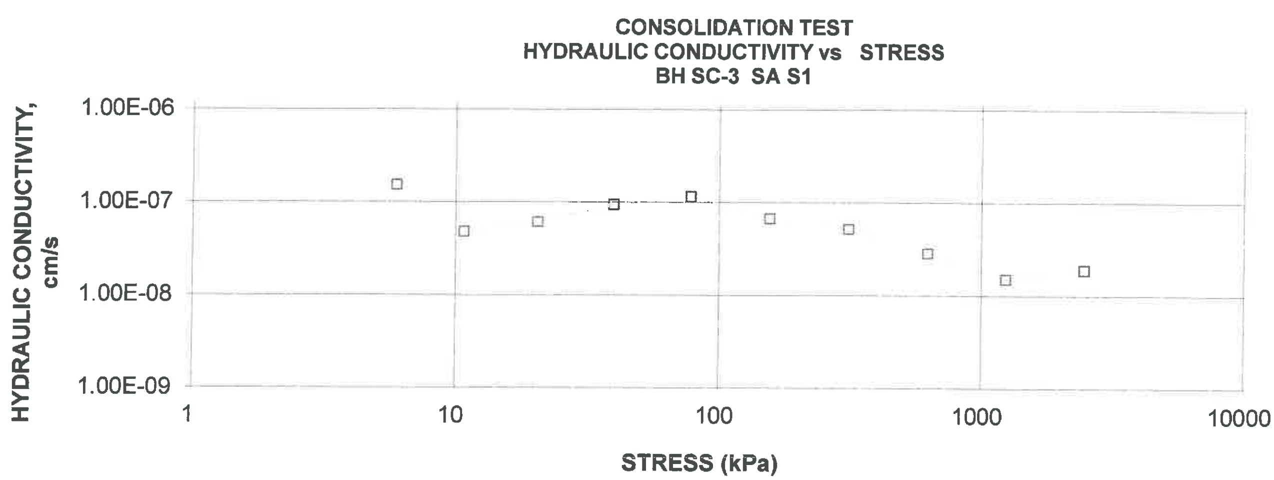
CONSOLIDATION TEST RESULTS

Clayey Silt (Upper Deposit)

FIGURE B5B







Project No. 09-1111-0018

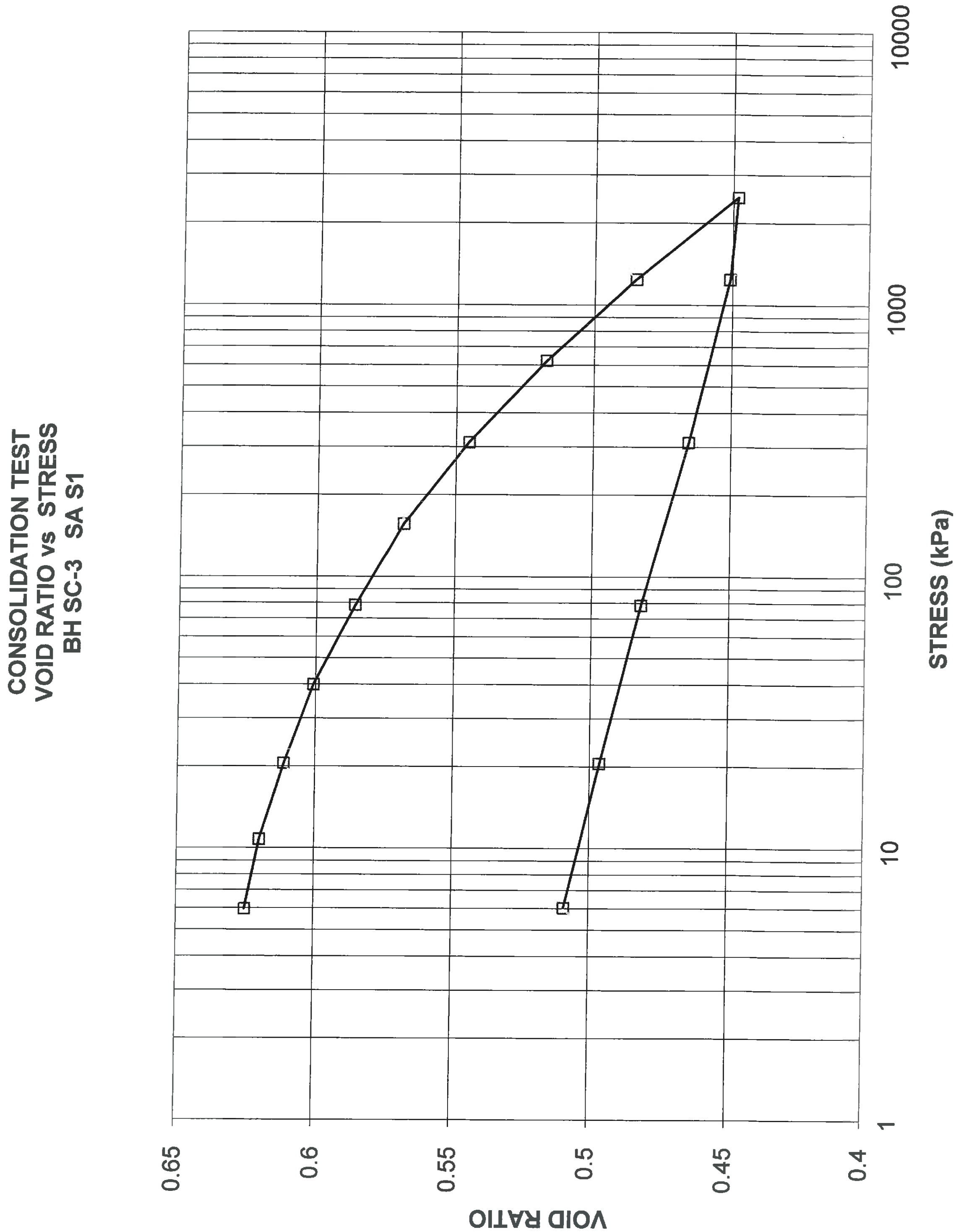
Prepared By: LH

Golder Associates

Checked By: JL

CONSOLIDATION TEST RESULTS
VOID RATIO VS LOG STRESS

FIGURE B5C



Project No.09-1111-0018

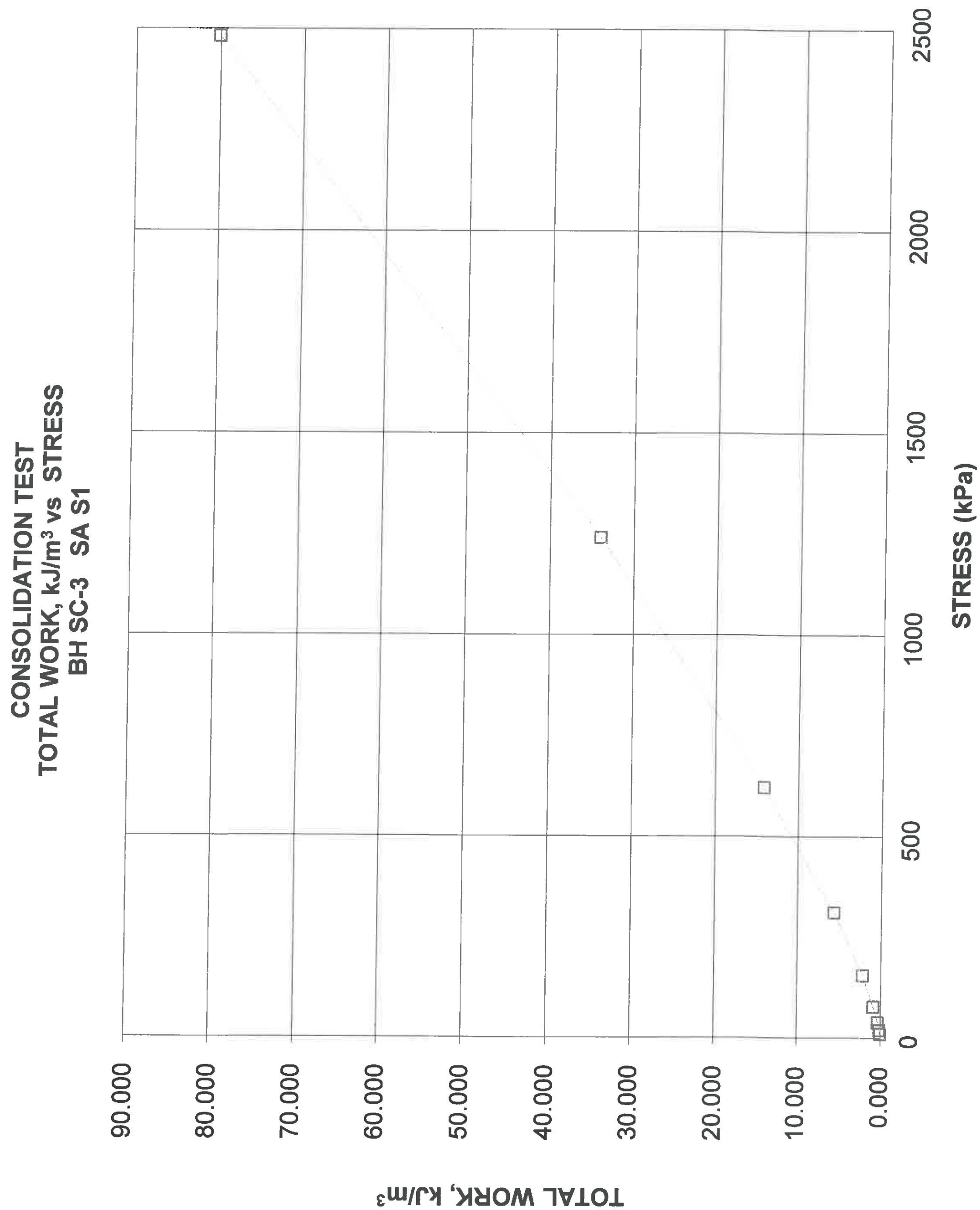
Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST RESULTS
TOTAL WORK VS STRESS

FIGURE B5D



Project No.09-1111-0018

Prepared By: LH

Golder Associates

Checked By: *[signature]*

CONSOLIDATION TEST RESULTS

Clayey Silt (Upper Deposit)

FIGURE B6A

SAMPLE IDENTIFICATION

Project Number	09-1111-0018	Sample Number	T1
Borehole Number	SC-5	Sample Depth, m	12.20-12.65

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	06/26/2012		
Date Completed	07/10/2012		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	20.11
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	16.29
Area, cm^2	31.58	Specific Gravity, measured	2.72
Volume, cm^3	80.09	Solids Height, cm	1.548
Water Content, %	23.49	Volume of Solids, cm^3	48.90
Wet Mass, g	164.26	Volume of Voids, cm^3	31.18
Dry Mass, g	133.01	Degree of Saturation, %	100.2

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.536	0.638	2.536				
5.95	2.513	0.623	2.525	1417	9.54E-04	1.52E-03	1.42E-07
10.66	2.501	0.615	2.507	1009	1.32E-03	9.80E-04	1.27E-07
20.63	2.487	0.606	2.494	1058	1.25E-03	5.70E-04	6.96E-08
39.97	2.466	0.592	2.476	540	2.41E-03	4.30E-04	1.02E-07
78.79	2.439	0.575	2.452	614	2.08E-03	2.77E-04	5.64E-08
156.24	2.404	0.552	2.421	457	2.72E-03	1.78E-04	4.74E-08
312.03	2.352	0.519	2.378	520	2.31E-03	1.31E-04	2.96E-08
622.07	2.276	0.470	2.314	427	2.66E-03	9.64E-05	2.51E-08
1242.57	2.209	0.427	2.243	240	4.44E-03	4.25E-05	1.85E-08
2482.49	2.150	0.388	2.179	265	3.80E-03	1.90E-05	7.07E-09
1242.57	2.150	0.389	2.150				
312.03	2.170	0.401	2.160				
78.79	2.191	0.415	2.180				
20.63	2.220	0.434	2.205				
5.95	2.235	0.443	2.228				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.24	Unit Weight, kN/m^3	22.06
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	18.48
Area, cm^2	31.58	Specific Gravity, measured	2.72
Volume, cm^3	70.59	Solids Height, cm	1.548
Water Content, %	19.39	Volume of Solids, cm^3	48.90
Wet Mass, g	158.80	Volume of Voids, cm^3	21.69
Dry Mass, g	133.01		

Prepared By: LH

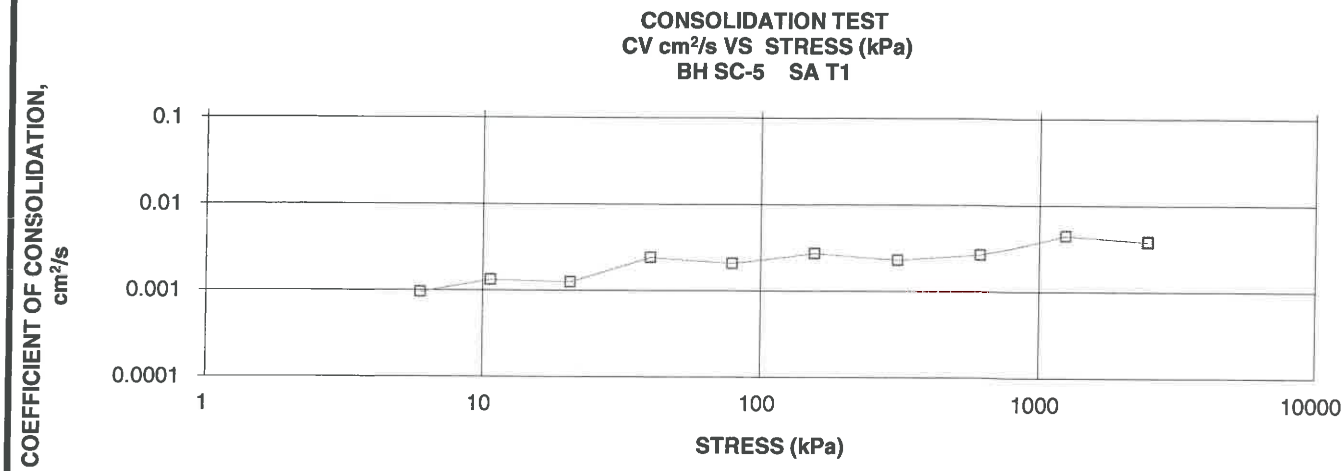
Golder Associates

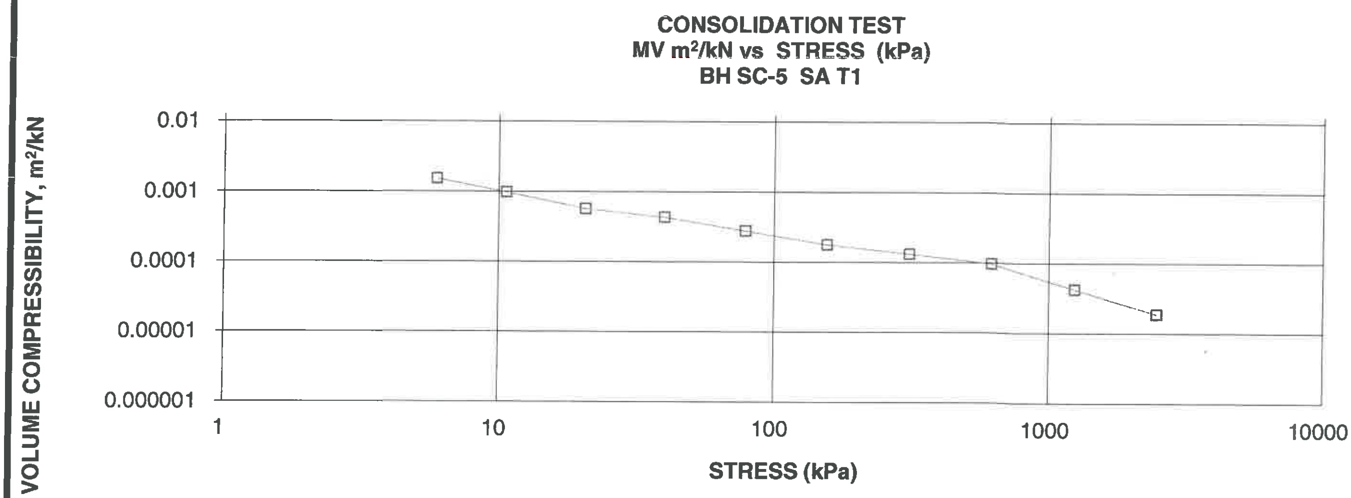
Checked By: [illegible]

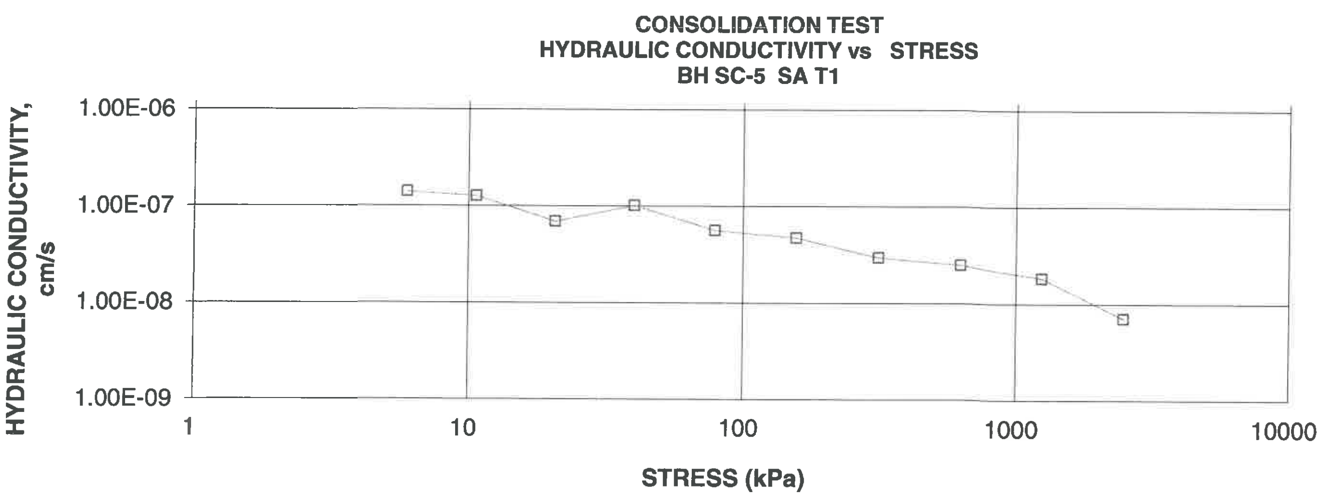
CONSOLIDATION TEST RESULTS

Clayey Silt (Upper Deposit)

FIGURE B6B







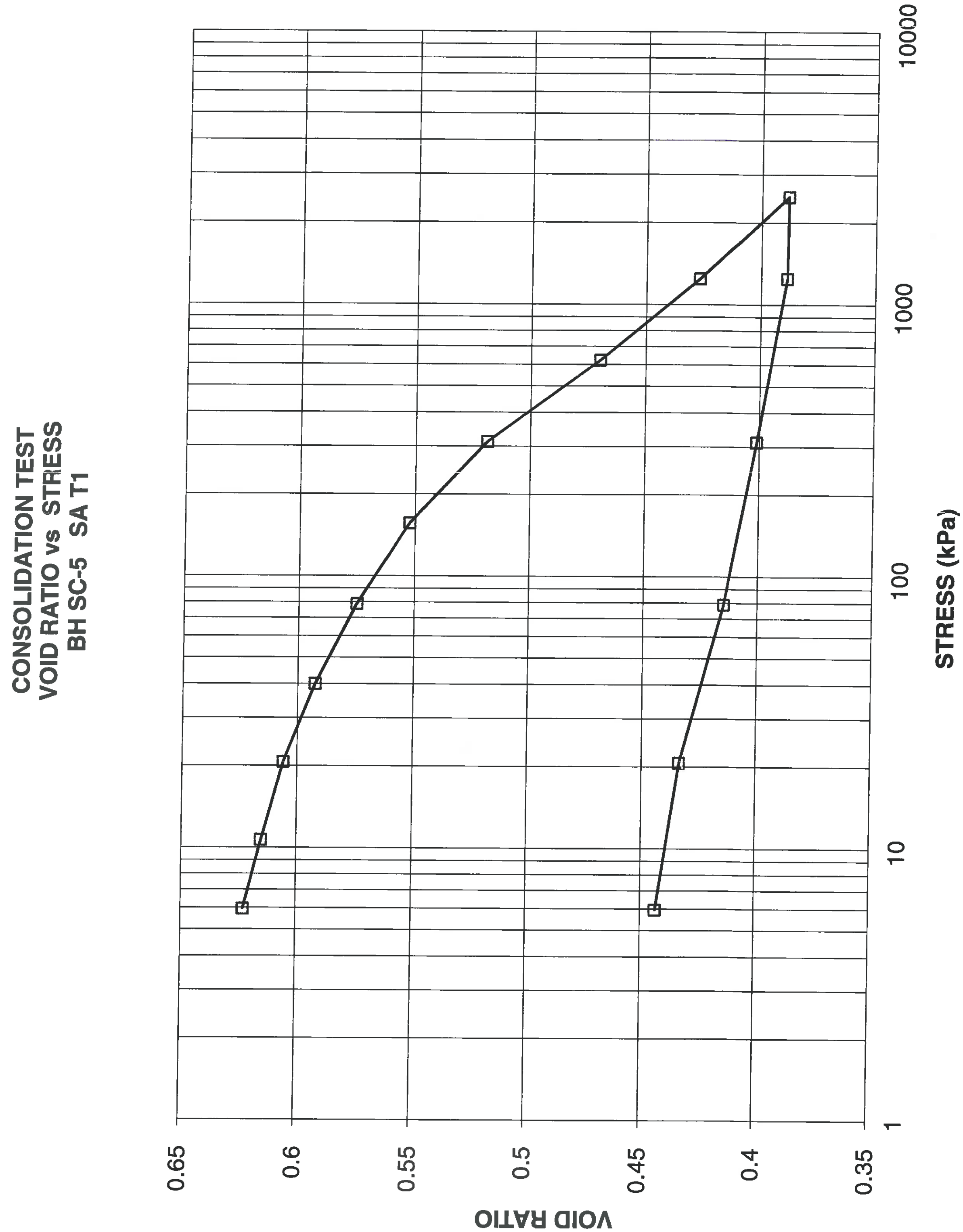
Project No. 09-1111-0018

Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST RESULTS VOID RATIO VS LOG STRESS	FIGURE B6C



Project No.09-1111-0018

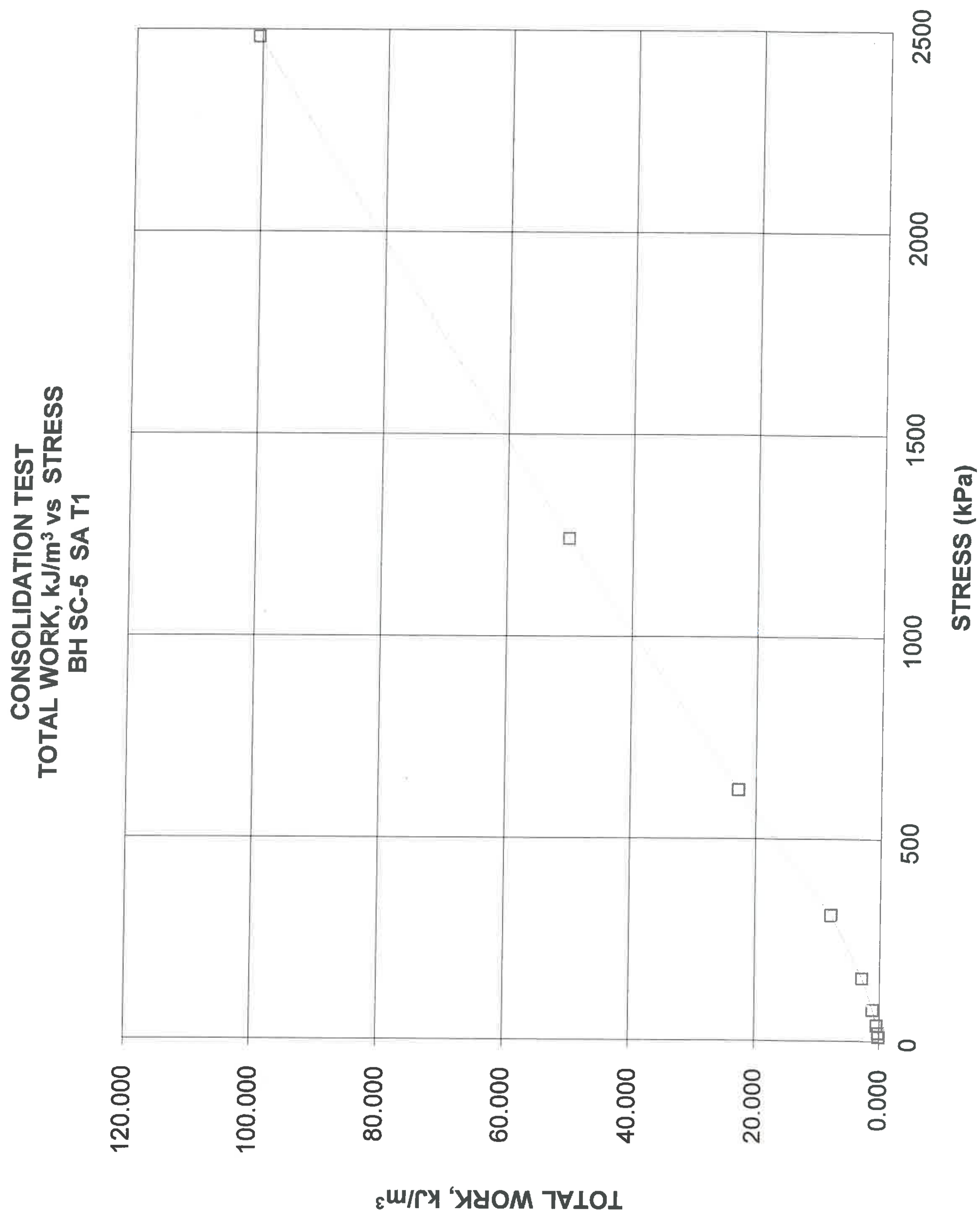
Prepared By: LH

Golder Associates

Checked By: *JC*

CONSOLIDATION TEST RESULTS
TOTAL WORK VS STRESS

FIGURE B6D



Project No.09-1111-0018

Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST RESULTS

Clayey Silt (Upper Deposit)

FIGURE B7A

SAMPLE IDENTIFICATION

Project Number	09-1111-0018	Sample Number	11
Borehole Number	SC-7	Sample Depth, m	10.67-11.28

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	06/20/2012		
Date Completed	07/05/2012		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	20.59
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	16.64
Area, cm^2	31.43	Specific Gravity, measured	2.76
Volume, cm^3	59.65	Solids Height, cm	1.167
Water Content, %	23.75	Volume of Solids, cm^3	36.67
Wet Mass, g	125.26	Volume of Voids, cm^3	22.98
Dry Mass, g	101.22	Degree of Saturation, %	104.6

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.898	0.627	1.898				
6.55	1.863	0.596	1.880	1848	4.06E-04	2.85E-03	1.13E-07
11.23	1.851	0.586	1.857	4133	1.77E-04	1.33E-03	2.30E-08
21.21	1.835	0.573	1.843	1370	5.26E-04	8.39E-04	4.32E-08
40.58	1.808	0.550	1.822	470	1.50E-03	7.26E-04	1.07E-07
79.64	1.783	0.528	1.796	622	1.10E-03	3.39E-04	3.65E-08
160.90	1.750	0.500	1.766	454	1.46E-03	2.16E-04	3.08E-08
313.19	1.706	0.462	1.728	406	1.56E-03	1.52E-04	2.32E-08
624.68	1.662	0.425	1.684	228	2.64E-03	7.41E-05	1.91E-08
1247.80	1.616	0.385	1.639	265	2.15E-03	3.91E-05	8.25E-09
2494.02	1.570	0.345	1.593	217	2.48E-03	1.95E-05	4.73E-09
1247.80	1.574	0.349	1.572				
313.19	1.592	0.365	1.583				
79.64	1.616	0.385	1.604				
21.21	1.631	0.398	1.624				
6.55	1.646	0.411	1.639				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.65	Unit Weight, kN/m^3	22.67
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	19.19
Area, cm^2	31.43	Specific Gravity, measured	2.76
Volume, cm^3	51.73	Solids Height, cm	1.167
Water Content, %	18.14	Volume of Solids, cm ^{3}	36.67
Wet Mass, g	119.58	Volume of Voids, cm ^{3}	15.06
Dry Mass, g	101.22		

Prepared By: LH

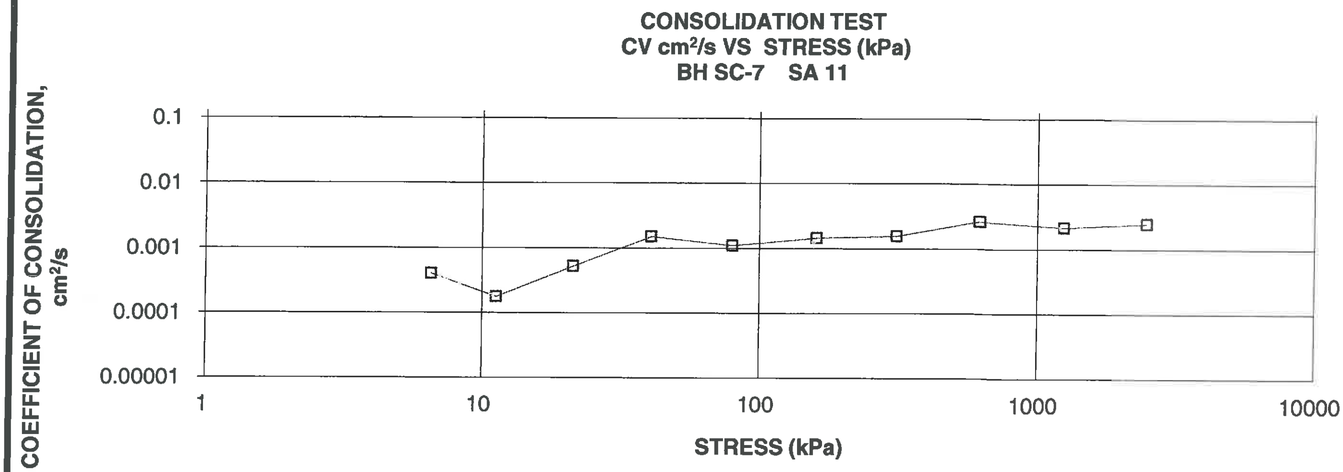
Golder Associates

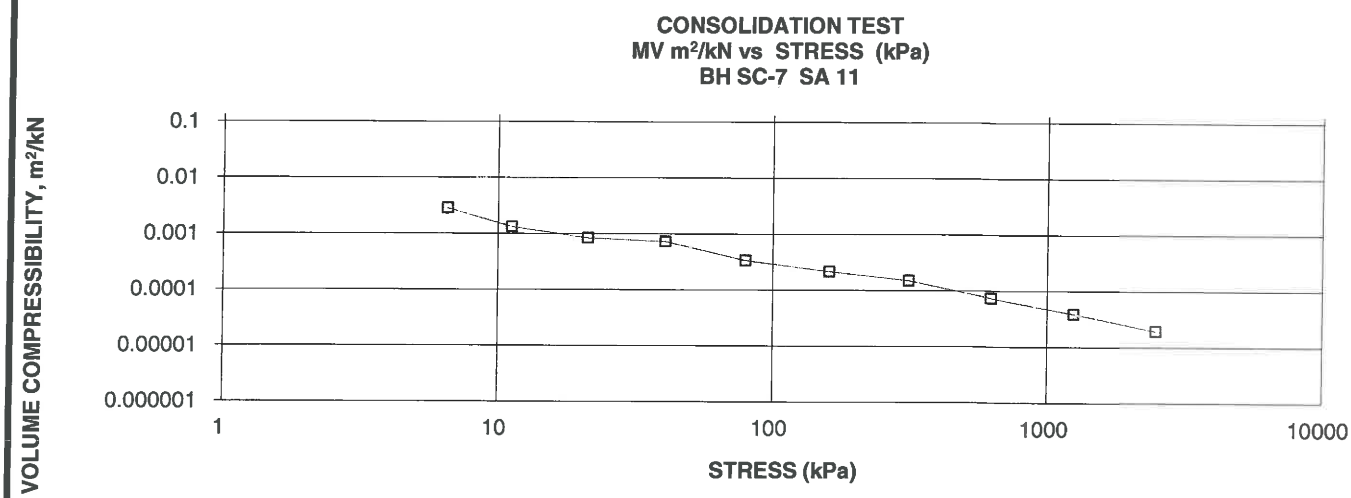
Checked By: JK

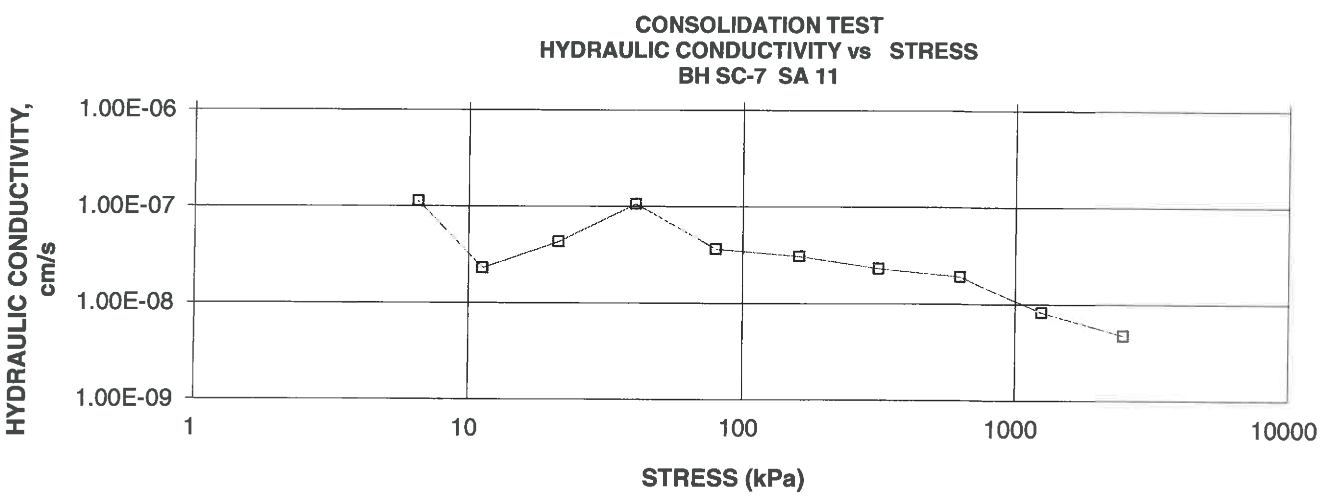
CONSOLIDATION TEST RESULTS

Clayey Silt (Upper Deposit)

FIGURE B7B







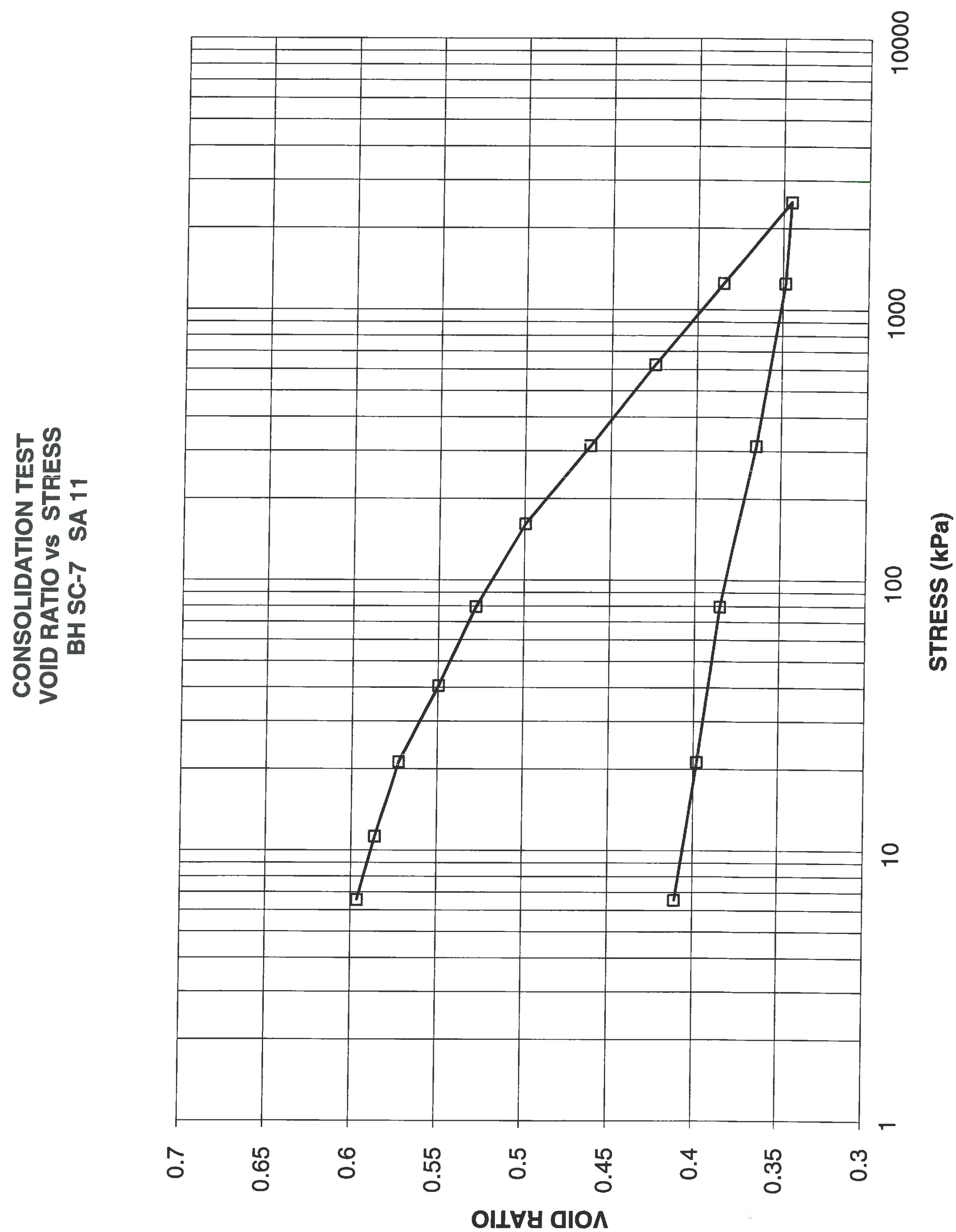
Project No. 09-1111-0018

Prepared By: LH

Golder Associates

Checked By: *[signature]*

CONSOLIDATION TEST RESULTS VOID RATIO VS LOG STRESS	FIGURE B7C



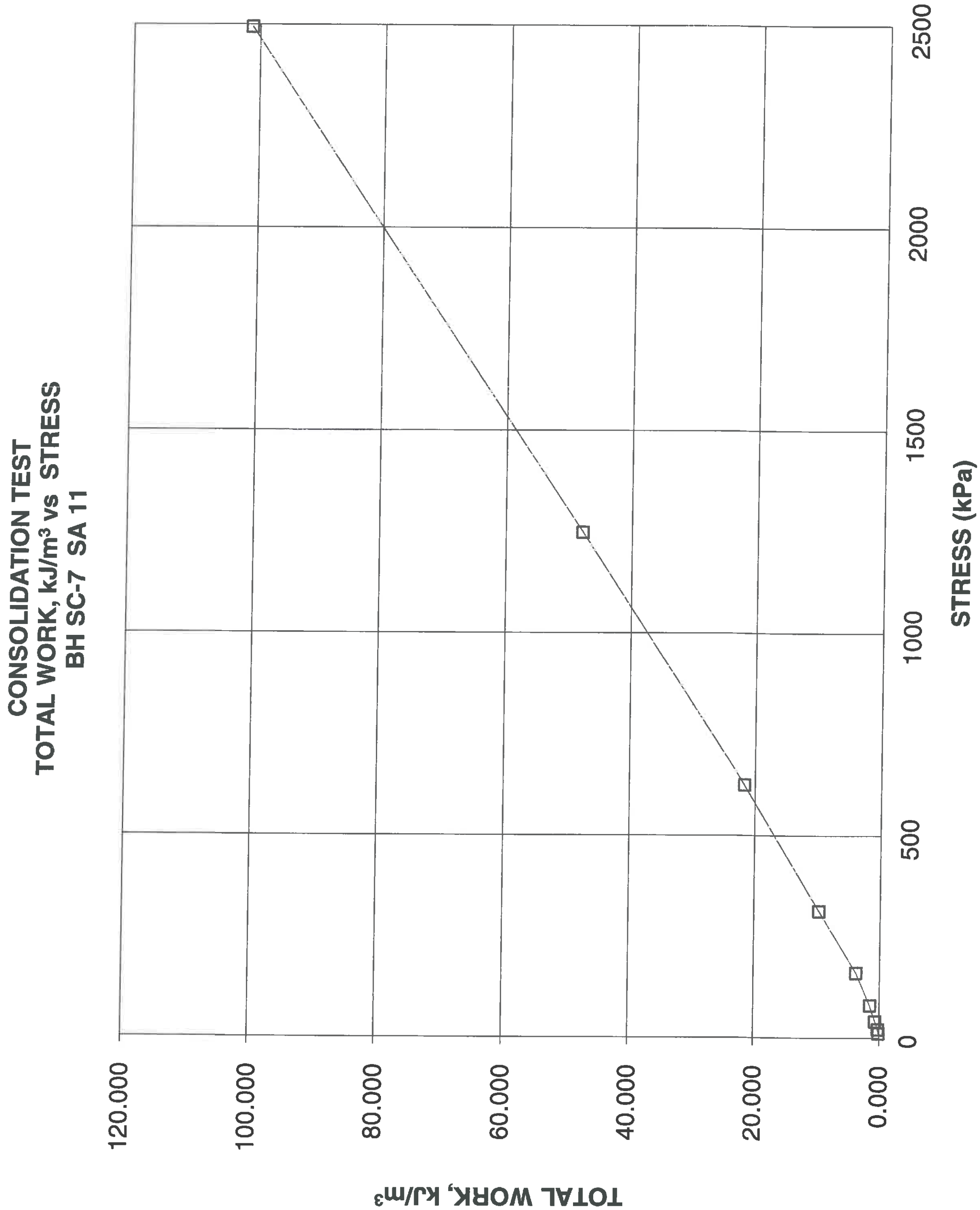
Project No.09-1111-0018

Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST RESULTS TOTAL WORK VS STRESS	FIGURE B7D



Project No.09-1111-0018

Prepared By: LH

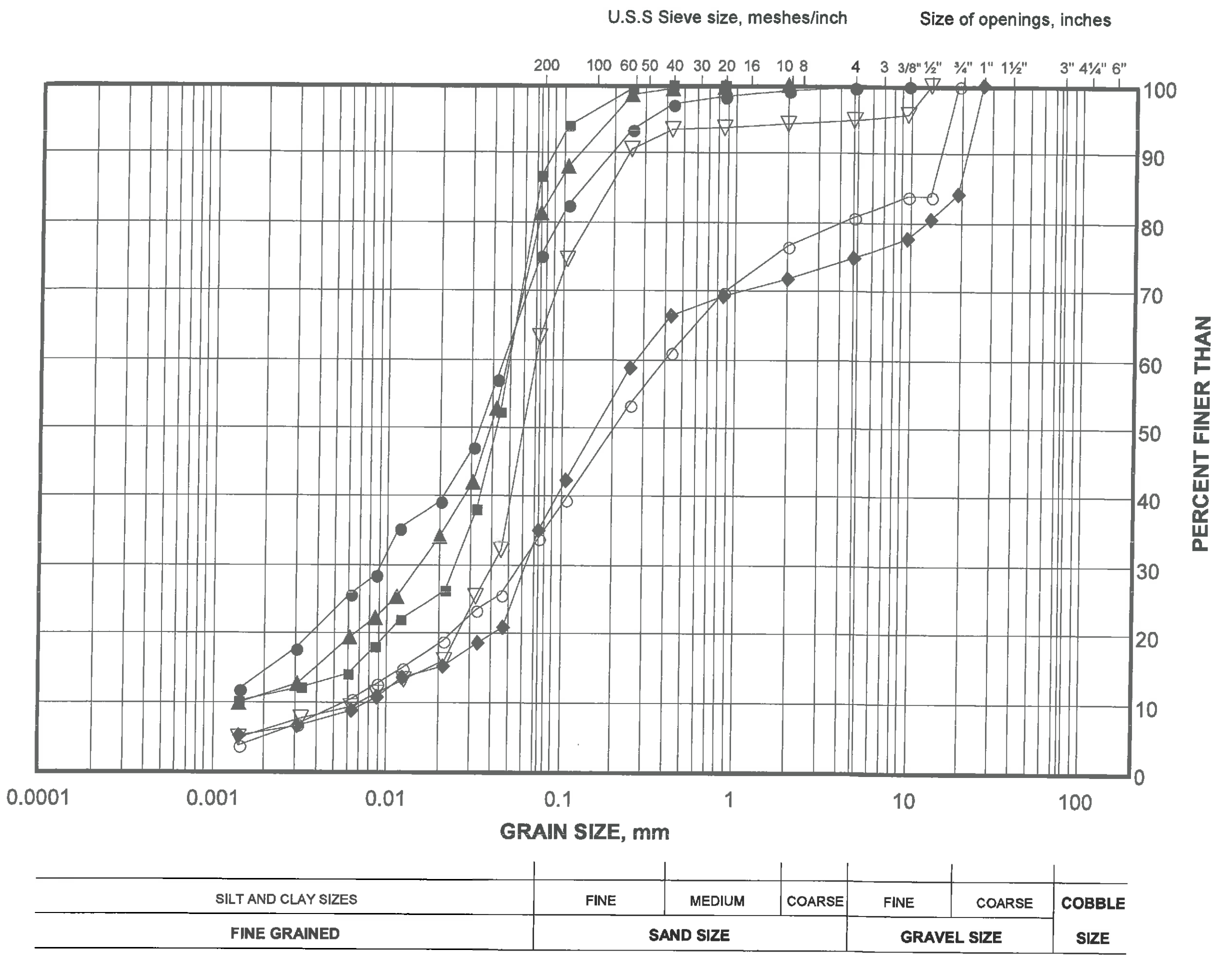
Golder Associates

Checked By: KL

GRAIN SIZE DISTRIBUTION TEST RESULTS

Silty Sand to Sandy Silt Interlayers

FIGURE B8



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SC-2	4	219.4
■	SC-9	4	218.4
◆	SC-10	5	218.8
▲	SC-4	5	217.5
▽	SC-14	5A	218.7
○	BO-9	7A	214.8

Project Number: 09-1111-0018

Checked By: [signature]

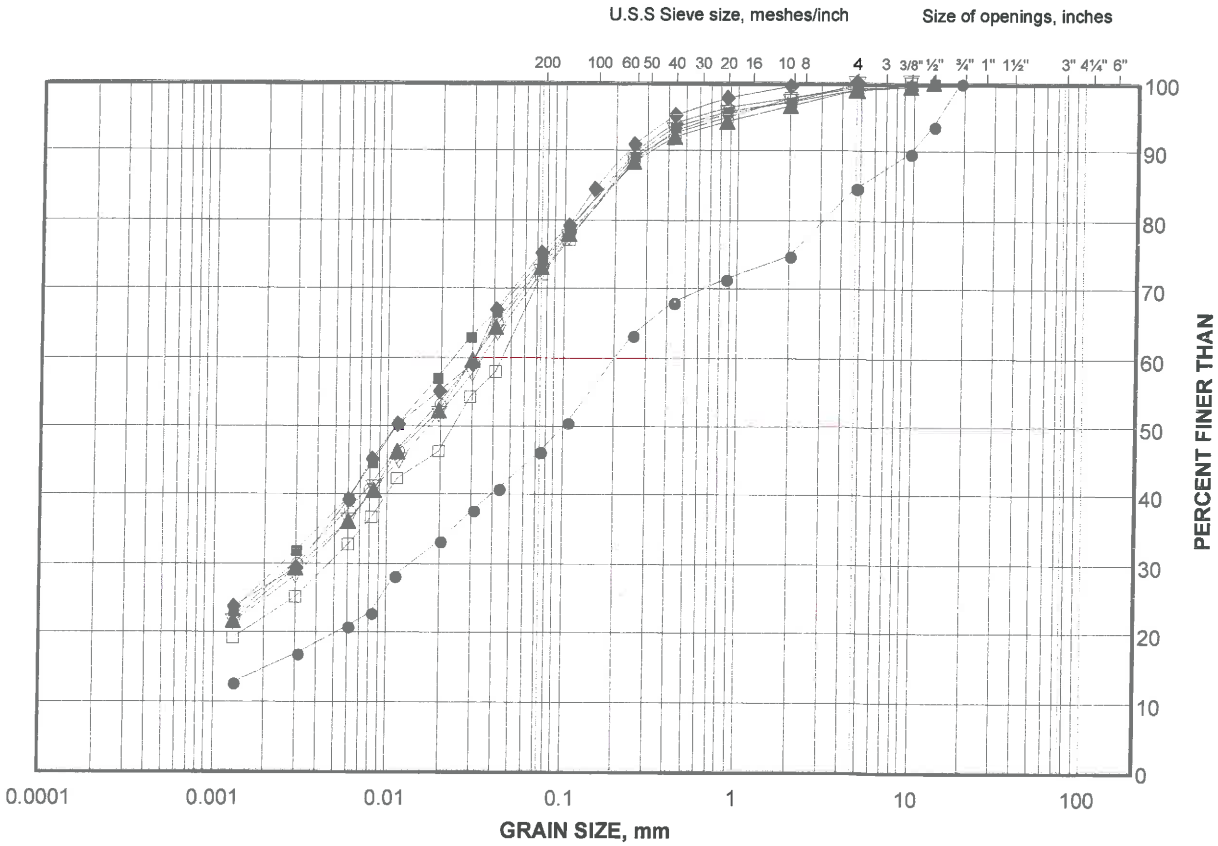
Golder Associates

Date: 08-Mar-13

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till to Sandy Silt to Silty Sand

FIGURE B9A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SC-3	10	209.2
■	SC-10	11	211.1
◆	F8-6	11	216.7
▲	SC-9	13	207.0
▽	SC-11	13	207.8
○	SC-14	13	208.0
□	SC-1	13	210.5

Project Number: 09-1111-0018

Checked By: [signature]

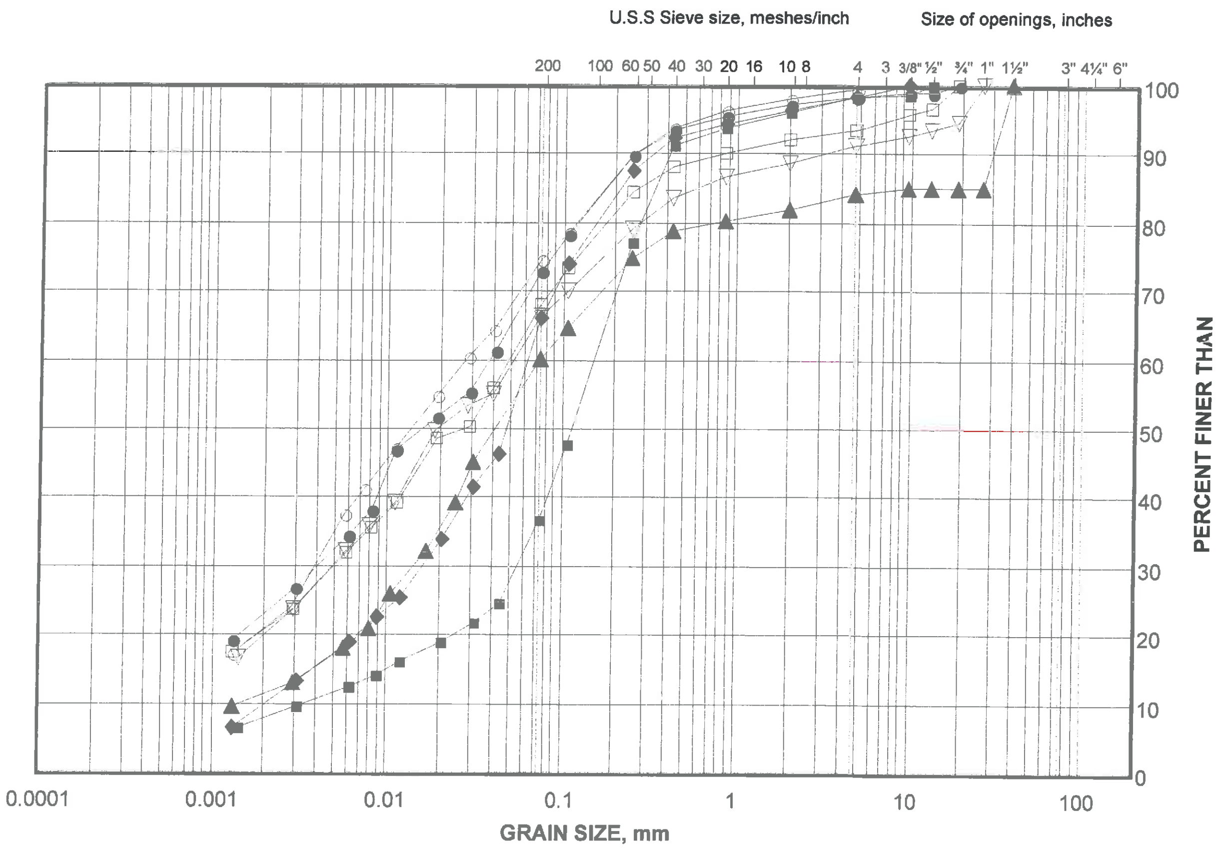
Golder Associates

Date: 08-Mar-13

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till to Sandy Silt to Silty Sand

FIGURE B9B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SC-13	14	211.9
■	SC-2	15	206.4
◆	SC-4	15	203.7
▲	SC-13	20	198.3
▽	SC-2	6	217.9
○	F8-3	7	216.1
□	SC-1	7	218.1

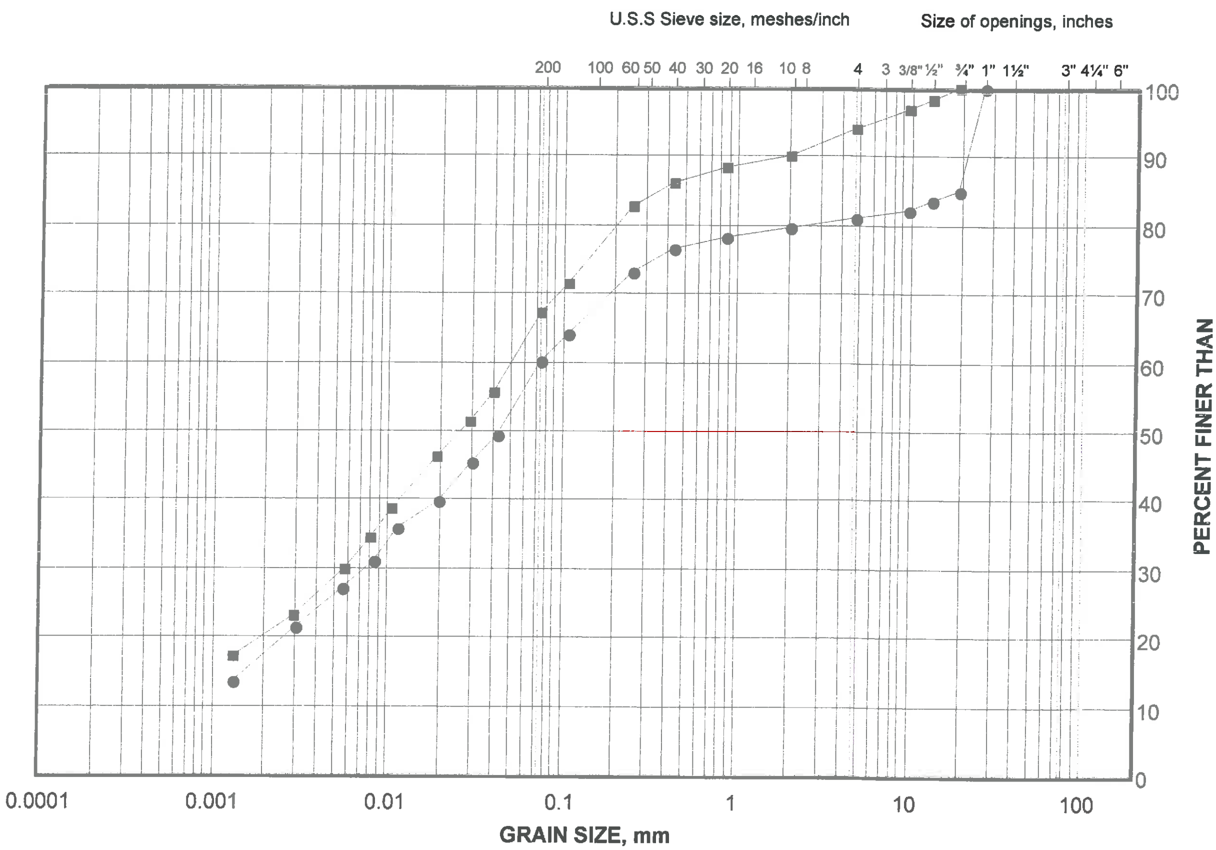
Project Number: 09-1111-0018

Checked By: [signature]

Golder Associates

Date: 08-Mar-13

GRAIN SIZE DISTRIBUTION TEST RESULTS Clayey Silt Till to Sandy Silt to Silty Sand	FIGURE B9C



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

LEGEND

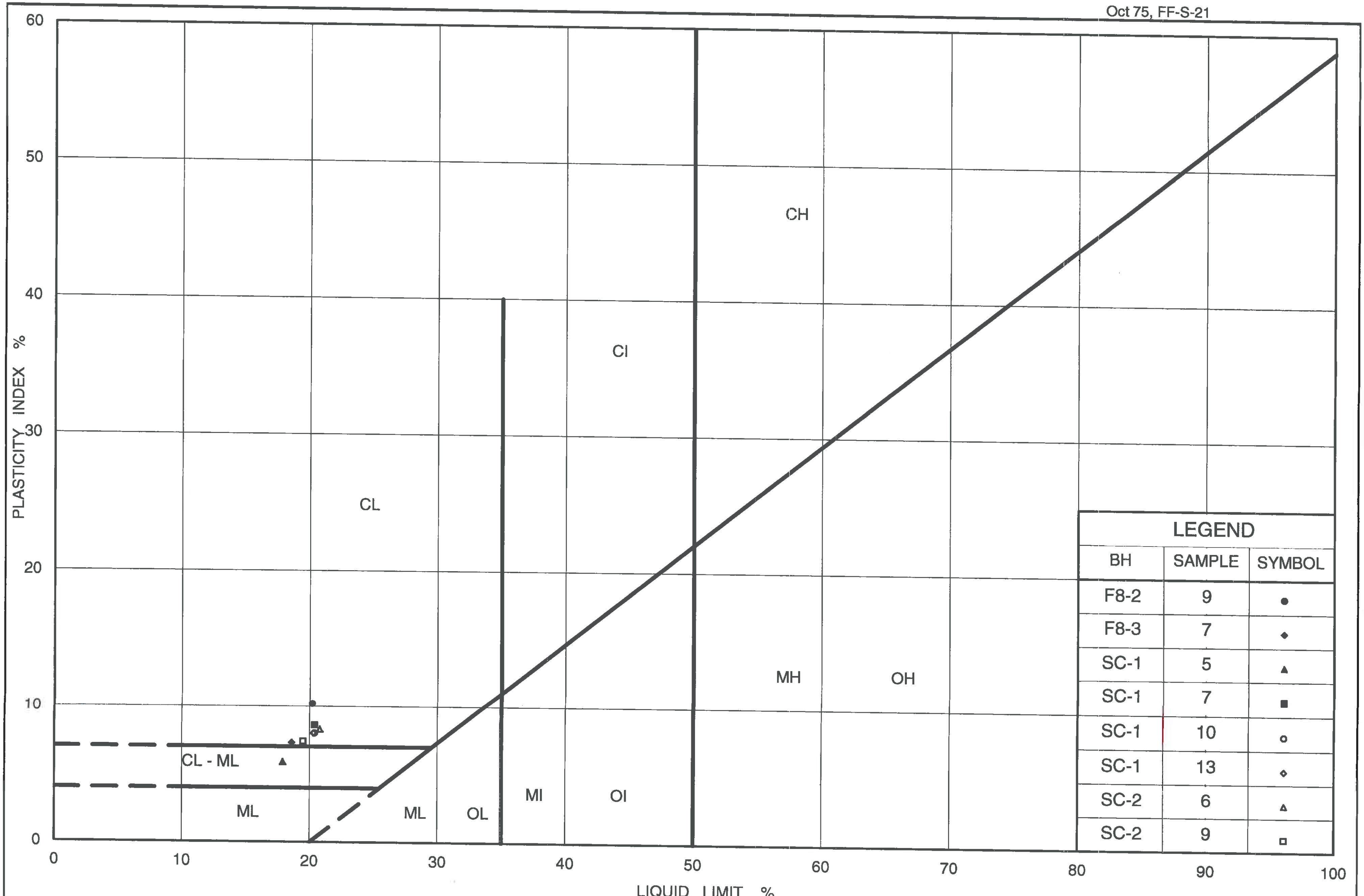
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SC-11	8	215.4
■	F8-2	9	219.8

Project Number: 09-1111-0018

Checked By:

Golder Associates

Date: 08-Mar-13



BH	SAMPLE	SYMBOL
F8-2	9	●
F8-3	7	◆
SC-1	5	▲
SC-1	7	■
SC-1	10	○
SC-1	13	◇
SC-2	6	△
SC-2	9	□

Ministry of Transportation

Ontario

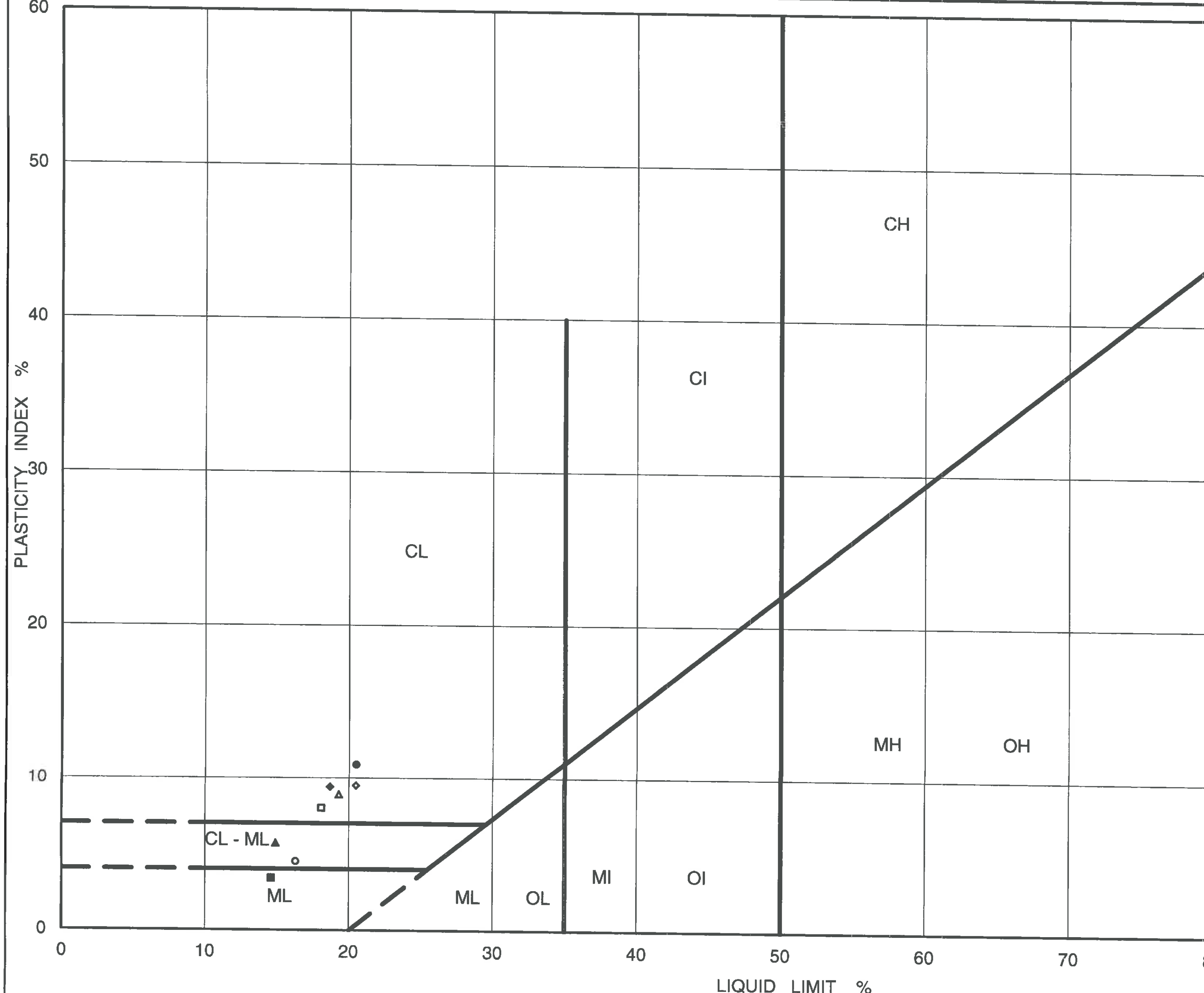
PLASTICITY CHART
Clayey Silt Till to Sandy Silt to Silty Sand Till

Figure No. B10A

Project No. 09-1111-0018

Checked By: [signature]

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
SC-2	11	●
SC-2	13	◆
SC-3	10	▲
SC-4	15	■
SC-4	17	○
SC-9	13	◇
SC-10	11	△
SC-10	14	□

Ministry of Transportation

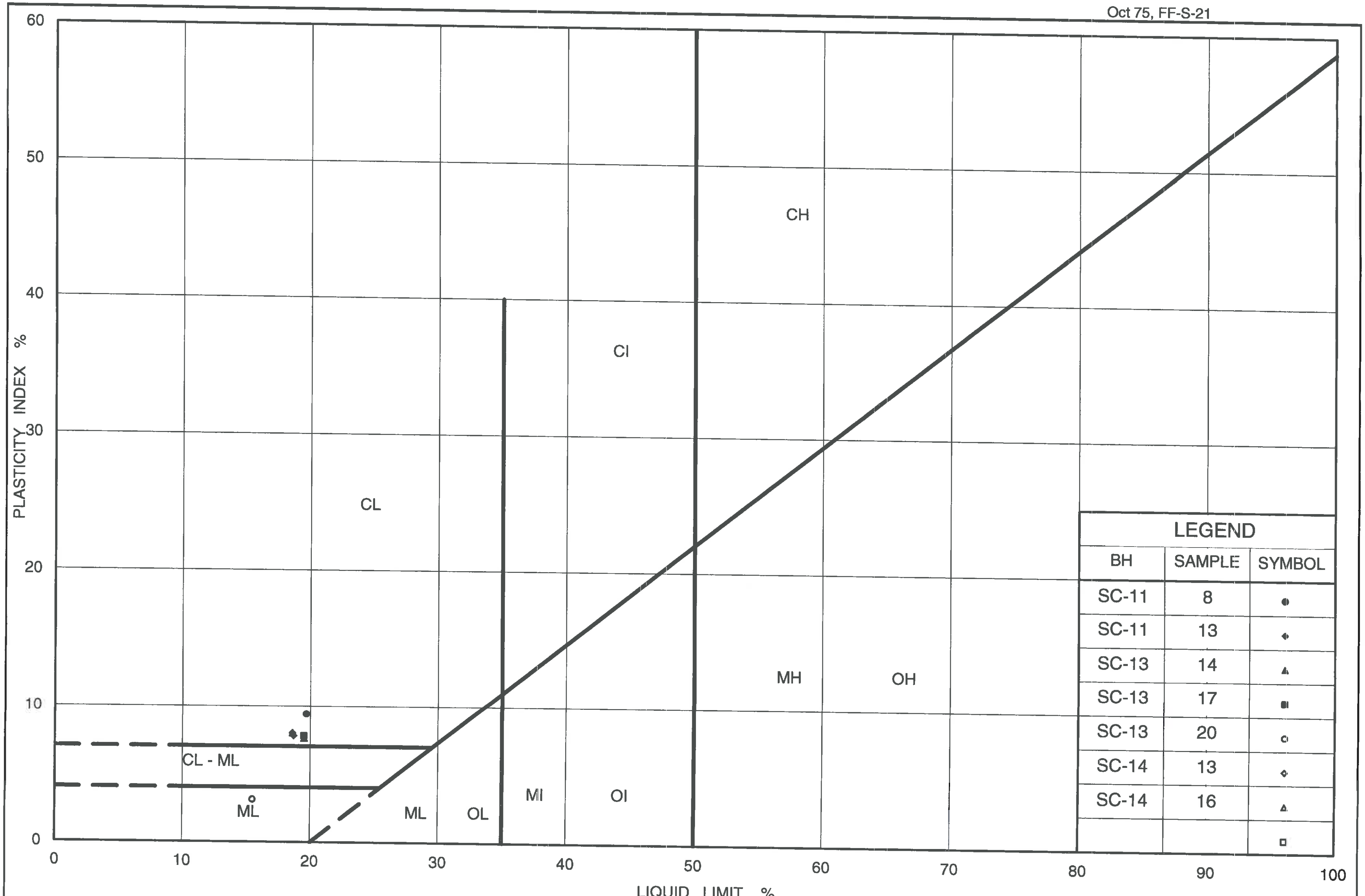
Ontario

PLASTICITY CHART
Clayey Silt Till to Sandy Silt to Silty Sand Till

Figure No. B10B

Project No. 09-1111-0018

Checked By: [signature]



Ministry of Transportation

Ontario

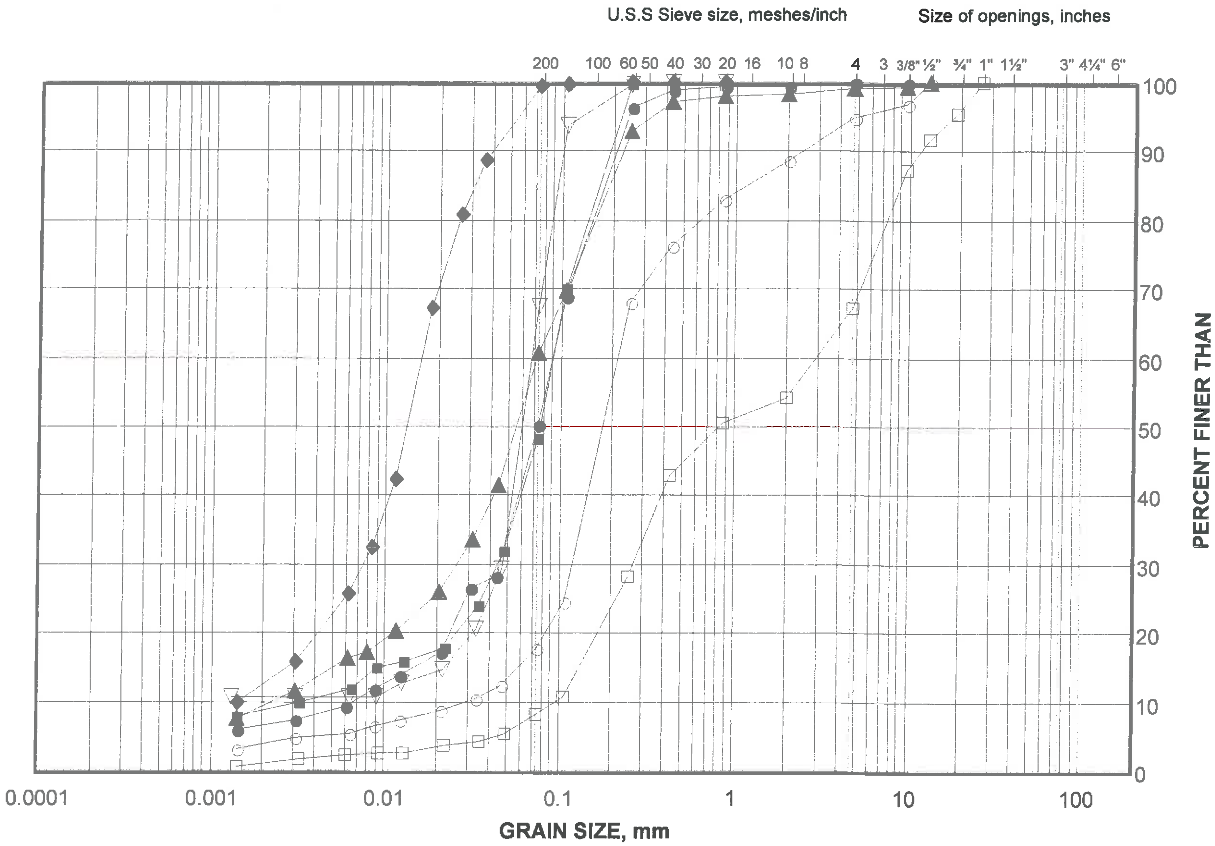
PLASTICITY CHART
Clayey Silt Till to Sandy Silt to Silty Sand Till

Figure No. B10C

Project No. 09-1111-0018

Checked By:

GRAIN SIZE DISTRIBUTION TEST RESULTS Sand and Silt to Sand and Gravel	FIGURE B11A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SC-3	11	207.6
■	SC-3	13	204.6
◆	BO-9	13	204.1
▲	SC-7	14	205.3
▽	BO-9	15	202.4
○	SC-9	17	200.9
□	SC-7	17	200.7

Project Number: 09-1111-0018

Checked By: [signature]

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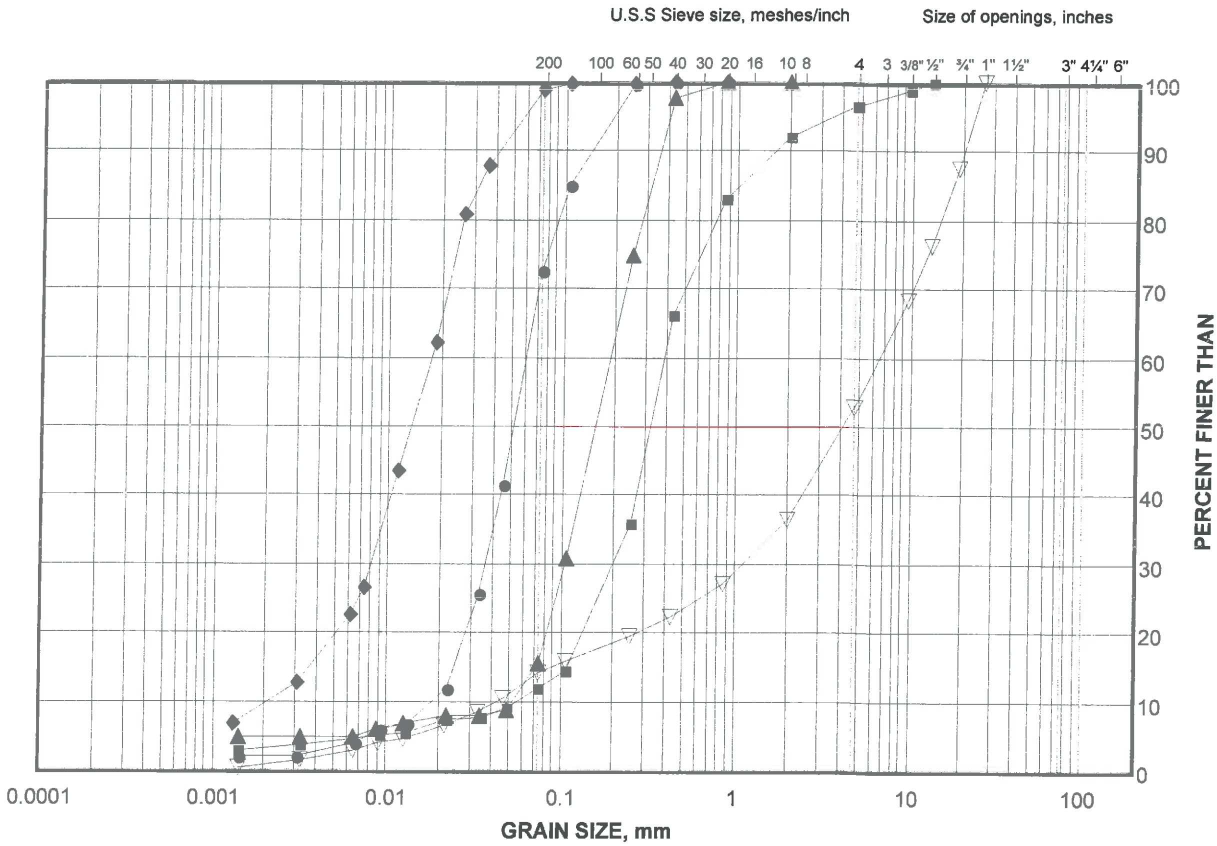
Date: 08-Mar-13

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Silt to Sand and Gravel

FIGURE B11B



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

LEGEND

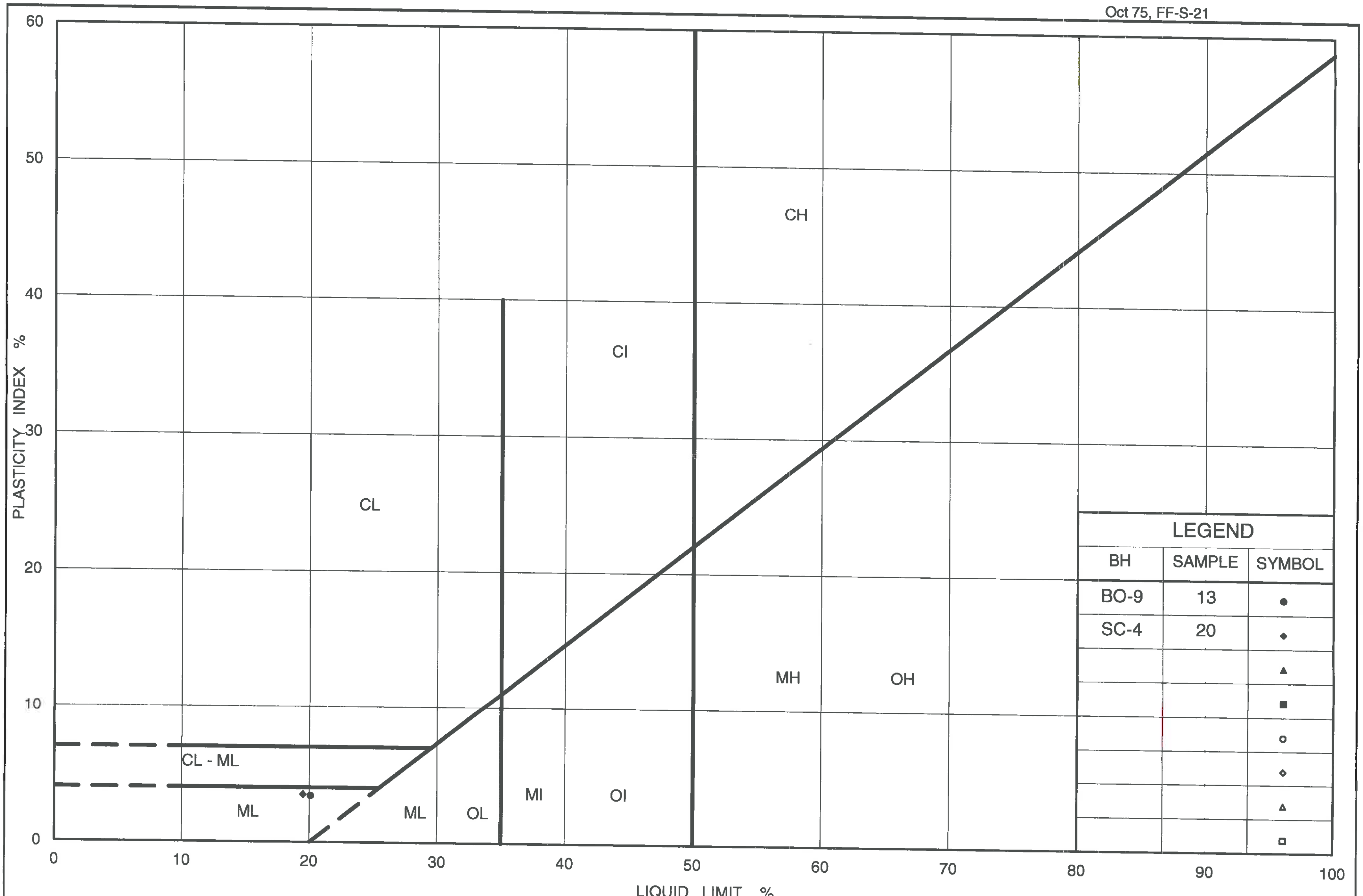
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	SC-4	18	199.2
■	SC-13	19A	201.3
◆	SC-4	20	196.1
▲	SC-7	21	194.6
▽	SC-7	27	180.9

Project Number: 09-1111-0018

Checked By: Wlozye

Golder Associates

Date: 08-Mar-13



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Sand to Sand and Silt

Figure No. B12

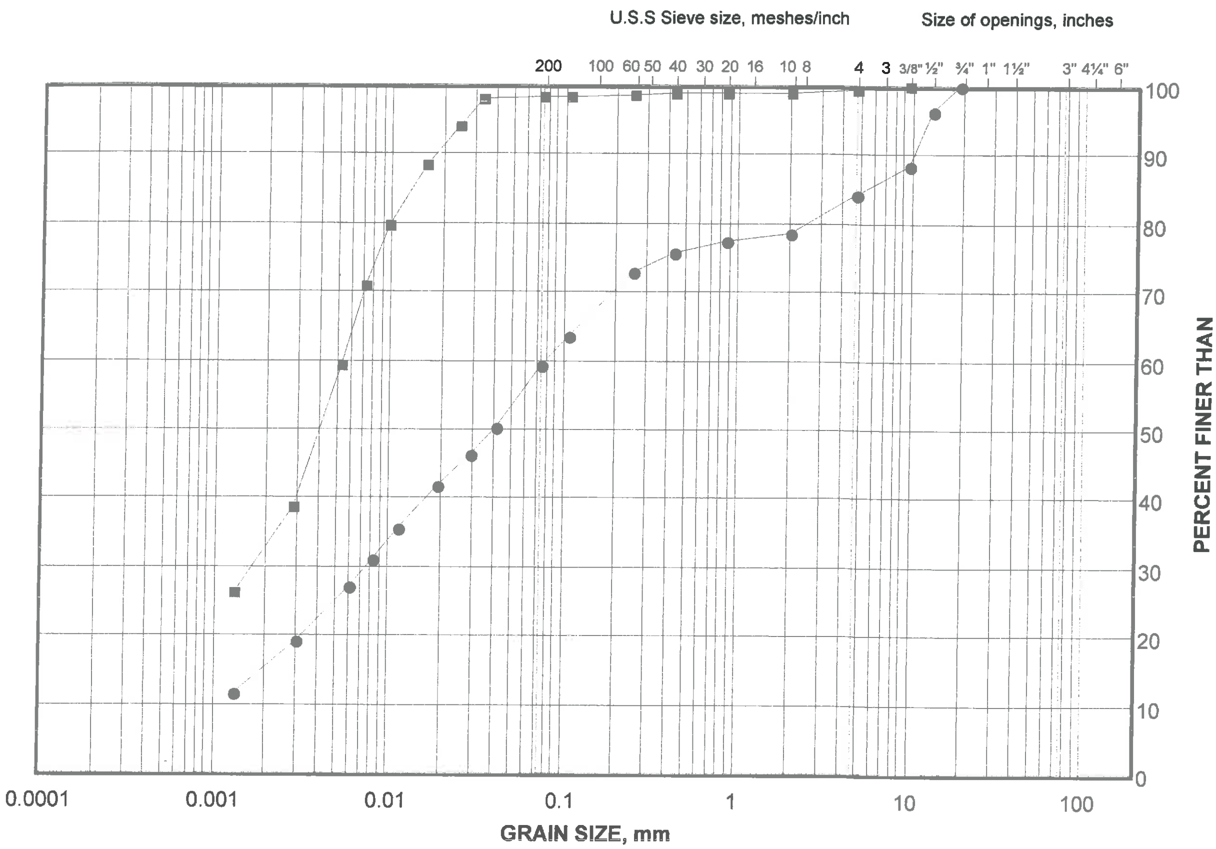
Project No. 09-1111-0018

Checked By:

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Interlayers

FIGURE B13



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

LEGEND

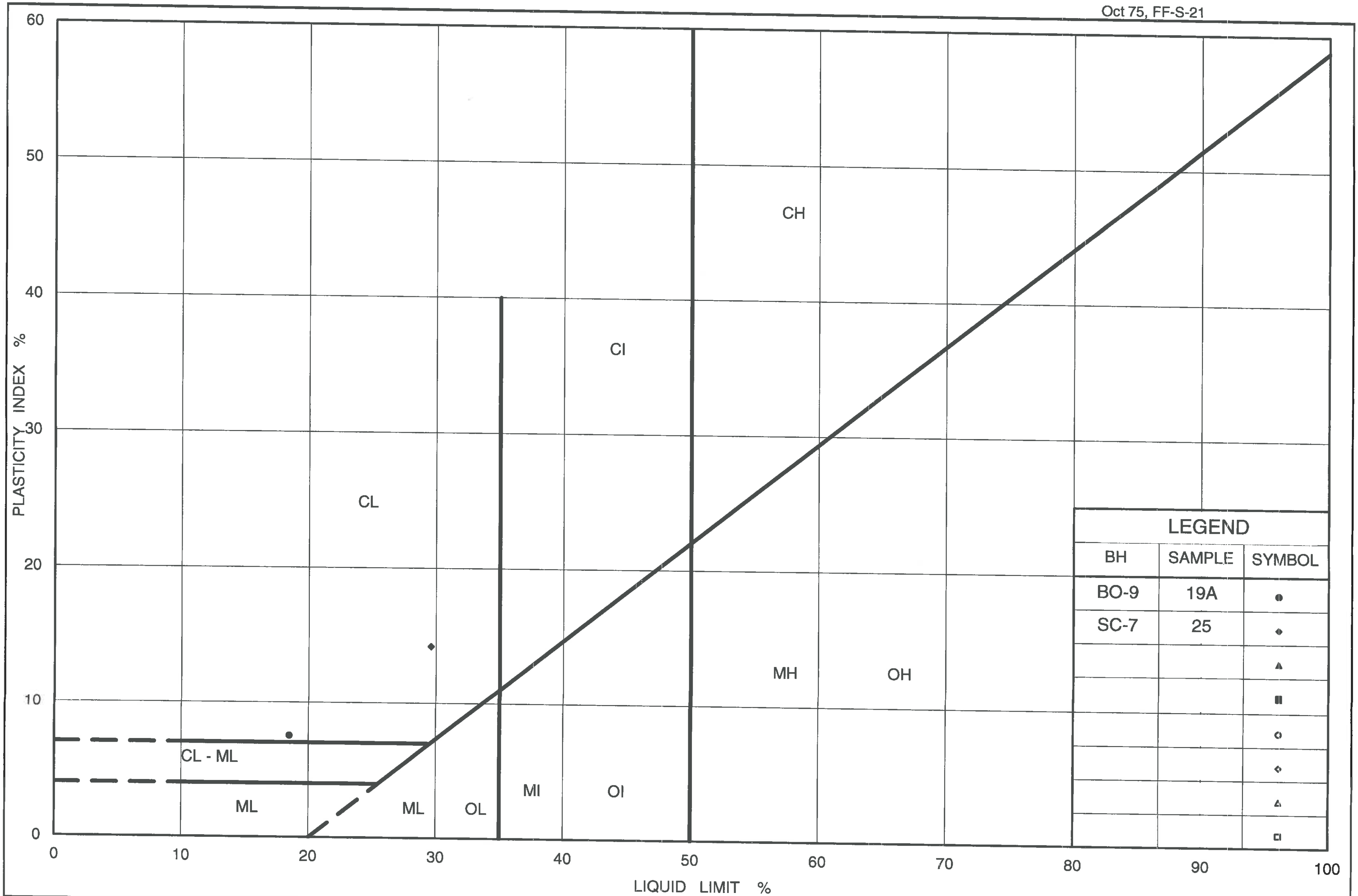
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	BO-9	19A	196.3
■	SC-7	25	186.9

Project Number: 09-1111-0018

Checked By:

Golder Associates

Date: 08-Mar-13



BH	SAMPLE	SYMBOL
BO-9	19A	●
SC-7	25	◆
		▲
		■
		○
		◇
		△
		□

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PLASTICITY CHART
Clayey Silt Interlayers

Figure No. B14

Project No. 09-1111-0018

Checked By: [signature]

APPENDIX C

Borehole Records from Previous Investigation

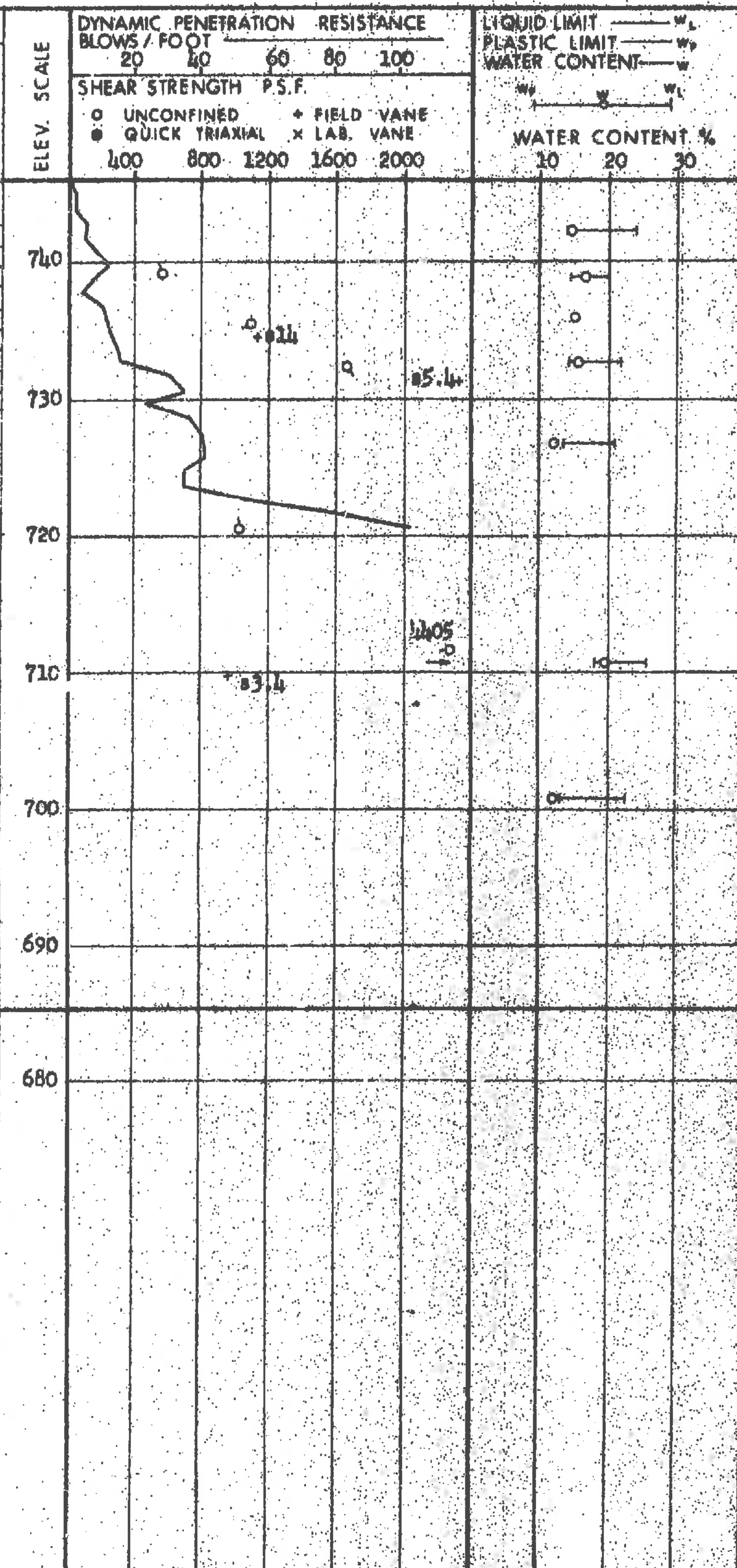
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1 BH 29-1

FOUNDATION SECTIO

JOB	70-11089	LOCATION	Sta. 587 + 10 o/s 50.5' Rt.	ORIGINATED BY	VK
W.P.	105-70-04	BORING DATE	Nov. 12, 1970	COMPILED BY	SAA
DATUM	Geodetic	BOREHOLE TYPE	Washboring, NX Casing	CHECKED BY	4K

(M)	ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS
227.4	745.9	Ground Level					
	0.0	Clayey silt with trace of sand & gravel (Embankment Fill) Firm - Hard	1	SS	12		
			2	TW	PM	132	1.0% org.
			3	TW	PM	132	1.0% org. ∇ 733.9
			4	TW	PM	133	in open BH Nov.17/70 223.7m
			5	SS	32		
			6	SS	48		
		Brown	7	SS	31	129	
218.5	716.9						
8.8	29.0	Silty sand with trace of gravel. Very Dense. Grey	8	SS	62		
217.3	712.9						
10.1	33.0	Clayey silt with trace of sand and gravel Stiff	9	TW	PM	130	
			10	TW	PM		
			11	TW	PM		
		Grey	12	TW	PM		
211.2	692.9						
16.2	53.0	Het.mix.of silt,sand & gravel,trace of clay (Glacial Till) Dense to Very Dense Grey	13	SS	47		
[illegible]	685.4		14	SS	124		
18.4	60.5	End of Borehole					



20
15 -o- 5 % STRAIN AT FAILURE
10

FORM OB-MT-126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

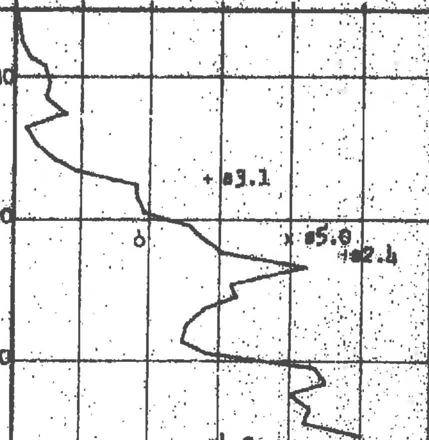
RECORD OF BOREHOLE No2 BH 29-2

FOUNDATION SECTION

JOB 70-11089	LOCATION Sta. 587 + 07 o/s 54' Lt.		ORIGINATED BY VK
W.P. 105-70-04	BORING DATE Nov. 13, 1970		COMPILED BY SAA
DATUM Geodetic	BOREHOLE TYPE Washboring-NX Casing		CHECKED BY

(m)	ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS
223.9	734.7	Ground Level					
	0.0	Clayey silt with trace of sand & gravel (Embankment Fill)	1	SS	19		729.? in open BH Nov.17/70 222.2m
		Firm to very stiff	2	SS	7		
			3	TW	PM		
		Brown	4	SS	12		
218.3	716.2		5	TW	PM		
5.6	18.5	Silty sand with trace of gravel Compact Grey	6	SS	23	133.5	
216.6	710.7						
7.3	24.0	Clayey silt with trace of sand & gravel	7	TW	PM		
		Stiff	8	TW	PM		
			9	TW	PM	128.5	
		Grey	10	TW	PM		
210.9	692.2						
13.0	42.5	Het. mix. of silt, sand & gravel, trace of clay Glacial Till	11	SS	89		
		Very Dense	12	SS	85		
		Grey	13	SS	113		
205.6	674.7		14	SS	108/3"		
18.3	60.0	End of Borehole					

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB. VANE
400 800 1200 1600 2000
LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w
WATER CONTENT % 10 20 30
ELEV. SCALE 730 720 710 700 690 680 670



Shear strength plots: +s3.1; ×s5.0; +s2.4; +s4.0; ×s4.1; +s4.3; +s2.5

20
15 ◇ 5 % STRAIN AT FAILURE
10

FORM OB-MT-126 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

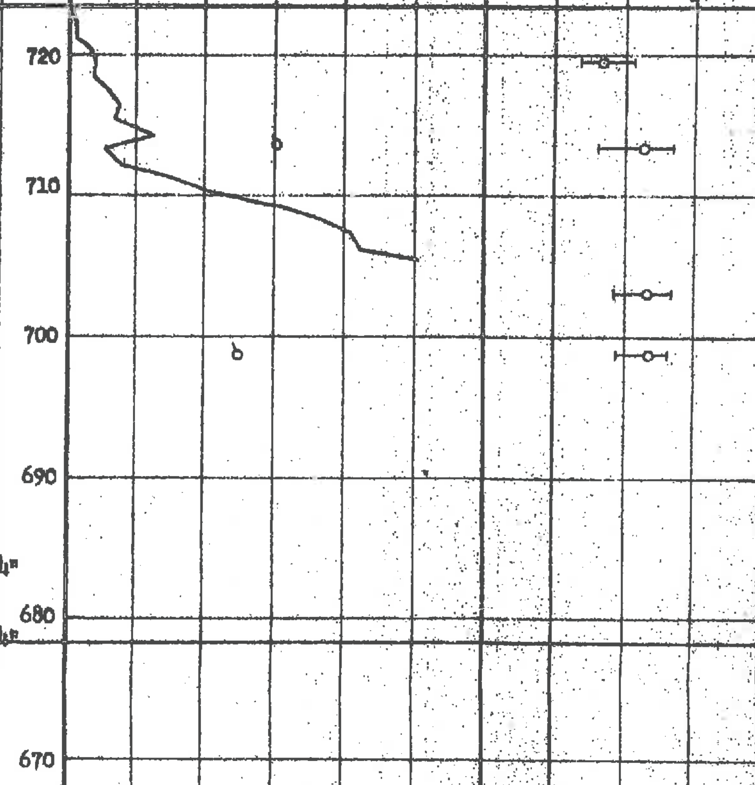
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3 BH 29-3

FOUNDATION SECTION

JOB 70-11089	LOCATION Sta. 587 + 87 O/S 59' Rt.	ORIGINATED BY VK
W.P. 105-70-04	BORING DATE Nov. 10, 1970	COMPILED BY SAA
DATUM Geodetic	BOREHOLE TYPE Washboring and NX Casing	CHECKED BY

(m)	ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
220.6	723.6	Ground Level					
	0.0	Clayey silt with trace of sand & gravel. Stiff to Very Stiff	1	SS	11		
218.0	715.1	Brown	2	SS	19		0 10 85 5
2.6	8.5	Grey	3	TW	PH	132	
			4	SS	28		
			5	SS	25		
			6	SS	25		
			7	TW	PH	129	
210.8	691.6		8	SS	12		
9.8	32.0	Het. mix. of silt, sand & gravel, trace of clay. Glacial Till	9	SS	85		
		Very Dense	10	SS	100/4"		0 37 53 [illegible]
206.7	678.1	Grey	11	SS	100/4"		
13.9	45.5	End of Borehole					



20
15 ⊕ 3 % STRAIN AT FAILURE
10

FORM OB-MT-126 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

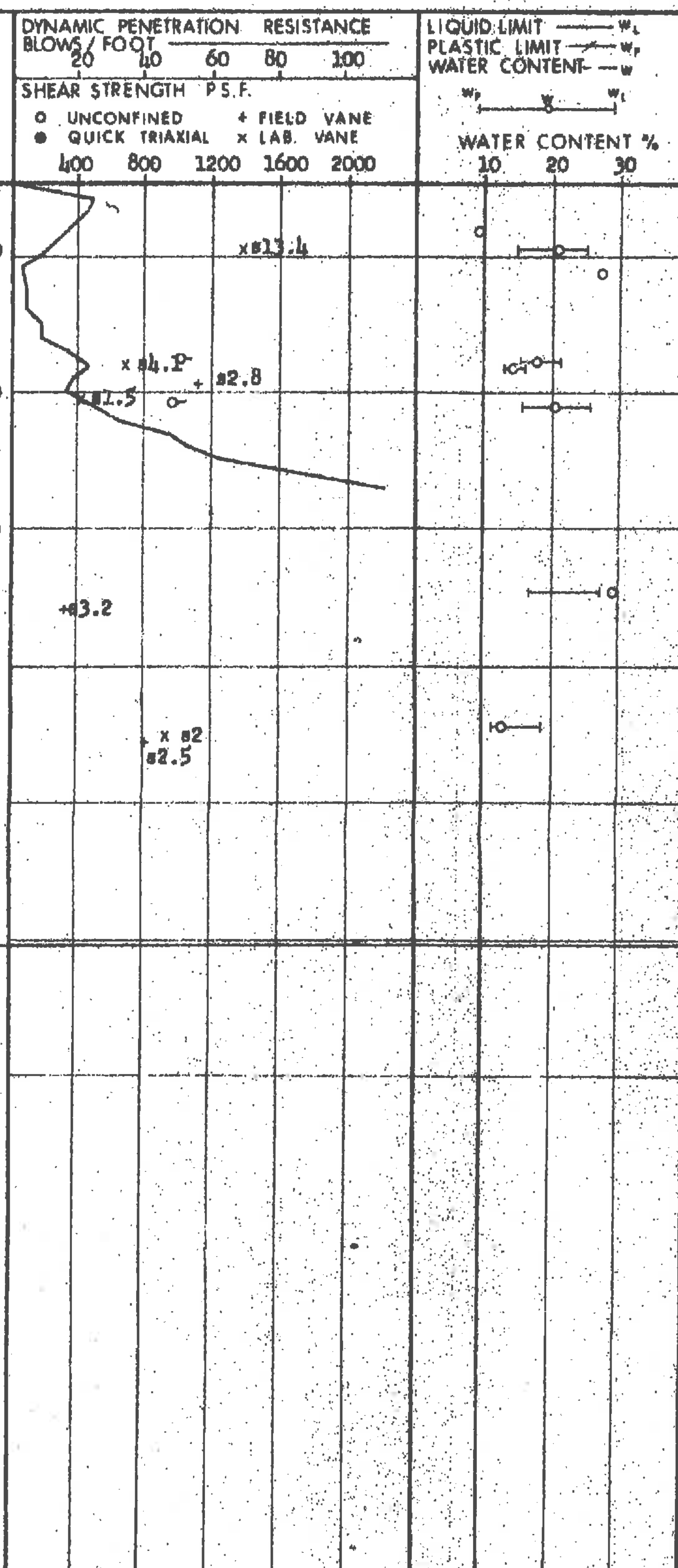
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4 BH 29-4

FOUNDATION SECTION

JOB 70-11089	LOCATION Sta. 588 + 26 o/s 54' Lt.	ORIGINATED BY	VK
W.P. 105-70-64	BORING DATE Oct. 19/70	COMPILED BY	SAA
DATUM Geodetic	BOREHOLE TYPE Cont. Flight Auger	CHECKED BY	[signature]

	ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
221.1	725.3	Ground Level						
	0.0	Sandy silt, trace of of ogranics. Loose. Brown	1	SS	6			8 34 46 12
219.9 / 1.2	721.3 / 4.0	Clayey silt with trace of sand & gravel and some organics	1A	TW	PH	720		0 6 [illegible]
			2	SS	5			Nov. 2.9 17/70 Org.
			2A	TW	PH			0.6% org.
			3	SS	5			
			4	TW	PH		137.5	0 66 (34)
		Firm to Very Stiff	5	TW	PH	710	130	
		Grey	6	SS	27			
			7	SS	26	700		
			8	SS	7			
			9	TW	PM	690		
			10	TW	PM		140	
208.0 / 13.1	682.3 / 43.0	Het. mix. of silt, sand & gravel, trace of clay Glacial Till	11	TW	PH	680		
		Very Dense	12	SS	174			6 29 51 14
204.2 / 17.0	669.8 / 55.5	Grey	13	SS	147	670		
		End of Borehole				660		



20
15 -o- 5 % STRAIN AT FAILURE
10

FORM OB-MT-[illegible]26 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

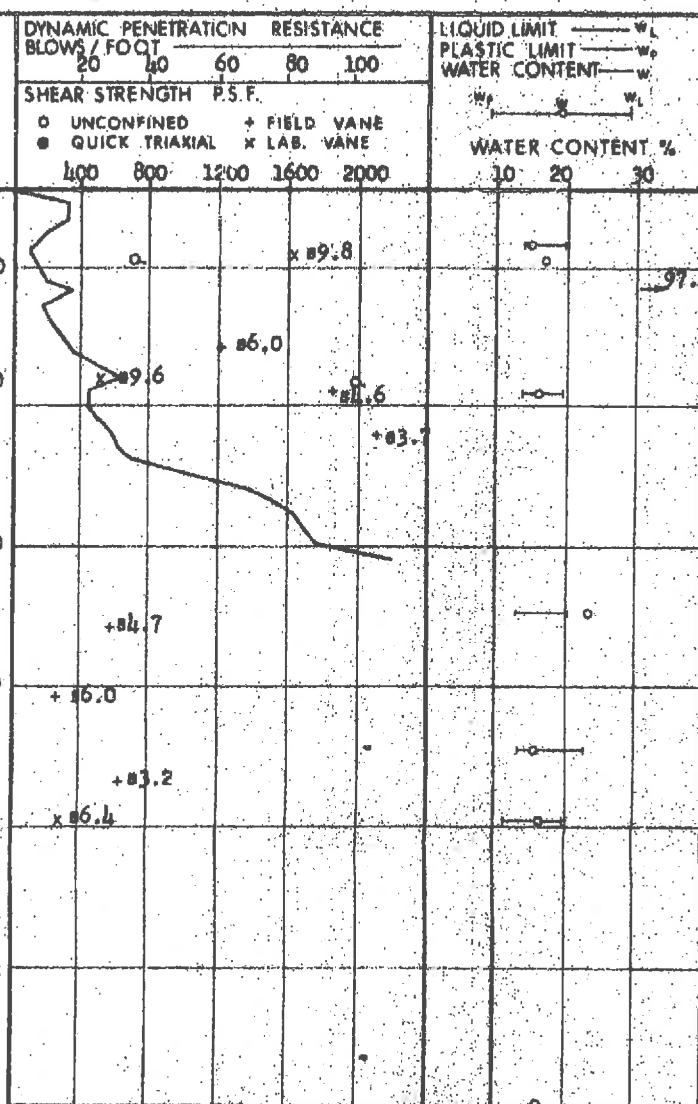
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5 BH 29-5

FOUNDATION SECTION

JOB 70-11C89 LOCATION Sta. 589 + 08 o/s 57' Rt. ORIGINATED BY VK
W.P. 105-70-04 BORING DATE Oct. 20/70 COMPILED BY SAA
DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY [illegible]

(M)	ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.	WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
221.1	725.3	Ground Level								
	0.0	Clayey silt with trace of sand & gravel & organics (Fill) Firm Brown	1	SS	4		x s9.8		109	0 25 65 10
			1A	TW	PM	720				2.2% Org.
219.2 / 1.8	719.3 / 6.0	Organic Clay Black Soft	2	SS	4			97.		47.1% Org.
			2A	TW	PM					
218.3 / 2.7	716.3 / 9.0	Clayey silt with trace of sand & gravel Soft to Very Stiff Grey	3	SS	7		+ s6.0			
			4	TW	PH	710	x s9.6; + s1.6		138	0 16 77 7
			5	SS	7		+ s3.7			
			6	SS	23					
			7	SS	17	700				
			8	TW	PM		+ s4.7			
			9	TW	PM	690	+ s6.0			No recovery
			10	TW	PM					No recovery
			10A	SS	8		+ s3.2			
			11	TW	PH	680	x s6.4			
206.1 / 14.9	676.3 / 49.0	Het. mix. of silt, sand & gravel, trace of clay Glacial Till Dense to Very Dense Grey	12	SS	34					
			13	SS	77	670				
			14	SS	159					
201.1 / 20.0	659.8 / 65.5	End of Borehole	15	SS	162	660				
						650				



20
15 -o- 5 % STRAIN AT FAILURE
10

FORM OB-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

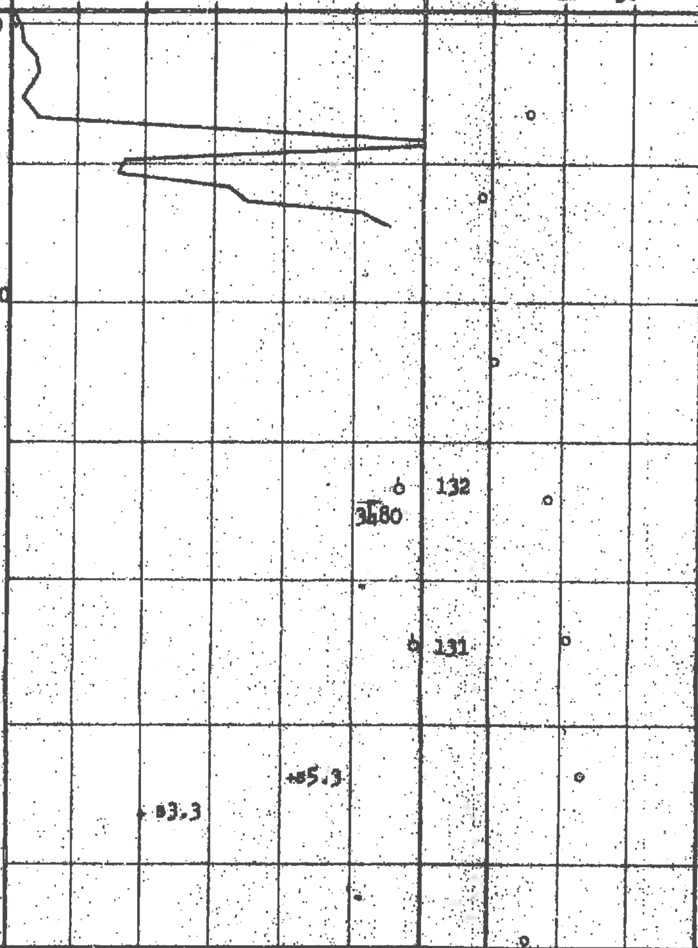
RECORD OF BOREHOLE No. 6 BH 29-6

FOUNDATION SECTION

JOB 70-11089	LOCATION Sta. 589 + 15 o/s 54.5' Lt.	ORIGINATED BY VK	
W.P. 105-70-04	BORING DATE Nov. 19/70	COMPILED BY SAA	
DATUM Geodetic	BOREHOLE TYPE Washboring-NX Casing	CHECKED BY [illegible]	

DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 400 800 1200 1600 2000
LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w — WATER CONTENT % 10 20 30
BULK DENSITY γ P.C.F.
REMARKS GR.SA.SI.CL

(m)	ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE
225.8	740.9	Ground Level				
	0.0	Clayey silt, trace of sand & gravel Stiff to Hard Brownish Grey (Embankment Fill)				740
			1	SS	21	
			2	SS	12	
222.5 / 3.4	729.9 / 11.0	Silty sand, trace of gravel. Very Dense	3	SS	80	730
221.6 / 4.3	726.9 / 14.0	Clayey silt, trace of sand and gravel Stiff to Hard Brownish grey to grey	4	SS	66	
			5	SS	77	
			6	SS	144	720
			7	SS	25	
217.0	711.9	Sandy silt to silt.				
216.5 / 9.3	710.4 / 30.5	Clayey silt	8	SS	13	710
			9	TW	PM	
			10	TW	PM	700
			11	TW	PM	
			12	TW	PM	690
			13	SS	12	
206.9 / 18.9	678.9 / 62.0	Het. mix. of silt, sand & gravel - Glacial Till Very Dense. Grey				680
205.6	674.4		14	SS	61	
20.3	66.5	End of Borehole				670



Bulk density: 132 (sample 9)

Remarks: ▼ 728.9 in open NX Nov. 20/70

20
15 -◇- 5 % STRAIN AT FAILURE
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FORM 08-MT-126 (REV 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 8 BH 29-8

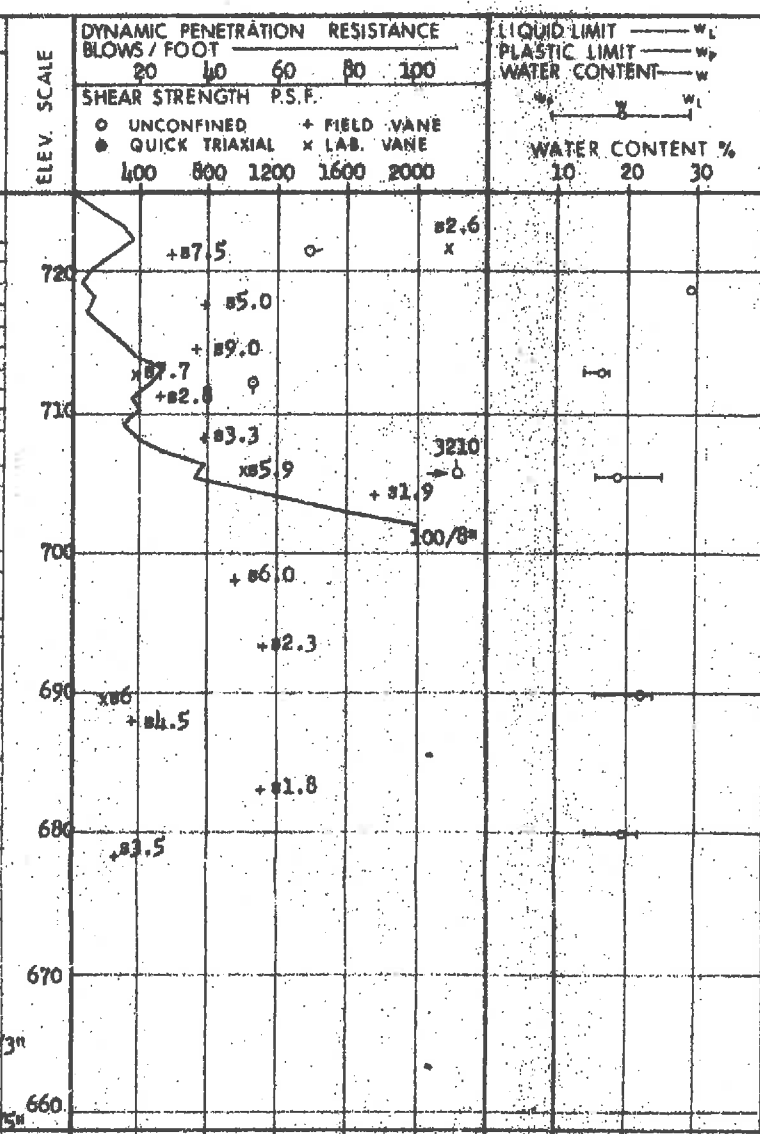
FOUNDATION SECTION

JOB 70-11089 LOCATION Sta. 588 + 46 o/s 2' Lt. ORIGINATED BY VK

W.P. 105-70-04 BORING DATE Oct. 21/70 COMPILED BY SAA

DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY

(m.)	ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
221.1	725.5	Ground Level						
	0.0	Sandy silt, trace of organics						Nov.17/70 ▼720.8
		Loose to compact		1	SS	11		
218.4	716.5	Brown		2	SS	7		0 21 69 10
2.7	9.0	Clayey silt with trace of sand & gravel		3	SS	8		0 1 87 12
				4	TW	PH	134	
		Soft to Stiff		5	TW	PH		
				6	TW	PH	133	
		Grey		7	SS	27		
				8	TW	PH		
				9	TW	PH		
				10	TW	PH		
206.2	676.5			11	TW	PH		
14.9	49.0	Het.mix.of silt,sand & gravel, trace of clay - Glacial Till		12	SS	145		
				13	SS	30		
		Dense to Very Dense		14	SS	100/3"		
200.9	659.0	Grey		15	SS	100/5"		
20.3	66.5	End of Borehole						



20
15 ◇ 5 % STRAIN AT FAILURE
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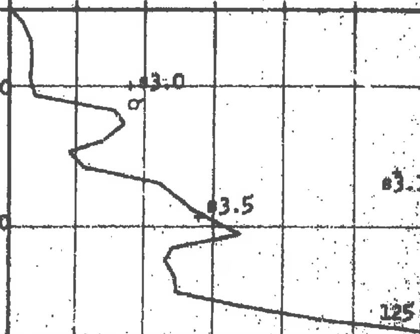
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 9 BH 29-9

FOUNDATION SECTION

JOB 70-11089	LOCATION 587 + 08 o/s 111' Rt.		ORIGINATED BY VK
W.P. 105-70-04	BORING DATE Nov. 18, 1970		COMPILED BY SAA
DATUM Geodetic	BOREHOLE TYPE Washboring-NX Casing		CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 400 800 1200 1600 2000	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 10 20 30	BULK DENSITY γ P.C.F.	REMARKS GR SA SI CL
(m) ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
221.1 725.4	Ground Level									
0.0	Clayey silt with trace of sand & gravel and organic		1	TW	PM					721.4
	Firm to very stiff		2	TW	PM	720	+3.0	o	121	in open BH Nov.19/70
	Brown - Grey		3	SS	17					
			4	SS	21		+3.5	+3.1		
			5	SS	19	710		o		
214.1 702.4			6	SS	28		125			
40 23.0	End of Borehole					700				



20
15 — 5 % STRAIN AT FAILURE
10

APPENDIX D

Non-Standard Special Provisions

OPERATIONAL CONSTAINT – Peat/Organic Deposit Excavation

Special Provision

This special provision outlines the procedure to be used for excavation of the peat/organic deposits along the following areas; the depth/elevation of subexcavation in these areas is shown on the Contract Drawings.

- From the north abutment of the South Canal bridges to Station 25+140 on the east side of Highway 400, adjacent to Wist Road.
- From the north abutment of the South Canal bridges to Station 25+110 on the west side of Highway 400, adjacent to Davis Road.

Staged excavation in strips of limited width shall be carried out to maintain the stability of the excavation and protection system along Highway 400, and to protect the existing Wist Road or Davis Road during the subexcavation and backfilling operations. The staged excavations procedures are outlined as follows:

a) The work may be carried out simultaneously from both ends of the area to be subexcavated, working towards the centre.

b) Removal of the peat/organic soils and overlying fill materials within the embankment widening or RSS wall footprint shall be carried out in short “strip” sections perpendicular to the Highway 400 and local road alignments, with the base of the excavation (as measured parallel to Highway 400 and the local road) not wider than 3 m.

c) Temporary excavation side slopes or back slopes through the peat/organic soils and overlying fill materials shall be no steeper than 1 horizontal to 1 vertical (1H:1V) adjacent to the existing local roads (Wist Road or Davis Road).

d) Excavation and backfilling operations shall be carried out simultaneously in a manner that the excavation is not left open for more than the 3 m “strip” width at any given time.

The Contractor shall maintain the operation of the Highway 400, Wist Road and Davis Road during excavation and backfilling operations including and not limited to traffic control.

Payment for the Contractor to provide the above requirements, including all equipment, labour and materials shall be deemed to be included in the contract bid price for the various tender items.

<u>DEWATERING STRUCTURE EXCAVATION</u> – Item No.

Special Provision

Amendment to OPSS 902

This amendment applies for dewatering of the foundation excavations for the replacement of the South Canal bridges.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.02 Submission Requirements

Section 902.04.02 is amended by the addition of the following Subsection:

902.04.02.03 Dewatering

At least two weeks prior to commencing dewatering operations, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

902.07 CONSTRUCTION

902.07.04 Dewatering Structure Excavation

Section 902.07.04 is amended by the addition of the following:

The dewatering system shall be adequate to lower the groundwater level to at least 0.3 m below the founding level for the new abutments and centre pier, to allow excavation, subgrade preparation and foundation construction in dry conditions.

The Contractor shall reference borehole records as shown elsewhere in the Contract Documents as a guide in determining dewatering requirements.

A continuous dewatering operation shall be provided to facilitate the foundation construction operations at all times. All components of the dewatering system shall be maintained in an effective, functioning and stable condition during the construction.

The work for dewatering shall be completed in accordance with the environmental and operational constraints specified elsewhere in the Contract Documents.

The Contractor is advised that construction of the new abutment and pier foundations at the South Canal bridges site will require excavation to near or below the groundwater level in the fine-grained, silty sand to silt deposit, which is generally present below existing fill materials and above the clayey silt deposit. Cohesionless soils below the groundwater table will be subjected to conditions of unbalanced hydrostatic head and can slough, boil and cave in during temporary excavation work.

GRANULAR FILTER BLANKET - Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR GRANULAR FILTER BLANKET

1.0 SCOPE

This Special Provision covers the requirements for the supply and placement of a granular filter blanket below the new pile caps at the South Canal bridges, to mitigate the potential for upward migration of fine soil particles along the pile shafts as a result of the artesian groundwater pressures at this site.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specification, General:

OPSS 102 Weighing of Materials

Ontario Provincial Standard Specification, Material:

OPSS 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

3.0 DEFINITIONS – not used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – not used

5.0 MATERIALS

The granular filter blanket shall consist of Granular "A" and conform to the gradation requirements of OPSS 1010.

6.0 EQUIPMENT - not used

7.0 CONSTRUCTION

07.01 General

The granular filter blanket shall be constructed below the abutment and pier pile caps following excavation to the appropriate subgrade level and prior to commencement of pile driving operations. The granular filter blanket shall remain in place throughout the pile driving operations and in the permanent structure condition.

The granular filter blanket shall consist of a minimum 0.5 m thick layer of Granular "A". The granular filter blanket shall extend a minimum of 1 m horizontally beyond the location of piles at each foundation element. The Contractor shall construct the granular filter blanket as shown elsewhere on the Contract Drawings.

8.0 QUALITY ASSURANCE - not used

9.0 MEASUREMENT FOR PAYMENT

Measurement of Granular Filter Blanket shall be by mass in tonnes according to the requirements of the Contract Documents.

10.0 BASIS OF PAYMENT

10.01 Granular Filter Blanket - Item

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

VIBRATION MONITORING - Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR VIBRATION MONITORING

1.0 SCOPE

This special provision describes requirements for vibration monitoring during pile installation and sheet-pile/protection system installation and/or removal for the South Canal bridges.

2.0 REFERENCES

The subsurface conditions at the structure sites are described in the relevant Foundation Investigation Reports for GWP 2835-02-00:

Foundation Investigation Report – South Canal Bridges Highway 400 Widening from North of King Road to North of South Canal Road, Regional Municipality of York

3.0 DEFINITIONS

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the Contract. The QVE shall be retained by the Contractor to ensure general conformance with the Contract Documents and issue certificates of conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

The Contractor shall submit details of the vibration monitoring plan to the Contract Administrator for information. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Equipment and methods used by the Contractor to perform the work that may cause vibration.
- Qualifications of vibration monitoring specialist.
- Details regarding proposed vibration monitoring instrumentation.
- Proposed location of instruments on the existing Highway 9 underpass and South Canal bridges.
- Proposed frequency of readings.
- Action plan to be taken to adjust deep foundation or protection system installation methods if readings show vibrations exceeding tolerable levels.

5.0 MATERIALS - Not Used

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Monitoring

The vibration monitoring equipment shall be placed on the existing South Canal bridge structures, for all stages in which a portion of the existing structure remains in service during construction of the adjacent replacement structure. The Contractor shall take readings on the existing structures throughout pile driving or protection system installation operations, as applicable at these sites, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on the existing bridge structures shall not exceed 100 mm/s (peak particle velocity).

If the readings are not within the limits stated above, the Contractor shall alter the deep foundation or protection system installation procedures until the vibrations at the existing bridge structures are within acceptable levels.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT - Not Used

10.0 BASIS OF PAYMENT

10.01 Vibration Monitoring - Item

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

DEEP FOUNDATIONS - Item No.

Non-Standard Special Provision

Amendment to OPSS 903

903.07 CONSTRUCTION

903.07.02 Driven Piles

903.07.02.01 Pile Driving Requirements and Restrictions

Section 903.07.02.01 is amended by the addition of the following:

The Contractor is advised that the soils at the South Canal bridge sites are glacially derived and, as such, should be expected to contain cobbles and boulders even where no such obstructions are specifically noted on the borehole records. Care shall be taken to avoid overdriving and damage to piles associated with the presence of cobbles and/or boulders.

903.07.02.07 Monitoring Driven Piles

903.07.02.07.04 Wave Equation Analysis

Section 903.07.02.07.04 is amended by the addition of the following:

The Contractor shall complete pile dynamic analyzer (PDA) testing on a minimum of 10 per cent of piles at each foundation element for each of the stages listed below, in conjunction with re-tapping of piles in accordance with Section 903.07.02.07.06. The piles to be re-tapped and subjected to PDA testing shall be selected by the Contractor based on their piling operation works and schedule, subject to agreement by the Contract Administrator.

- Stage 2 New Construction – NBL
- Stage 2 New Construction – SBL
- Stage 3 New Construction – NBL
- Stage 3 New Construction - SBL

<u>OPERATIONAL CONSTRAINT</u> - Preload Period – Embankment Widening Construction

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Special Provision

The Contractor shall schedule his operation to include the following preloading times for the eastward and westward widening of the embankments on Highway 400 in the vicinity of the South Canal bridges. To allow time for the settlement of the embankment widening and/or two-stage retained soil system (RSS) wall, the following time constraints shall apply:

- For the south approach embankments, extending from the south abutment to 20 m south of that abutment, the embankment widening shall be constructed up to the top of the granular sub-base material, and the fills shall remain in place for a minimum period of six (6) weeks before paving.

- North of the South Canal bridges, extending from the north limit of the EPS behind the north abutment, to Station 25+120, the embankment widening or two-stage RSS wall construction shall be constructed up to the top of the granular sub-base material, and the fills shall remain in place for a minimum period of eight (8) months before paving and before installation of the permanent facing panels on the RSS wall.

- From Station 25+120 to the north limit of the contract, the embankment widening shall be constructed up to the top of the granular sub-base material, and the fills shall remain in place for a minimum period of six (6) months before paving.

Prior to placement of the Granular A base material and paving, the Contractor shall conduct a survey to determine the elevations of the top of the Granular B sub-base material, and shall place additional Granular B Type II material as and where required to achieve the pavement design sub-base elevation.

The Contractor shall not proceed with final granular placement and paving until approval has been given by the Contract Administrator.

RIGID EXPANDED POLYSTYRENE EMBANKMENT FILL

Special Provision

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene (EPS) embankment fill and associated works as shown on the Contract Drawings.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

National Standards of Canada

CAN/ULC – S102.2-10
CAN/ULC – S701-11

NCHRP

Report 529	Guideline and Recommended Standard for Geofoam Applications in Highway Embankments

ASTM

ASTM D1621	Test Method for Compressive Properties of Rigid Cellular Plastics
ASTM C203	Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation
ASTM C177	Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus
ASTM D2842	Test Method for Water Absorption by Rigid Cellular Plastics
ASTM D2863	Test Method for Measuring the Minimum Oxygen Content
ASTM D2126	Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging
ASTM D6817	Standard Specification for Rigid Cellular Polystyrene Geofoam

OPSS - Ontario Provincial Standard Specification

OPSS 212	Borrow
OPSS 501	Compaction
OPSS 517	Dewatering
OPSS 1010	Aggregates – Granular A, B, M, and Selected Subgrade Material
OPSS 1860	Geotextiles

Subsurface Conditions

The subsurface conditions at the site are described in the Foundation Investigation Report for South Canal Bridges.

3.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene: Molded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum-based raw material.

Rigid Extruded Expanded Polystyrene: Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot: The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: An Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

4.2 Delivery, Storage, Handling and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturer's requirement.

4.3 Construction

The contractor shall submit full details of the following:

a. The method of foundation excavation and preparation.
b. Construction of levelling pad.
c. The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer by layer basis.
d. The method and limits of placement of polyethylene geomembrane and/or sheeting.
e. The method of placement of 125 mm reinforced concrete slab (or equivalent).
f. The method of placement of subbase material.
g. The method of placement of side slope cover.

5.0 MATERIALS

5.1 Granular Levelling Pad

The levelling pad shall consist of a Granular "A" or Granular "B" material with gradation and physical requirements as specified in OPSS 1010.

5.2 Polyethylene Geomembrane

The EPS shall be encapsulated with a polyethylene geomembrane with two coextruded textured surfaces such as [GSE HDT 30A000] or approved equivalent. A minimum 10mil smooth [black] polyethylene sheeting may be used on the sides only.

Polyethylene geomembrane shall be flexible and, by its own weight, shall cover and conform closely to 90 degree edges and corners of EPS blocks without additional heating of the polyethylene geomembrane.

Polyethylene geomembrane shall be free from pin holes, tears, and any defects.

5.3 Rigid Expanded Polystyrene

5.3.1 General

The Contractor shall submit the following:

a. A general statement as to the type, composition, and method of production of the material.

b. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.

c. Certification of compliance of physical and mechanical properties.

d. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.

e. The physical and mechanical properties of the rigid expanded polystyrene including the following:
    - Geometry
    - Nominal Density
    - Compressive Strength
    - Flexural Strength
    - Thermal Resistance
    - Dimensional Stability
    - Flammability
    - Water Absorption

f. Aging and durability characteristics of the polystyrene including the chemical, biological and ultraviolet degradation resistance of the rigid polystyrene.

g. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.

h. To the Contract Administrator, a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

EPS blocks shall meet ASTM D6817 Standard Specification for rigid cellular polystyrene geofoam

5.3.2 Detail Requirements

Requirements shall be as shown in Table 1 and as described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	mm	1200 × 600 × 300 with tolerances ± 1% 10mm in 3m ± 0.5% -3, +5	
Compressive Strength	kPa (min.) @ 1% Deformation	50 (EPS Type 22) 65 (EPS Type 24) 75 (EPS Type 29)	ASTM D1621 (Procedure A)
Compressive Strength	kPa (min.) @ 5% Deformation	110 (EPS Type 22) 140 (EPS Type 24) 170 (EPS Type 29)	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min.)	240 (EPS Type 22) 276 (EPS Type 24) 345 (EPS Type 29)	ASTM C203 (Procedure B)
Dimensional Stability	% linear change (max.)	1.5	ASTM D2126
Thermal Resistance	m^2.°C/W (min. for 25mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min.)	24	ASTM D2863
Water Absorption	% by Volume (max.)	4 (EPS Type 22) 3 (EPS Type 24) 2 (EPS Type 29)	ASTM D2842

5.3.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm.

The maximum deviation from the specified linear dimensions shall be ± 1%. The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner to corner dimensions (squareness) shall be 0.5%. The thickness shall be within –3 to +5 mm.

5.3.2.2 Compressive Strength

At no time shall the vertical stress on the EPS exceed the 1% strain limit. A Factor of Safety of 1.2 shall be applied to all loads, in accordance with NCHRP 529.

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 110 kPa for EPS Type 22 at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain. The compressive strength for other grades of EPS are listed in Table 1.

5.3.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa for EPS Type 22. The flexural strength shall be determined in accordance with ASTM C203, method 1, Procedure B. The flexural strength for other grades of EPS are listed in Table 1.

5.3.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

5.3.2.5 Thermal Resistance

The thermal resistance shall be 0.7 m^2. °C /W for a 25mm thickness using the following equation and using the average value from three specimens:

$$R_{25mm} = \frac{R_{measured}}{Thickness(mm)} \times 25$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

5.3.2.6 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - S102.2-10 having a flame spread rating less then 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863.

5.3.2.7 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% for EPS Type 22 by volume. The water absorption for other grades of EPS are listed in Table 1.

5.3.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalis. A table identifying the chemical resistance as resistant, limited or not resistant shall be submitted.

5.3.2.9 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

5.3.2.10 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

6.0 EQUIPMENT

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

7.0 CONSTRUCTION

7.1 Qualification

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

7.2 Delivery, Storage and Handling

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

EPS blocks shall be stored for a minimum 72 hours at ambient room temperature (20 to 25 degrees Celsius) after an EPS block is released from the mould.

EPS blocks shall be stored above ground. EPS blocks shall be protected from moisture and sunlight in accordance with manufacturer's recommendations.

EPS shall not be exposed to open flame or other ignition source. The constructor shall protect the EPS blocks from petroleum based products such as gasoline and diesel fuel and organic solvents such as acetone, benzene and paint thinner.

7.3 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

Any unsuitable area as determined by the Engineer, shall be excavated and replaced with suitable compacted backfill. The native soil foundation and/or embankment subgrade shall be free from deleterious, loose, or otherwise unsuitable soils.

7.4 Levelling Pad

Clear and grub site and remove any subgrade material unsuitable for EPS block placement as determined by the Engineer.

Dewater as required. There shall be no standing water or accumulated snow or ice on the subgrade within the area where the EPS blocks are placed. EPS blocks shall not be placed on a frozen subgrade.

Place, level and compact to 95% standard Proctor density, a 150mm layer of Granular 'A' or Granular 'B' material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The levelling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The levelling pad shall not be placed on frozen ground.

EPS shall not be founded directly on existing asphalt pavement. The constructor shall remove existing pavement in addition to any material containing hydrocarbons and replace with clean granular material. Where an EPS embankment is founded above a pre-existing subsurface pavement layer there shall be 200mm (min.) of free draining levelling course below EPS blocks.

7.5 Installation of Blocks

1. The individually marked blocks shall be placed on the prepared levelling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.

2. Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90º to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers. A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.

3. Sloping end adjustments at the abutments shall be accomplished by levelling terraces in the subsoil in accordance with the block thickness.

4. Top surface of EPS blocks shall be stepped (or cut on a slope) to match [superelevation or crossfall] and/or grading.

5. Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.

6. The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.

7. The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.

8. Exposed blocks shall be covered immediately to avoid possible burrowing by animals.

9. Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.

10. The top surface and side surfaces of the expanded polystyrene shall be covered with a polyethylene geomembrane and/or 10 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

11. The contractor shall install the concrete base pad as detailed in the Contract Drawings.

12. No construction equipment shall be permitted to drive directly on the polyethylene geomembrane. Damage to the geomembrane resulting from construction activities, equipment, or operations shall be repaired by the constructor as per manufacturer's recommendations.

13. The side slope of the rigid expanded polystyrene embankment shall be covered with clean sand salvaged from earth excavation operations. Alternatively, Granular B Type 1 to be used.

14. The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments the Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

15. The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Embankment the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.

7.8 Construction Loading

The pavement system includes all material placed above the EPS blocks within the limits of the roadway (including shoulders) unless note otherwise. The pavement system shall be constructed above the EPS blocks as shown on the design drawings.

No vehicles or construction equipment shall drive directly on the EPS blocks or the geomembrane. Bedding sand and crushed base course for the pavement system shall be pushed onto the embankment using appropriate light weight equipment. A nominal thickness of 300mm granular material shall cover the compact the lower layers. Vibratory equipment (e.g. vibrating drum roller) shall not be used to compact the first 300mm of granular material.

The bearing pressures at the surface of the EPS blocks due to construction loads (including any granular material) shall not exceed the 1% deformation compressive resistance values listed in Table 1. A factor of 1.2 shall be applied to all construction loads.

For the purposes of calculating EPS bearing pressures due to construction loads, use a load dispersion angle of 1H:1V through compacted material and 1H:2V through uncompacted material.

8.0 QUALITY ASSURANCE

8.1 General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

8.2 Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three blocks shall be tested.

For each EPS grade produced by the block supplier, a minimum of one sample shall be tested per 500 m^3 for the first 2000 m^3. A minimum of one sample per 2000 m^3 shall be tested thereafter.

8.3 Acceptance / Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor’s expense.

9.0 MEASUREMENT FOR PAYMENT

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

10.0 BASIS OF PAYMENT

The concrete base pad and granular levelling pad shall be paid for with the appropriate tender items as detailed elsewhere in the Contract Documents.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

LIGHTWEIGHT MATERIAL

Non Standard Special Provision

1.0 SCOPE

This special provision covers the requirements for the supply and placement of lightweight blast furnace slag for the westward widening of the Highway 400 embankment north of the South Canal bridges.

2.0 REFERENCES

ASTM

ASTM D422-63	Standard Test Method for Particle-Size Analysis of Soils
ASTM D2216	Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D2850-95	Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test
ASTM D5856-95	Standard Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction Mold Permeameter
ASTM D6938-10	Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods

OPSS – Ontario Provincial Standard Specifications

OPSS 102	General Specification for Weighing of Materials
OPSS 206	Construction Specification for Grading

3.0 DEFINITIONS

Quality Verification Engineer: means an Engineer with a minimum of five (5) years experience related to embankment materials and construction, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and issue of certificate(s)of conformance.

4.0 SUBMISSION AND DESIGN REQUIREMENTS

The Contractor shall submit to the Contract Administrator Certificates of Conformance sealed and signed by the Quality Verification Engineer as follows:

- Prior to the placement of the lightweight fill material on the Contract, the Contractor shall submit to the Contract Administrator a Certificate of Conformance stating that the material satisfies the material properties specified in Table 1. The material properties shall be determined using the test procedures specified in Table 1.

- Following embankment construction, the Contractor shall submit to the Contract Administrator a Certificate of Conformance stating that the material satisfies the requirements of this specification and that the work has been carried out in general conformance with the contract documents and specifications.

In addition, the Contractor shall submit to the Contract Administrator, for information only, all Quality Control Test Results.

5.0 MATERIALS

The Lightweight Blast Furnace Slag shall satisfy the physical, mechanical and chemical property requirements specified in Table 1:

Table 1: Material Properties and Construction Requirements

Property	Requirement	Test Method
Angle of Internal Friction	> 35 °	ASTM D2850-95
Hydraulic Conductivity	> 8 E-03 cm/s	ASTM D5856-95, Method A
Chemical Composition	The material shall meet the Leachate Criteria established under Ontario Regulation 347.	
In-Situ Wet Unit Weight, maximum when placed and compacted in accordance with the requirements of this Special Provision	< 14.5 kN/m^3	ASTM D6938-10

The Contractor shall retain a laboratory that has been inspected and accepted by the MTO under the "Soil and Rock - High Complexity Testing" Specialty to undertake the testing of the material properties. Laboratory testing shall be signed and sealed by an Engineer, licensed to practice in the Province of Ontario

6.0 EQUIPMENT

Compaction equipment technical details are provided in Table 2.

Table 2 – Compaction Equipment Technical Details

	Bomag 142 D	Bomag BPR 30/38 D
Weights		
Operating weight (kg)	4690±	175±
Mass per square metre of base plate (kg/m^2)	N/A	1439
Dimensions		
Drum width (mm)	1426±	N/A
Drum diameter (mm)	1058±	N/A
Width of Base Plate (mm)	N/A	380
Length of Base Plate (mm)	N/A	730
Drive		
Performance DIN 6271 IFN (kW)	37±	3.7
Performance SAE (Kw)	39.5	N/A
Speed (rpm)	2300	3600
Vibratory System		
Frequency (Hz)	32±	68±

Amplitude (mm)	1.24±	N/A
Centrifugal force (Kn)	66±	30±

7.0 CONSTRUCTION

The Contractor is advised that the lightweight blast furnace slag is susceptible to crushing if overcompacted, and that careful construction supervision is required.

The Contractor shall place the lightweight fill material and shall achieve compaction without crushing the material, as crushing increases its unit weight.

The Contractor shall place the lightweight fill material without exceeding the specified in-situ unit weight, and while maintaining crushing of the material below 5%.

To prevent overcrushing and overcompaction, the lightweight fill shall be placed in accordance with OPSS 206.07 with the following amendments:

- For embankments, the lightweight fill shall be placed in lifts of 300 mm and compacted by three (3) passes using single drum vibratory equipment such as a Bomag 142 or equivalent.

- For backfill to structures, the lightweight fill shall be placed in lifts of 300 mm and compacted with eight (8) passes of manually guided tamper such as a Bomag BPR 30/38 D or equivalent.

- The Contractor shall place and spread the loose lifts using a rubber tire front-end loader such as a Caterpillar 980 F or equivalent.

8.0 QUALITY CONTROL

8.1 General

Quality Control (QC) testing shall be carried out by the Contractor for purposes of ensuring that the lightweight fill material is placed and compacted to the requirements specified in the Contract. Field density and field moisture determination shall be made in accordance with ASTM D6938-10.

Acceptability of compaction shall be based on achieving the target in situ unit weight.

8.2 Control Strip

Under the Supervision of the Quality Verification Engineer, the Contractor shall build a control strip to verify that the placement and compaction procedure will achieve the requirements of this Special Provision without evidence of crushing and without exceeding the specified maximum in-situ unit weight of 14.5 kN/m^3.

Prior to incorporating any of the material into the work, the Contractor shall build a minimum trial area of 100 m^2 (approximately 5 m x 20 m) in area consisting of two equal lifts of 300 mm thickness. The Contractor shall give the Contract Administrator written notice of the construction of the control strip 48 hours prior to commencement of this work.

Material placed in the control strip shall have the moisture content that will yield the specified in-situ unit weight. For the control strip determination, the nuclear gauge method will not be considered an acceptable method of determining the in-situ moisture content of the lightweight material. Moisture content shall be determined by the oven dry method on selected compacted embankment material samples in accordance with ASTM D2216.

After the trial area is complete, samples for moisture content and in-situ unit weight determination testing shall be as per ASTM D6938-10.

In addition, gradation as per ASTM D422-63 before and after compaction effort shall be performed to determine that crushing is kept within 5%.

All test results will be used to determine compliance with the specification. Any proposed changes to the specified compaction method shall be reviewed and approved by the Contract Administrator prior to implementation. The requirements of the control strip must be satisfied as part of the acceptance criteria of any proposed change to the specified compaction method of this Special Provision.

9.0 MEASUREMENT OF PAYMENT

The unit measurement will be tonnes and the method of determining the weight of material for payment shall conform to OPSS 102.

10.0 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.

ULTRA LIGHTWEIGHT MATERIAL

Non Standard Special Provision

1.0 SCOPE

This special provision covers the requirements for the supply and placement of ultra-lightweight blast furnace slag for the eastward widening of the Highway 400 embankment north of the South Canal bridges, including the construction of the reinforced soil system (RSS) wall.

2.0 REFERENCES

ASTM

ASTM D422-63	Standard Test Method for Particle-Size Analysis of Soils
ASTM D2216	Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D2850-95	Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test
ASTM D5856-95	Standard Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction Mold Permeameter
ASTM D6938-10	Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods

OPSS – Ontario Provincial Standard Specifications

OPSS 102	General Specification for Weighing of Materials
OPSS 206	Construction Specification for Grading

3.0 DEFINITIONS

Quality Verification Engineer: means an Engineer with a minimum of five (5) years experience related to embankment materials and construction, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and issue of certificate(s)of conformance.

4.0 SUBMISSION AND DESIGN REQUIREMENTS

The Contractor shall submit to the Contract Administrator Certificates of Conformance sealed and signed by the Quality Verification Engineer as follows:

- Prior to the placement of the ultra lightweight fill material on the Contract, the Contractor shall submit to the Contract Administrator a Certificate of Conformance stating that the material satisfies the material properties specified in Table 1. The material properties shall be determined using the test procedures specified in Table 1.

- Following embankment construction, the Contractor shall submit to the Contract Administrator a Certificate of Conformance stating that the material satisfies the requirements of this specification and that the work has been carried out in general conformance with the contract documents and specifications.

In addition, the Contractor shall submit to the Contract Administrator, for information only, all Quality Control Test Results.

5.0 MATERIAL

The Ultra-Lightweight Blast Furnace Slag shall satisfy the physical, mechanical and chemical property requirements specified in Table 1:

Table 1: Material Properties and Construction Requirements

Property	Requirement	Test Method
Angle of Internal Friction	> 35 °	ASTM D2850-95
Hydraulic Conductivity	> 8 E-03 cm/s	ASTM D5856-95, Method A
Chemical Composition	The material shall meet the Leachate Criteria established under Ontario Regulation 347.	
In-Situ Wet Unit Weight, maximum when placed and compacted in accordance with the requirements of this Special Provision	< 12.5 kN/m^3	ASTM D6938-10

The Contractor shall retain a laboratory that has been inspected and accepted by the MTO under the "Soil and Rock - High Complexity Testing" Specialty to undertake the testing of the material properties. Laboratory testing shall be signed and sealed by an Engineer, licensed to practice in the Province of Ontario.

6.0 EQUIPMENT

Compaction equipment technical details are provided in Table 2.

Table 2 – Compaction Equipment Technical Details

	Bomag 142 D	Bomag BPR 30/38 D
Weights		
Operating weight (kg)	4690±	175±
Mass per square metre of base plate (kg/m^2)	N/A	1439
Dimensions		
Drum width (mm)	1426±	N/A
Drum diameter (mm)	1058±	N/A
Width of Base Plate (mm)	N/A	380
Length of Base Plate (mm)	N/A	730
Drive		
Performance DIN 6271 IFN (kW)	37±	3.7
Performance SAE (Kw)	39.5	N/A
Speed (rpm)	2300	3600
Vibratory System		
Frequency (Hz)	32±	68±

Amplitude (mm)	1.24±	N/A
Centrifugal force (Kn)	66±	30±

7.0 CONSTRUCTION

The Contractor is advised that the ultra-lightweight blast furnace slag is susceptible to crushing if overcompacted, and that careful construction supervision is required.

The Contractor shall place the ultra-lightweight fill material and shall achieve compaction without crushing the material, as crushing increases its unit weight.

The Contractor shall place the ultra-lightweight fill material without exceeding the specified in-situ unit weight, and while maintaining crushing of the material below 5%.

To prevent overcrushing and overcompaction, the ultra-lightweight fill shall be placed in accordance with OPSS 206-07 with the following amendments:

- For embankments, the ultra-lightweight fill shall be placed in lifts of 300 mm and compacted by three (3) passes using single drum vibratory equipment such as a Bomag 142 or equivalent.
- For backfill to structures, the ultra-lightweight fill shall be placed in lifts of 300 mm and compacted with eight (8) passes of manually guided tamper such as a Bomag BPR 30/38 D or equivalent.
- The Contractor shall place and spread the loose lifts using a rubber tire front-end loader such as a Caterpillar 980 F or equivalent.

8.0 QUALITY CONTROL

8.1 General

Quality Control (QC) testing shall be carried out by the Contractor for purposes of ensuring that the ultra-lightweight fill material is placed and compacted to the requirements specified in the Contract. Field density and field moisture determination shall be made in accordance with ASTM D6938-10.

Acceptability of compaction shall be based on achieving the target in situ unit weight.

8.2 Control Strip

Under the Supervision of the Quality Verification Engineer, the Contractor shall build a control strip to verify that the placement and compaction procedure will achieve the requirements of this Special Provision without evidence of crushing and without exceeding the specified maximum in-situ unit weight of 12.5 kN/m^3.

Prior to incorporating any of the material into the work the Contractor shall build a minimum trial area of 100 m^2 (approximately 5 m x 20 m) in area consisting of two equal lifts of 300 mm thickness. The Contractor shall give the Contract Administrator written notice of the construction of the control strip 48 hours prior to commencement of this work.

Material placed in the control strip shall have the moisture content that will yield the specified in-situ unit weight. For the control strip determination, the nuclear gauge method will not be considered an acceptable method of determining the in-situ moisture content of the ultra lightweight material. Moisture content shall be determined by the oven dry method on selected compacted embankment material samples in accordance with ASTM D2216.

After the trial area is complete, samples for moisture content and in-situ unit weight determination testing shall be as per ASTM D6938-10.

In addition, Gradation as per ASTM D422-63 before and after compaction effort shall be performed to determine that crushing is kept within 5%.

All test results will be used to determine compliance with the specification. Any proposed changes to the specified compaction method shall be reviewed and approved by the Contract Administrator prior to implementation. The requirements of the control strip must be satisfied as part of the acceptance criteria of any proposed change to the specified compaction method of this Special Provision.

9.0 MEASUREMENT OF PAYMENT

The unit measurement will be tonnes and the method of determining the weight of material for payment shall conform to OPSS 102.

10.0 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.

RETAINED SOIL SYSTEM, TRUE ABUTMENT - Item No.
RETAINED SOIL SYSTEM, FALSE ABUTMENT - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, HIGH PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, MEDIUM PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, LOW PERFORMANCE - Item No.
RETAINED SOIL SYSTEM WITH FINISHING CAP, WALL/SLOPE, HIGH PERFORMANCE - Item No.
RETAINED SOIL SYSTEM WITH FINISHING CAP, WALL/SLOPE, MEDIUM PERFORMANCE - Item No.
RETAINED SOIL SYSTEM WITH TRAFFIC BARRIER, WALL/SLOPE, HIGH PERFORMANCE - Item No.
RETAINED SOIL SYSTEM WITH TRAFFIC BARRIER, WALL/SLOPE, MEDIUM PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, ROADBASE EMBANKMENT - Item No.

Special Provision No. 599S22

1.0 SCOPE

This special provision covers the requirements for the design, supply and construction of Retained Soil Systems (RSS).

Special requirements apply for the design of the steel reinforcing strips where lightweight or ultra-lightweight slag fill is used as backfill to the RSS wall(s), at the locations specified elsewhere in the Contract Documents. The galvanized steel reinforcing strips shall be designed to be thicker (i.e., to have a sacrificial thickness) to mitigate the potential for corrosion in the slag fill environment.

Additional requirements for RSS precast concrete facing elements shall be as specified elsewhere in the Contract.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, General:

OPSS 180 Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Construction:

OPSS 501 Compaction
OPSS 539 Protection Schemes

Ministry of Transportation Publications

MTO Designated Sources of Materials (DSM)
Generic Requirements for Retained Soil Systems for DSM
Ontario Highway Bridge Design Code 1991 - 3rd Edition (OHBDC)

3.0 DEFINITIONS

For the purposes of this special provision the following definitions apply:

Approved Product Drawings: means the documentation for an RSS which has been submitted to the Ministry by the Manufacturer for approval and listing in the DSM, in accordance with the Generic Requirements for Retained Soil Systems for DSM.

Associated Backfill: means all backfill other than engineered backfill necessary to construct the RSS, and to reinstate the excavation for the RSS.

Design Engineer: means the Engineer who produces the working drawings; the Design Engineer shall be certified by the Manufacturer as having the appropriate experience and expertise to provide design services for the Manufacturer’s RSS.

Design Check Engineer: means the Engineer who checks the original design; the Design Check Engineer shall be certified by the Manufacturer as having the appropriate experience and expertise to provide design services for the Manufacturer’s RSS.

Engineered Backfill: means all backfill that is part of the engineered materials comprising the RSS and/or the RSS foundation.

External Stability: means the stability of the foundation and slope/embankment on which the RSS relies for support during and after construction.

Internal Stability: means the stability of the engineered materials comprising the RSS.

Manufacturer: means the party who supplies and/or specifies the design, materials and components for the RSS selected by the Contractor.

Quality Verification Engineer: means an Engineer recognized by the Manufacturer as having demonstrated experience and expertise to provide quality verification services for the Manufacturer's RSS. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and to issue Certificates of Conformance.

Retained Soil System (RSS): means a proprietary system which uses mechanical soil stabilization to retain horizontal loads in excess of 2 m in height for applications such as true and false abutment structures, retaining walls and steep slopes; or, to retain vertical loads for applications such as embankments over soft ground.

Stamped: means working drawings that have been reviewed and stamped “Conforms with Contract Documents”. The stamp shall include the date and signature of the Quality Verification Engineer

4.0 SUBMISSION AND DESIGN REQUIREMENTS

4.1 Submissions

4.1.1 Working Drawings

All submissions shall bear the seal and signature of the Design Engineer and the Design Check Engineer.

The Contractor shall submit working drawings for the design, fabrication and construction of the RSS to the QVE for review and stamping.

The Contractor shall have a copy of the stamped working drawings on site at all times.

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit to the Contract Administrator three (3) sets of the stamped working drawings. The Contract Administrator will forward one set of the stamped working drawings to the Pavement and Foundation Section, Ministry of Transportation, Downsview, for information purposes.

4.1.2 Working Drawing Requirements

Working drawings shall include at least the following:

- All design, fabrication and construction drawings and specifications for the RSS, including details regarding the thickness of the galvanized steel reinforcing strips where slag fill is used;
- Details of all excavation, unwatering, drainage and backfilling required to construct the RSS, including type and source of associated backfill;
- Details at joints and connections to other structures where shown in the Contract Drawings
- Details of all protection systems;
- Statement of bearing resistance required by the RSS foundation, and the bearing resistance provided in accordance with the OHBDC;
- Statement of satisfactory internal and external stability;
- All design, fabrication and construction drawings and specifications for traffic barriers and base, and finishing caps, where applicable;
- Details of how all relevant Operational Constraints and Environmental Constraints, as specified elsewhere in the Contract, will be adhered to.
- A copy of the Approved Product Drawings covering material and construction details

4.1.3 Certificate of Conformance

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of each of the following operations and prior to commencement of each subsequent operation:

- Foundation base preparation
- On-site delivery of manufactured and fabricated components
- Alignment of RSS as per contract documents
- Backfill material

The Certificates of Conformance shall state that the materials and work have been supplied and installed in general conformance with the stamped working drawings and Contract documents.

Upon completion of the RSS installation, the Contractor shall submit to the Contract Administrator a final Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the RSS has been constructed in general conformance with the stamped working drawings and Contract documents.

4.1.4 Warranty

The Contractor shall submit an unconditional warranty to the Owner, to implement all repair and maintenance requirements to the RSS related to design, materials and workmanship for a period of three (3) years from the date of certification of completion of the Contract.

4.2 Design

4.2.1 General

The Contractor shall verify the existing site conditions and ground elevations before preparing the working drawings, and notify the Contract Administrator immediately if site conditions differ from those described in the Contract.

The Application, Performance, and Appearance requirements for the RSS shall be as specified elsewhere in the Contract.

The geometric requirements of the RSS, including alignment and profiles, typical cross-sections, and location of traffic barriers and/or finishing caps, as well as other constraints influencing the design of the RSS, shall be as specified elsewhere in the Contract.

4.2.2 RSS Selection

The Contractor may select any RSS designated as A (Accepted) or as DE (Demonstration) on the DSM List that meets the specified Contract requirements. RSS qualified as DE (Demonstration) status will require inspection, instrumentation, monitoring and reporting by the Manufacturer, in accordance with the Generic Requirements for Retained Soil Systems for DSM.

The RSS selected and designed by the Contractor shall meet all of the requirements for the RSS specified in the Contract.

4.2.3 Design of Steel Reinforcing Strips for Use With Slag Fill

Where lightweight or ultra-lightweight slag fill is to be used as backfill to the RSS, the galvanized steel reinforcing strips will be subjected to higher corrosion rates as compared to sand and gravel backfill. For this application, the galvanized steel reinforcing strips shall be designed and supplied with sufficient thickness for a 75-year design life, based on the following properties for the slag fill:

Electro-Chemical Parameter	Criterion	Test Method
Chlorides	<200 ppm	D4327
Total Sulphates	<1,000 ppm	D2492
Resistivity	>1,000 ohm-cm	G187
pH	5-10	D4972

4.2.4 Foundation Investigation Report

A Foundation Investigation Report that describes the subsurface conditions at the RSS is available, as specified elsewhere in the Contract. The Owner warrants that the information provided in the Foundation Investigation Report can be relied upon with the following limitations and exceptions:

Any interpretations of data or opinions expressed in the report are not warranted; and

Although the raw measured data presented is warranted, the Contractor must satisfy himself as to sufficiency of the information presented and obtain any updating or additional information, and perform any studies, analyses or investigations the Contractor deems necessary in order to prepare his design, at no additional cost to the Owner.

4.2.5 Protection Systems

Where the stability, safety or function of an existing roadway, railway, and other works can be impaired by an excavation or temporary slope, the Contractor shall provide protection systems as required, including sheet-piling, shoring, and the driving of piles where necessary, to prevent damage to such works.

Design of protection systems shall be in accordance with SP 539S01.

5.0 MATERIALS

5.1 General

All materials for the selected RSS shall conform to Approved Product Drawings for that RSS.

5.2 Steel Reinforcing Strips for Use with Slag Fill

Where lightweight or ultra-lightweight slag fill is used as backfill for the RSS, the galvanized steel reinforcing strips shall have sufficient thickness for a 75-year design life.

5.3 Associated Backfill

Associated backfill shall be suitable for the particular application, and be approved by the Design Engineer as compatible with the RSS.

7.0 CONSTRUCTION

7.1 General

The work shall include the construction of the RSS, with traffic barriers and finishing caps where specified, and all excavation, unwatering, drainage and backfilling required to construct the RSS.

Associated backfill shall be compacted in accordance with OPSS 501.

7.2 RSS

The RSS shall be constructed in conformance with the stamped working drawings.

7.3 Protection Systems

Protection systems shall be constructed in accordance with the stamped working drawings.

Protection systems shall be removed in accordance with SP 539S01.

7.4 Management of Excess Materials

Excess materials resulting from carrying out the work shall be removed and managed as specified elsewhere in the Contract.

8.0 QUALITY ASSURANCE

The Contractor shall submit representative samples of the RSS components to the Contract Administrator when requested.

10.0 BASIS OF PAYMENT

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

NOTES TO DESIGNER:

Include SP 599S23 for Precast Concrete Facing Elements
Include SP 539S01 for Protection Systems

WARRANT: Always with these tender items.

APPENDIX E

Instrumentation and Monitoring Plan

SETTLEMENT PLATES – Item No.
SETTLEMENT PINS – Item No.
SETTLEMENT PROFILERS – Item No.
SHAPE ACCEL ARRAYS – Item No.

Special Provision

1.0 GENERAL

The Contractor shall retain a Foundation Engineering consultant registered in MTO's Consultant Registry, Appraisal and Qualifications System (RAQS) for "Geotechnical Specialty – High Complexity", to undertake the supply and installation of geotechnical monitoring instrumentation.

"The Contractor" shall be understood to refer to the Contractor and their Foundation Engineering consultant.

1.1 Scope

This special provision and the other item-specific special provisions contain the requirements for the supply and installation of the following geotechnical monitoring instrumentation:

- Settlement Plates (SP);
- Settlement Pins (S);
- Vibrating Wire Piezometers (VWP);
- Standpipe Piezometers (SSP);
- Settlement Profilers (PR);
- Inclinometers (INC); and
- Shape Accel Arrays (SAA).

This special provision also contains the requirements for the supply and installation of temporary survey Benchmarks (BM).

1.2 Purpose

The purpose of these instruments is to monitor the progress of settlement, lateral displacement and dissipation of excess porewater pressure in the foundation soils under the embankment widening and two-stage retained soil system (RSS) wall construction in the Holland Marsh area. The purpose of the survey Benchmarks is to provide non-settling references for the surveying of the monitoring instruments.

The rate and staging of fill placement and the duration of the preloading period prior to paving and opening to traffic, and prior to installation of the permanent facing panels for the two-stage RSS wall, will be controlled by the instrumentation readings, as specified elsewhere in the Contract Documents. The completed, preloaded embankment and RSS wall area shall remain undisturbed until such time as the monitoring shall indicate that a sufficient degree of consolidation of the foundation soil has been achieved. Pavement construction and installation of the permanent facing panels for the two-stage RSS wall shall not take place until sufficient consolidation has been achieved as determined by the Contract Administrator.

1.3 Or Equal

The term “or equal” shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

1.4 Notification

The Contract Administrator shall be notified a minimum of 15 working days in advance of commencing the installation of instruments.

1.5 Submission Requirements

The Contractor shall submit details of the proposed installation methods including locations and types of the data acquisition system, monitoring enclosure, survey benchmarks and installation schedule, to the Contract Administrator, a minimum of 15 working days before the start of instrument installation.

2.0 SITE CONDITIONS

2.1 Subsurface Conditions

The subsurface conditions at the site are described in the following report:

- Foundation Investigation Report – Embankment Widening and RSS Wall Construction, Highway 400 Widening from North of King Road to North of South Canal Bridges, Regional Municipality of York, GWP 2025-13-00”, by Golder Associates Ltd., dated January 14, 2015.

2.2 Equipment Operation and Weather Conditions

All monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. The instruments shall be capable of operating within the manufacturer’s stated accuracy throughout the temperature range. Monitoring will be conducted year-round by the Contract Administrator.

3.0 MONITORING INSTRUMENT INSTALLATIONS

3.1 Drawings

Reference shall be made to the following drawings that are contained elsewhere in the Contract Documents:

- Monitoring Instrumentation Plans; and
- Typical Instrument Installation Details.

3.2 Quantities and Locations of Instruments

The quantities and location of instruments are presented in Table 1A and are shown on the Contract Drawings.

Table 1A – Instrument Quantities and Locations

Monitoring Section	Approx. Station	Quantities						
		SP	S	VWP	SSP	PR	INC	SAA
Hwy 400 NBL Sta 24+800 to 24+830	24+805	1	1	-	-	-	-	-
	24+825	1	1	-	-	-	-	-

Monitoring Section	Approx. Station	Quantities						
		SP	S	VWP	SSP	PR	INC	SAA
Hwy 400 SBL Sta 24+770 to 24+790	24+770	1	1	-	-	-	-	-
	24+790	1	1	-	-	-	-	-
Hwy 400 NBL Sta 24+920 to 25+140	24+925	1	1	1	1	1	1	1
	24+975	1	1	1	-	-	-	-
	25+025	1	1	1	-	1	1	1
	25+075	1	1	1	-	-	-	-
	25+125	1	1	1	-	-	-	-
Hwy 400 NBL Sta 25+140 to 25+750	25+200	1	1	-	-	-	-	-
	25+275	1	1	-	-	-	-	-
	25+350	1	1	-	-	-	-	-
	25+425	1	1	-	-	-	-	-
	25+500	1	1	-	-	-	-	-
	25+575	1	1	-	-	-	-	-
	25+650	1	1	-	-	-	-	-
	25+725	1	1	-	-	-	-	-
Hwy 400 SBL Sta 24+880 to 25+200	24+890	1	1	1	1	1	-	1
	24+940	1	1	1	-	1	-	1
	24+990	1	1	-	-	-	-	-
	25+040	1	1	-	-	-	-	-
	25+115	1	1	-	-	-	-	-
	25+190	1	1	-	-	-	-	-
TOTALS:		**23**	**23**	**7**	**2**	**4**	**2**	**4**

3.3 Materials and Equipment

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless otherwise noted.

3.4 Instrument Location

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground elevation at each instrument location.

3.5 Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to underground utilities caused by the Contractor's work shall be repaired by the Contractor at no cost to the Owner or Contract Administrator.

3.6 Marking and Labelling

The location of any above-ground monitoring fixture shall be made clearly visible to nearby traffic before, during and after embankment/RSS wall construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

Instruments and their data cables shall be clearly labelled in the field, with each instrument having a unique identifier. The labelling shall remain legible for the entire duration of monitoring.

3.7 Protection of Instruments

The Contractor shall adequately protect all instruments such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced by the Contractor at no cost to the Owner or Contract Administrator.

3.8 Survey Personnel

Surveying to establish the benchmarks and other elevations shall be carried out by a registered surveyor with appropriate equipment. The surveyor shall be retained by the Contractor.

3.9 Accuracy of Surveying for Elevations

Elevations shall be surveyed to an accuracy of ± 2 mm or better.

3.10 Boreholes

The Contractor shall make a basic stratigraphic log of boreholes as they are being drilled for the installation of monitoring instruments. In situ or laboratory geotechnical testing is not required.

Boreholes shall be advanced using conventional drilling methods and shall be as straight and vertical as practicable.

3.11 Installation Program

Instrument installation shall commence immediately after completion of subexcavation as specified elsewhere in the Contract Documents, and prior to the commencement of the embankment or RSS wall construction. Table 1B gives a summary of the installation schedule requirements.

Table 1B – Instrument Installation Program

Instrument Type	Start Installation	Finish Installation
SP	After subexcavation, and before start of embankment/RSS wall construction	Extended as fill placement proceeds, to completion of embankment/RSS wall to preload grade
NP	At completion of embankment/RSS wall construction to preload grade	At completion of embankment/RSS wall construction to preload grade
VWP	After subexcavation, and before start of embankment/RSS wall construction	Adjust as fill placement proceeds, to completion of embankment/RSS wall to preload grade
SSP	Before start of embankment/RSS wall construction	Before start of embankment/RSS wall construction
PR	After subexcavation, and before start of embankment/RSS wall construction	Extended as fill placement proceeds, to completion of embankment/RSS wall to preload grade
INC	After subexcavation, and before start of embankment/RSS wall construction	Before start of embankment/RSS wall construction

Instrument Type	Start Installation	Finish Installation
SAA	After subexcavation, and before start of embankment/RSS wall construction	Before start of embankment/RSS wall construction

4.0 BENCHMARK INSTALLATION

4.1 Number and Locations

The minimum number and approximate locations of the Benchmarks are shown on the Contract Drawings and in Table 2. The number and locations of Benchmarks shall be adjusted in the field such that:

- Direct sighting is possible from all instruments to at least one Benchmark;
- Each Benchmarks is located in an area that will not experience a change in loading (due to grade raise or excavation) that could induce settlement or heave in the ground in which the Benchmark is installed; and
- Each Benchmark is located in such a way to minimize interference with and damage by construction activities.

Table 2 – Survey Benchmark (BM) Locations

Monitoring Area	Approx. Station	Approx. Offset from CL (m)	Approx. Elevation of Bottom of Anchor (m)*	Approx. Length of Rod Incl. Stick-Up (m)*
Hwy 400 NBL Sta 24+800 to 24+830	24+850	60 m east	208	13
Hwy 400 SBL Sta 24+770 to 24+790	24+775	75 m west	208	13
Hwy 400 NBL Sta 24+920 to 25+140	25+050	East shoulder of Wist Road	200	20
Hwy 400 NBL Sta 25+140 to 25+750	25+400	East shoulder of Wist Road	200	20
Hwy 400 SBL Sta 24+880 to 25+200	25+050	West shoulder of Davis Road	196	25

* The rod anchor elevation is approximate and should be adjusted in the field to extend approximately 1 m into soils having Standard Penetration Test "N" values of greater than 50 blows per 0.3 m of penetration.

4.2 Materials

The Contractor shall supply all materials and equipment required for the installation of the Benchmarks.

4.2.1 Rod

The Contractor shall supply a steel pipe, Schedule 40, with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 1.3.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

4.2.2 Sand

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

4.2.3 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

4.2.4 Rod Anchor Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 14 kg of bentonite (OPSS 1205), 49 litres of water and 40 kg of cement (Type GU – OPSS 1301).

4.2.5 Friction-Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

4.3 Installation

The Contractor shall install Benchmarks in accordance with the following:

4.3.1 Borehole

The borehole shall be advanced to the rod anchor elevations provided in Table 1 using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction-reducing sleeve and rod anchor. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

4.3.2 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

4.3.3 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole to form a concrete/soil anchor.

Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 0.5 m length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

4.3.4 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod above the rod anchor and sand, extending up to ground surface.

4.3.5 Installation Details

The elevation, easting and northing of the top of the Benchmark rod shall be surveyed.

5.0 MONITORING

5.1 Notification and Reporting

The Contractor shall notify the Contract Administrator no later than three working days after installing a Benchmark. At this time the Contractor shall also supply the following information to the Contract Administrator:

- Northing and easting of each Benchmark, in MTM NAD 83 coordinates;
- Elevation of the rod anchor and top of rod referenced to geodetic datum;
- Dates of installation;
- Stratigraphic log of subsurface conditions at the Benchmarks;
- Installation notes and sketches; and
- Description of the Benchmark, friction-reducing sleeve and rod anchor.

Notification and reporting requirements for all other instruments are provided in the item-specific special provisions.

5.2 Personnel/Access

Data collection, interpretation and reporting shall be conducted by the Contract Administrator or his representative.

The Contractor shall provide access and assistance to the Contract Administrator's representative reading all geotechnical instruments. This may include, but not necessarily be limited to, the following:

- Safe access to each instrument location;
- A stable platform to support the technician and equipment to access instruments at times when the casing is more than 1.2 m above ground level; and/or
- Power and area lighting.

5.3 Monitoring Program

The Contractor shall meet with the Contract Administrator and staff responsible for the ongoing monitoring immediately after installation of the instruments and before the start of embankment and RSS wall construction. At this meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments, and all equipment to be supplied by the Contractor, as identified in the item-specific special provisions.

Monitoring by the Contract Administrator's representative for the baseline readings shall commence within seven working days after the hand-over meeting. The monitoring shall continue on a schedule to be determined by the Contract Administrator throughout the construction of the embankment widening and RSS wall, and for up to approximately 14 months following the completion of construction to the preload grade.

6.0 DECOMMISSIONING OF INSTRUMENTS

At the end of the monitoring period, the Contractor shall decommission all the temporary survey Benchmarks (BM) by removing the rod and friction-reducing sleeve to at least 1.5 m below grade by excavating and backfilling with compacted granular fill in accordance with the specifications for fill placement.

At the end of the monitoring period, the Contractor shall decommission all Settlement Plates (SP), Settlement Pins (S), Vibrating Wire Piezometers (VWP), Standpipe Piezometers (SSP), Settlement Profilers (PR) and Inclinometers (INC), unless otherwise advised by the Contract Administrator. Decommissioning of instrumentation shall be carried out per the item-specific special provisions and according to the Ontario Water Resources Act, Regulation 903 (as amended).

The Shape Accel Arrays (SAA) shall be kept and protected for long-term monitoring, and shall not be decommissioned.

7.0 MEASUREMENT AND BASIS OF PAYMENT

Payment at the contract price for the above tender items shall include full compensation for all labour, materials and equipment to do the work including the supply and installation of survey benchmarks.

SETTLEMENT PLATES – Item No.

Special Provision

1.0 GENERAL

1.1 Scope

This special provision contains the requirements for the supply and installation of Settlement Plates (SP).

The purpose of the Settlement Plates is to monitor settlements of the embankment/RSS wall base. Settlement is measured by survey of the top of the rod with reference to stable, non-settling Benchmarks. The settlement readings shall help to establish the timing for completion of the preload period.

1.2 General Procedure

The settlement rods shall be attached to a plate at the ground surface following completion of the subexcavation and backfill operations. As embankment or RSS wall construction proceeds, the rods shall be extended above the new top of embankment.

Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate.

A protective surround shall be extended with the rods as embankment/RSS wall construction proceeds.

As the Settlement Plates are located within the new highway shoulders, the rods shall be cut down to a minimum of 0.3 m below the subgrade level after the monitoring program is complete.

1.3 Location

The Contractor shall install Settlement Plates on the shoulder of the widened Highway 400 embankment, at the locations shown on the Contract Drawings and given in Table 1.

Table 1 – Settlement Plate (SP) Locations

Monitoring Section	Approx. Station	Approx. Elevation of Ground Surface (m) *	Estimated Thickness of Embankment (m)
Hwy 400 NBL Sta 24+800 to 24+830	24+805	223	6
	24+825	223	6
Hwy 400 SBL Sta 24+770 to 24+790	24+770	223	6
	24+790	222	7
Hwy 400 NBL Sta 24+920 to 25+140	24+925	220	6.5
	24+975	219	7
	25+025	219	6
	25+075	219	4.5
	25+125	219	4
Hwy 400 NBL	25+200	219	2.5

Monitoring Section	Approx. Station	Approx. Elevation of Ground Surface (m) *	Estimated Thickness of Embankment (m)
Sta 25+140 to 25+750	25+275	219	<2
	25+350	219	<2
	25+425	219	<2
	25+500	219	<2
	25+575	219	<2
	25+650	219	<2
	25+725	219	<2
Hwy 400 SBL Sta 24+880 to 25+200	24+890	221	5.5
	24+940	221	5.5
	24+990	220	4
	25+040	219.5	4
	25+115	219	2.5
	25+190	219	<2
TOTAL:	**23**		

* Ground surface elevation estimated following completion of subexcavation and backfill operation, prior to start of embankment/RSS wall construction.

2.0 MATERIALS

The Contractor shall supply all materials and equipment required for the installation of the Settlement Plates.

2.1 Plate

The Contractor shall supply a steel plate with thickness of at least 6.35 mm. The plate shall be at least 0.5 m by 0.5 m in plan dimensions.

2.2 Rod

The Contractor shall supply a steel pipe with an outside diameter not less than 25 mm, supplied in lengths as required to complete the installation as described in Section 1.3.

The top end of each rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

2.3 Friction-Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50 mm outer diameter PVC pipe cut perpendicular to the axis of the pipe.

2.4 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod and friction-reducing sleeve within the embankment.

The surround shall consist of 300 mm diameter corrugated steel pipe (CSP – OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the friction-reducing sleeve shall be filled with medium to coarse sand.

3.0 INSTALLATION

The Contractor shall install Settlement Plates as shown on the Contract Drawings and the typical installation detail, in addition to what is stated below.

3.1 Settlement Plate

The settlement plate shall be installed horizontally on the ground surface following completion of the subexcavation and backfilling works.

The elevation of the plate shall be surveyed before fill placement commences for the embankment/RSS wall construction.

3.2 Rod

The Settlement Plate rod shall be fixed to the centre of the plate and perpendicular to the plate.

The rod shall be extended in 1.5 m increments as the embankment increases in height.

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

3.3 Friction-Reducing Sleeve

The friction-reducing sleeve shall extend over the entire length of the rod that is below ground and within the embankment fill, except that the cap on top of the Settlement Plate rod shall extend 25 mm above the top of the friction sleeve at all times.

3.4 Protective Surround

The CSP, friction-reducing sleeve and sand surround shall be extended with the rods.

The Settlement Plate rod shall be in the centre of the CSP and friction-reducing sleeve.

The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the friction-reducing sleeve.

3.5 Installation Details

The elevation, northing and easting of the centre of the base of the plate shall be surveyed by the Contractor.

The elevation, northing and easting of the top of the rod shall be surveyed by the Contractor.

The total distance from the base of the plate to the top of the rod shall be measured and recorded by the Contractor to an accuracy of ± 2 mm or better.

The Contractor is responsible for preventing damage to the settlement rod during the fill placement process. If the rod or extension is damaged during fill placement, the rods, friction-reducing sleeve and protective surround shall be replaced before resuming the fill placement.

4.0 COORDINATION WITH MONITORING

4.1 Notification and Reporting

The Contractor shall notify the Contract Administrator no later than three working days after installing Settlement Plates (SP). At this time the Contractor shall also supply the following information to the Contract Administrator:

- Northing and easting of each Settlement Plate in MTM NAD 83 coordinates;
- Elevation of the plate and top of rod referenced to geodetic datum;
- Dates of installation;
- Installation notes and sketches; and
- Description of the settlement plate, rod and friction-reducing sleeve.

Adjustments in the length of any Settlement Plate rod during ongoing construction activities shall be coordinated with the Contract Administrator to allow surveying by others of the elevation of the top of the rod immediately before and immediately after adjustment. This surveying is necessary to accurately track the settlement data.

4.2 Monitoring

Monitoring of the Settlement Plates shall be carried out by others under the Contract Administrator assignment. Monitoring shall be conducted during the embankment and RSS wall construction, throughout the preloading period, and following completion of the preloading period. The Contractor shall provide access to the Settlement Plates for monitoring including, but not limited to, a scaffolding platform and ladder if required and snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed for reading the instruments.

5.0 REMOVAL/DECOMMISSIONING

After completion of the settlement monitoring period, the settlement rods shall be removed to at least 0.3 m below grade by excavating and cutting of the protective surround, friction-reducing sleeve and rod. The excavations should be backfilled with compacted granular fill in accordance with the specifications for fill placement.

6.0 BASIS OF PAYMENT

Measurement for payment will be made on the basis of the number of units of Settlement Plates (SP) installed, including extension through the fills, and then decommissioned following completion of the monitoring period.

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, equipment and materials to do the work, including all appurtenances, extension through the fills, the required reporting, and decommissioning.

<u>SETTLEMENT PINS</u> – Item No.

Special Provision

1.0 GENERAL

1.1 Scope

This special provision contains the requirements for the supply and installation of Settlement Pins (S).

The purpose of the Settlement Pins is to monitor settlements of the embankment/RSS wall fill. Settlement is measured by survey of the top of the pin with reference to stable, non-settling Benchmarks. The settlement readings shall help to establish the timing for completion of the preload period.

1.2 General Procedure

The Settlement Pins shall be cast into concrete at the top of the embankments/RSS wall, as shown on the Contract Drawings. The concrete will be cast in situ in a hold dug at the locations of the Settlement Pins.

1.3 Location

The Contractor shall install Settlement Pins on the shoulder of the widened Highway 400 embankment, at the locations shown on the Contract Drawings and given in Table 1. In general, the Settlement Pins shall be located on the widened Highway 400 shoulder, within approximately 1 m of the corresponding Settlement Plate (SP) at each of the monitoring stations identified below.

<u>Table 1 – Settlement Pin (S) Locations</u>

<table>
<tr><th>Monitoring
Section</th><th>Approx.
Station</th></tr>
<tr><td rowspan="2">Hwy 400 NBL
Sta 24+800 to 24+830</td><td>24+805</td></tr>
<tr><td>24+825</td></tr>
<tr><td rowspan="2">Hwy 400 SBL
Sta 24+770 to 24+790</td><td>24+770</td></tr>
<tr><td>24+790</td></tr>
<tr><td rowspan="5">Hwy 400 NBL
Sta 24+920 to 25+140</td><td>24+925</td></tr>
<tr><td>24+975</td></tr>
<tr><td>25+025</td></tr>
<tr><td>25+075</td></tr>
<tr><td>25+125</td></tr>
<tr><td rowspan="8">Hwy 400 NBL
Sta 25+140 to 25+750</td><td>25+200</td></tr>
<tr><td>25+275</td></tr>
<tr><td>25+350</td></tr>
<tr><td>25+425</td></tr>
<tr><td>25+500</td></tr>
<tr><td>25+575</td></tr>
<tr><td>25+650</td></tr>
<tr><td>25+725</td></tr>
<tr><td rowspan="2">Hwy 400 SBL
Sta 24+880 to 25+200</td><td>24+890</td></tr>
<tr><td>24+940</td></tr>
</table>

Monitoring Section	Approx. Station
	24+990
	25+040
	25+115
	25+190
TOTAL:	**23**

2.0 MATERIALS

The Contractor shall supply all materials and equipment required for the installation of the Settlement Pins.

2.1 Pin

The Contractor shall supply a minimum 25 mm diameter reinforcing steel bar (OPSS.PROV 905), cut 0.4 m long.

The top of the reinforcing steel bar shall be angled or rounded in such a way that a single survey point can be clearly identified and returned to.

2.2 Concrete

The Contractor shall supply concrete (OPSS.PROV 1350) of minimum 25 MPa compressive strength and set time sufficient to secure the Nail Pins within two days of pouring.

3.0 INSTALLATION

The Contractor shall install Settlement Pins as shown on the Contract Drawings and the typical installation detail.

4.0 COORDINATION WITH MONITORING

4.1 Notification and Reporting

The Contractor shall notify the Contract Administrator no later than three working days after installing Nail Pins. At this time the Contractor shall also supply the following information to the Contract Administrator:

- Northing and easting of each Settlement Pin in MTM NAD 83 coordinates;
- Elevation of the Settlement Pin referenced to geodetic datum;
- Dates of installation; and
- Installation notes and sketches.

4.2 Monitoring

Monitoring of the Settlement Pins shall be carried out by others under the Contract Administrator assignment. Monitoring shall be conducted after completion of the embankment and RSS wall construction, throughout the preloading period, and following completion of the preloading period. The Contractor shall provide access to the Settlement Pins for monitoring including, but not limited to snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed for reading the instruments.

5.0 REMOVAL/DECOMMISSIONING

After completion of the settlement monitoring period, the Settlement Pins shall be removed by excavating the concrete surround. The excavations shall be backfilled with compacted granular fill in accordance with the specifications for fill placement.

6.0 BASIS OF PAYMENT

Measurement for payment will be made on the basis of the number of units of Settlement Pins (S) installed and then decommissioned following completion of the monitoring period.

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, equipment and materials to do the work, including all appurtenances and required reporting.

VIBRATING WIRE PIEZOMETER – Item No.

Special Provision

1.0 GENERAL

1.1 Scope

This special provision contains the requirements for the supply and installation of Vibrating Wire Piezometers (VWP).

The purpose of the piezometers is to monitor porewater pressures at depth within the foundation soils. The piezometer readings shall help to confirm the timing for the fill placement for the embankment widening and RSS wall construction, and the timing for completion of the preloading period.

1.2 General Procedure

The piezometers shall be installed in boreholes after completion of the subexcavation and backfilling operations, but prior to any embankment widening or RSS wall construction. The boreholes shall be of sufficient diameter to accommodate installation of the VWP sensor, filter sand and grout.

The VWP signal cables shall be extended out of the embankment widening/RSS wall footprint area through a metal or plastic conduit buried in a trench, as shown in the typical instrument installation details.

Boreholes containing VWP sensors shall be located at least 3 m from other instrument boreholes.

1.3 Locations

The Contractor shall install VWP sensors under the widened Highway 400 embankment shoulder, at the locations and elevations given in Table 1.

Table 1 – Vibrating Wire Piezometer (VWP) Locations and Elevations

Monitoring Section	Approx. Station	Approx. Ground Surface Elevation (m)*	Tip Elevation (m)
Hwy 400 NBL Sta 24+920 to 25+140	24+925	220	212
	24+975	219	210
	25+025	219	208
	25+075	219	208
	25+125	219	205
Hwy 400 SBL Sta 24+880 to 25+200	24+890	221	212
	24+940	221	210
TOTALS:	**7**		

* Ground surface elevation estimated following completion of subexcavation and backfill operation, prior to start of embankment/RSS wall construction.

2.0 MATERIALS

The Contractor shall supply materials and equipment, including drill rigs, required for installation of the Vibrating Wire Piezometers.

2.1 Vibrating Wire Piezometers

The Contractor shall supply VW borehole piezometers by Slope Indicator Model 52611020 (-5 psi to 50 psi), RST model VW2100-0.35 – or equal; compatible with the Slope Indicator Model CR1000 data-logger, RST Model ELGL1200 – or equal. All VW piezometers shall be of the same make and supplier.

All piezometers shall be calibrated prior to installation and the calibration data for each piezometer shall be provided to the Contract Administrator.

2.2 Signal Cable

The Contractor shall supply Slope Indicator Model 50613524 cable, RST Model EL380004 cable – or equal. The length of cable for each piezometer shall be carefully estimated from the Contract Drawings to ensure that there is sufficient additional length of signal cable for each piezometer to provide enough slack in the borehole and along the monitoring trenches to reach the location of each terminal. The cables and terminals shall be protected from construction equipment at all times.

2.3 Bentonite

The Contractor shall supply bentonite in pellet form in sufficient quantity to form borehole plugs as required.

The Contractor shall supply bentonite in powder form in sufficient quantity for the bentonite-cement grout mix for general borehole backfilling.

2.4 Filter Sand

The Contractor shall supply clean washed sand for filter around VW sensors. The sand shall be Sakcrete washed general-purpose sand – or equal.

2.5 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

2.6 Trench Burial and Conduit

The signal cable for each VWP shall be buried in a shallow trench as shown on the Contract Drawings, to extend outside of the embankment widening/RSS wall construction areas. The Contractor shall supply suitable conduits (e.g. Schedule 40, 75 mm steel pipe or Schedule 80, 75 mm rigid PVC pipe) to protect the signal cables in the trenches and above ground surface. If appropriate, several signal cables may be housed in a single conduit and laid in a common trench.

2.7 Data Acquisition System (Data-Logger)

The signal cables from the vibrating wire piezometers shall be connected to the nearest data-logger, Slope Indicator Model 56701000 (CR1000), RST Model ELGL1200 – or equal. The data-loggers shall consist of the following:

- ENC 16/18 Waterproof Enclosure Model 56705020, Model ELF0638 – or equal;
- SC32A Serial Interface (with RS232 transfer cable) Model 56704010, Model CS-SC32A – or equal;
- VW Interface Model 56701510 or 56701500, Model CS-AVW200 – or equal;
- AM16/32 Multiplexer Model 56702110, Model ELGL2042 – or equal;
- A suitable power supply that shall be able to last for a minimum of 2 years for long term settlement monitoring (i.e. a large capacity rechargeable battery coupled with solar panel);
- LoggerNet Software Model 56708020, Model CS-Loggernet – or equal.

The Contractor shall submit a detailed proposal on the setup of the data-logging system (i.e. numbers and locations of the data-logging unit(s)) to the Contract Administrator for review, prior to ordering the data-logger(s). The Contractor shall program the data-loggers according to the following:

- Recording Software: VWP data shall be recorded two (2) times a day (i.e. one (1) reading every 12 hours); and
- Test Software: Once this program is transferred to the data-logger, the system shall be able to be tested and data recorded manually on site.

The real-time data shall be retrieved on site by direct wire (i.e. RS232 Cable) with a portable laptop computer as specified in the next section.

2.8 Portable Laptop Computer

For the purposes of monitoring the VWPs the Contractor shall supply the following:

- A new Portable Laptop Computer (with a three-year warranty): Intel Core i5 or equivalent (2.4 GHz or higher) with Windows 7 (English), 4 GB memory, a minimum of 250 GB hard drive storage, a DVD+/-RW and Microsoft Office 2010, to retrieve, read and store the VW piezometer readings.
- An extra battery for the above portable laptop computer and a vehicle adaptor for computer charger.

The portable laptop computer will become property of the MTO and shall be handed to the Contract Administrator after the installation of instruments for the monitoring program.

The calibration factors for all vibrating wire piezometers shall be entered in the portable laptop computer by the Contractor for initialization of the instruments.

2.9 Wooden Posts

Wooden posts for the support of the data acquisition system enclosures shall be 100 mm by 100 mm in cross-section, minimum 3 m long pressure treated lumber, installed a minimum of 1.5 m into the ground.

3.0 INSTALLATION

3.1 General

Installation of the VWPs shall be in accordance with the manufacturer’s recommendations in addition to what is stated or emphasised below.

3.2 Borehole Installation

The borehole shall be advanced to 300 mm below the tip elevation using suitable drilling techniques. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

The piezometer sensor shall be saturated, per the manufacturer's recommendations. In addition, the borehole shall be filled with water upon installation of the sensor into the base of the hole to maintain saturation of the sensor throughout the installation process.

The piezometer shall be installed according to the typical installation detail shown in the Contract Documents.

3.3 Protection for Long-Term Monitoring (Monitoring Shed)

The data-loggers shall be installed in a walk-in Monitoring Shed to prevent vandalism and minimize exposure of the data-loggers to extreme weather conditions. The Monitoring Shed shall be lockable and weather-resistant. The Monitoring Shed shall be seated on a gravel pad and securely tied down to ground. The location of the Monitoring Shed shall not be susceptible to ground settlement. The Contractor shall submit a detailed proposal of the Monitoring Shed (i.e. materials and locations) to the Contract Administrator for review, prior to construction.

The Contractor shall ensure access to the Monitoring Shed at all times, including but not limited to snow clearing in the winter.

3.4 Completion of Installation

It is known that the process of installing VWPs can temporarily alter the porewater pressure acting on the piezometer tip. The installation of a VWP shall not be considered to be complete until the porewater pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. The Contractor shall take daily readings of the porewater pressure at each VWP until the value has stabilized. Stabilization shall be deemed to have occurred as follows:

- When no change in the measured value has occurred over a period of five (5) consecutive days and the measured value is within 10 percent of the anticipated hydrostatic value; and
- When the daily rate of change is less than four (4) kPa per day for three (3) consecutive days and the measured value is within 5 percent of the anticipated hydrostatic value.

The Contractor should be prepared to wait for a period of 10 to 15 days after completion of installation of VWPs for the readings to stabilize.

4.0 COORDINATION WITH MONITORING

4.1 Notification and Reporting

The Contractor shall notify the Contract Administrator no later than three working days after the installation of a VWP. At this time, the Contractor shall also supply the following information to the Contract Administrator.

- Northing and easting of each VWP in MTM NAD 83 coordinates;
- Elevations of VW sensors referenced to geodetic datum;
- Dates of installation;
- Stratigraphic log of subsurface conditions;
- Installation notes and sketches;

- Model, make and serial numbers of VWP sensors, readout unit and signal cable; and
- Calibration details of VW sensors.

4.2 Monitoring

Monitoring of the VWPs shall be done by others. The Contractor shall transfer the Portable Laptop Computer to the Contract Administrator, including all the data-logging software and hardware, operation instructions and calibration constants. The contractor shall also transfer the keys for the locks of the Monitoring Shed(s). The Contractor shall be available for an on-site meeting with the Contract Administrator to transfer these items and explain/provide responses to questions from the Contract Administrator regarding the data-logging system.

Monitoring shall be conducted during the embankment widening and RSS wall construction, throughout the preloading period, and following completion of the preloading period. The Contractor shall provide access to the data-loggers for monitoring including, but not limited to, snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed.

5.0 BASIS OF PAYMENT

Measurement for payment will be made on the basis of the number of units of Vibrating Wire Piezometers (VWP) installed.

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, equipment and materials to do the work, including all appurtenances and required reporting.

<u>STANDPIPE PIEZOMETER</u> – Item No.

Special Provision

1.0 GENERAL

1.1 Scope

This special provision contains the requirements for the supply and installation of Standpipe Piezometers.

The purpose of the Standpipe Piezometers is to monitor the groundwater level within the compressible clay deposits, as a reference for the Vibrating Wire Piezometer measurements.

1.2 General Procedure

The Standpipe Piezometers may be installed at any time prior to the start of embankment widening and RSS wall construction.

The Standpipe Piezometers shall be installed in vertical boreholes.

1.3 Location

The Contractor shall install Standpipe Piezometers in areas that will not experience a change in loading (due to either grade raise or excavation). Suggested locations are shown on the Contract Drawings and given in Table 1 below; however, these locations may be adjusted by the Contractor based on their construction activities, subject to approval from the Contract Administrator.

<u>Table 1 – Standpipe Piezometer (SSP) Locations and Elevations</u>

Monitoring Section	Approx. Station	Approx. Ground Surface Elevation (m)	Tip Elevation (m)
Hwy 400 NBL Sta 24+920 to 25+140	24+925 Offset East	220	212
Hwy 400 SBL Sta 24+880 to 25+200	24+850 Offset West	220	212
TOTALS:	**2**		

2.0 MATERIALS

The Contractor shall supply material and equipment, including drill rigs, required for installation of the Standpipe Piezometers.

2.1 Pipe and Couplings

The Contractor shall supply Schedule 40, flush-jointed PVC pipe with an internal diameter no smaller than 19 mm, and appropriate couplings.

2.2 Perforated Section

The Contractor shall supply a 1.5 m long perforated pipe section, consisting of Schedule 40, flush-jointed, 19 mm PVC slotted pipe for each Standpipe Piezometer.

2.3 Bottom Cap

The Contractor shall supply a bottom cap to fit the perforated section.

2.4 Top Caps

The Contractor shall supply vented top caps to fit the pipe.

2.5 Filter Sand

The Contractor shall supply clean washed sand for backfilling around the perforated section. The sand shall be Sakrete washed general purpose sand – or equal.

2.6 Bentonite

The Contractor shall supply bentonite (OPSS 1205) in pellet form for backfilling above the filter sand.

2.7 Grout

The Contractor shall supply cement-bentonite grout for general backfilling. A suitable grout mix design consists of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

2.8 Protective Housing

The Contractor shall supply a protective housing consisting of a galvanized steel pipe or box section with a minimum internal dimension of 100 mm, equipped with a locking cap to enclose the portion of the standpipe that is above the ground.

3.0 INSTALLATION

Installation of the Standpipe Piezometers shall be as shown on the Contract Drawings in addition to what is stated or emphasised below.

The borehole shall be advanced to 300 mm below the tip elevation using suitable drilling techniques. The sides of the borehole shall be stable and the borehole shall be free of debris.

The Standpipe Piezometers must be of sufficient length above the ground surface to ensure that the anticipated piezometric head is accommodated, and to allow for snow accumulation.

The protective housing shall be cemented in place around the standpipe so as to remain secure and stable throughout the duration of the monitoring.

4.0 COORDINATION WITH MONITORING

4.1 Notification and Reporting

The Contractor shall notify the Contract Administrator no later than three working days after installing Standpipe Piezometers. At this time the Contractor shall also supply the following information to the Contract Administrator:

- Northings and eastings of each Standpipe Piezometer in MTM NAD 83 coordinates;
- Elevation of the ground surface at the Standpipe Piezometer location, referenced to geodetic datum;
- Dates of installation;
- Stratigraphic log of subsurface conditions at the Standpipe Piezometers;
- Installation/backfilling notes, including the depth of the Standpipe Piezometer screen and filter pack, descriptions of the screen and standpipe, and details regarding the stick-up above ground surface.

4.2 Monitoring

Monitoring of the Standpipe Piezometers shall be done by others. Monitoring shall be conducted during the embankment widening and RSS wall construction, during the preloading period, and for approximately six months following completion of the preloading period. The Contractor shall provide access to the Standpipe Piezometers for monitoring including, but not necessarily limited to, snow clearing in the winter. The contractor shall provide general area lighting as needed for reading the instruments.

5.0 DECOMMISSIONING

After completion of the monitoring period, the Standpipe Piezometers shall be decommissioned in accordance with Ontario Water Resources Act, Regulation 903 (as amended).

5.1 BASIS OF PAYMENT

Measurement for payment will be made on the basis of the number of units of Standpipe Piezometers (SSP) installed and then decommissioned following completion of the monitoring period.

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, equipment and materials to do the work, including all appurtenances, the required reporting, and decommissioning.

<u>SETTLEMENT PROFILER</u> – Item No.

Special Provision

1.0 GENERAL

1.1 Scope

This special provision contains the requirements for the supply and installation of Settlement Profilers (PR).

The purpose of the Settlement Profilers is to monitor settlements under the embankment/RSS wall. The settlement readings shall help to establish the timing for completion of the preload period.

1.2 General Procedure

The Settlement Profilers shall be installed to the target elevations following completion of the subexcavation and backfilling operation, and prior to starting construction of the embankment widening/RSS wall. The system shall be installed in conjunction with inclinometer casing as specified in the Contract Drawings.

The base of the Settlement Profilers shall be installed at the bottom of a borehole drilled to stable ground.

Sensing rings for use in borehole applications are to be installed at desired elevations along the pipe.

The installation phase shall be complete when the surrounding embankment is at the final height for the preloading period, and extension of the pipe system is no longer required.

1.3 Location

The Contractor shall install Settlement Profilers so that they are positioned under the shoulder of the widened Highway 400 embankment, at the approximate locations shown on the Contract Drawings and given in Table 1.

<u>Table 1 – Settlement Profiler (PR) Locations</u>

Monitoring Section	Approx. Station	Approx. Elevation of Base of Settlement Profiler (m) *	Estimated Final Pipe/Casing Length (m) **	Approximate Spacing of Sensing Rings (m)
Hwy 400 NBL Sta 24+920 to 25+140	24+925	205	23	Every 1 m from base to original grade (approx. 15 total)
	25+025	200	25	Every 1 m from base to original grade (approx. 20 total)

Monitoring Section	Approx. Station	Approx. Elevation of Base of Settlement Profiler (m) *	Estimated Final Pipe/Casing Length (m) **	Approximate Spacing of Sensing Rings (m)
Hwy 400 SBL Sta 24+880 to 25+200	24+890	205	23	Every 1 m from base to original grade (approx. 15 total)
	24+940	200	27	Every 1 m from base to original grade (approx. 20 total)
TOTAL:	**4**			

* The actual elevation of the base of the pipe and the sensing rings shall be determined by the Contractor during drilling of the borehole, based on socketing a minimum of 1.5 m into the very dense/hard till deposit.

** The Contractor shall provide an additional 6 m of inclinometer casing and pipe per location to allow for a deeper installation than anticipated.

2.0 MATERIALS

The Contractor shall supply all materials and equipment required, including drill rigs, for the installation of the Settlement Profilers.

2.1 Sensing Rings

The Contractor shall supply sensing rings (stainless steel straps) of Model 02842004 or equal for user-installed rings, or Model 50801800 or equal for factory-installed rings.

2.2 Pipes

The Contractor shall supply 3-inch internal diameter corrugated pipes of Model 50801600 or equal for use with 70 mm inclinometer casing.

2.3 Couplings and End Caps

The Contractor shall connect the pipe segments using 3-inch internal diameter couplings of Model 50801602 or equal. The couplings shall be sealed using mastic tape of Model 51003800 or equal, as per manufacturer's specifications. An end cap of Model 50801601 or equal shall be used at the bottom of the corrugated pipe.

2.4 Inclinometer Casing

The Contractor shall supply the 70 mm QC inclinometer casing of Model 51150210 and 51150211 or equal. Telescopic sections shall be used to allow axial movement of the inclinometer casing while minimizing distortion due to vertical strain as necessary.

2.5 Grout

The annular space between the corrugated pipe and the borehole shall be filled with grout that has similar strength as the surrounding soil, to be designed by the Contractor. The grout mix shall have a low drying shrinkage.

The Contractor shall submit a grout mix design to the Contract Administrator for information purposes, no later than 15 days prior to the start of installation of the Settlement Profilers.

2.6 Readout Probe

The Contractor shall supply a readout unit with 100 m of cable of Model 50810315 or equal, and Teflon-coated, non-stretch, flat survey tape.

3.0 INSTALLATION

The Contractor shall install Settlement Profilers per the manufacturer's recommendations, in addition to what is shown on the Contract Drawings and the typical installation detail, and stated or emphasized below.

3.1 Boreholes

The boreholes shall be +/- 2 percent of vertical. The boreholes shall be of sufficient diameter to enable installation of the inclinometer casing and pipe and grouting of the annular space between the pipe and borehole.

The inclinometer casing and pipe socket length shall extend a minimum of 1.5 m into the very dense/hard till material, and shall be confirmed by the Contractor during drilling of the borehole.

3.2 Inclinometer Casing and Pipe

Care shall be taken not to apply torsion to the inclinometer casing or pipe during installation.

The joints in the inclinometer casing shall be wrapped with Denso Petrolatum Tape or equal.

The couplings shall be sealed with Mastic tape at the coupling joint, then wrapped with tape over the coupling; cable-ties shall then be strapped over the taped joints.

When installing and grouting around the inclinometer casing and pipe, the buoyancy force acting on the casing must be balanced. Clean water can be added inside the inclinometer casing or access pipe, but additional force may be required. If so, the force shall be applied below the lowest telescopic section and is ideally applied at the base of the inclinometer casing. The casing or pipe shall not be pushed down from the top as this will cause telescope sections to prematurely contract or collapse, and thus render the telescopic sections unusable.

3.3 Sensing Rings

Sensing rings are fixed to the pipe by the user, or can be factory-installed.

3.4 Grouting

Prior to grouting, the Contractor shall lower the dummy probe to confirm that the probe can reach the bottom of the inclinometer casing.

The annulus between the borehole and pipe shall be grouted up to the existing ground level. All drilling slurry shall be flushed out of the borehole. Grout shall displace any water from the borehole.

Once grouting is completed, the Contractor shall lower the dummy probe to the bottom of the inclinometer casing to confirm that it has been correctly installed.

3.5 Protective Surround

A PVC pipe shall be placed around the inclinometer casing/pipe system to a slightly lower height. The internal diameter of the pipe and its couplings shall be such that the access pipe is free to slide inside, but without excessive play.

A protective surround, consisting of a corrugated steel pipe and sand backfill, shall be placed around the portion of pipes that are above ground.

The above-ground portion of the access pipe shall be greater than 0.3 m in length.

3.6 Extension of Inclinometer Casing and Pipe

As embankment/RSS wall construction proceeds, the inclinometer casing and pipe and the protective surround shall be extended so that they are always above the current ground level.

Each inclinometer casing and pipe shall be inspected by the Contract Administrator following completion of installation and before the start of embankment construction. The Contractor shall re-grout any casings that are found to be loose or where the grout has settled, at no cost to the Owner.

3.7 Protective Housing After Embankment Construction

Following completion of the embankment/RSS wall construction, the protective housing described elsewhere in this special provision shall be cemented in place around the inclinometer casing or access pipe so as to remain secure and stable throughout the duration of the monitoring period.

4.0 COORDINATION WITH MONITORING

4.1 Notification and Reporting

The Contractor shall notify the Contract Administrator no later than three working days after installing Settlement Profilers (PR). At this time the Contractor shall also supply the following information to the Contract Administrator:

- Northing and easting of each Settlement Profiler in MTM NAD 83 coordinates;
- Elevation of the ground level and top of pipe;
- Dates of installation;
- Stratigraphic log of subsurface conditions at the Settlement Profiler location;
- Installation notes and sketches, including socket details, and the depths of the inclinometer casing and pipe; and
- Elevations/depths of the datum and sensing rings.

4.2 Monitoring

Monitoring of the Settlement Profilers shall be carried out by others under the Contract Administrator assignment. Monitoring shall be conducted during the embankment and RSS wall construction, throughout the preloading period, and following completion of the preloading period. The Contractor shall provide access to the Settlement Profilers for monitoring including, but not limited to, a scaffolding platform and ladder if required and snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed for reading the instruments.

5.0 REMOVAL/DECOMMISSIONING

After completion of the settlement monitoring period, the Settlement Profilers shall be decommissioned in accordance with Ontario Water Resources Act, Regulation 903 (as amended).

6.0 BASIS OF PAYMENT

Measurement for payment will be made on the basis of the number of units of Settlement Profilers (PR) installed, including extension through the fills, and then decommissioned following completion of the monitoring period.

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, equipment and materials to do the work, including all appurtenances, extension through the fills, the required reporting, and decommissioning.

<u>INCLINOMETERS</u> – Item No.

Special Provision

1.0 GENERAL

1.1 Scope

This special provision contains the requirements for the supply and installation of Inclinometers (INC).

The purpose of the Inclinometers is to monitor lateral displacements in the foundation soils in front of the retained soil system (RSS) wall.

1.2 General Procedure

The inclinometers shall be installed to the ground surface elevations after completion of the subexcavation and backfilling operation, but prior to beginning the embankment widening/RSS wall construction. As the embankment height increases in lifts, the inclinometer casing shall be extended upward through the embankment fill.

The installation phase shall be complete when the surrounding embankment is at the final design height for the preloading period, and extension of the inclinometer casing is no longer required.

1.3 Location

The Contractor shall install Inclinometers so that they are positioned under the shoulder of the widened Highway 400 embankment, at the approximate locations shown on the Contract Drawings and given in Table 1.

<u>Table 1 – Inclinometer (INC) Locations</u>

Monitoring Section	Approx. Station	Approx. Elevation of Base of Inclinometer (m) *	Estimated Final Pipe/Casing Length (m) **
Hwy 400 NBL Sta 24+920 to 25+140	24+925	205	23
	25+025	200	25
TOTAL:	**2**		

* The actual elevation of the bottom of the inclinometer shall be determined by the Contractor during drilling of the borehole, based on socketing a minimum of 1.5 m into the very dense/hard till deposit.

** The Contractor shall provide an additional 6 m of inclinometer casing per location to allow for a deeper installation than anticipated.

2.0 MATERIALS

The Contractor shall supply all materials and equipment required, including drill rigs, for the installation of the Settlement Profilers.

2.1 Casing and Fittings

The Contractor shall supply inclinometer QC casing, manufactured by Slope Indicator Company or equal. The casing shall be 70 mm outer diameter, Slope Indicator Model 51150210 or 51150211 or equal. Fittings for the casing shall be consistent in manufacturer and system.

2.2 Telescopic Casing Sections

The Contractor shall supply telescopic casing sections of Slope Indicator model 51150220 or equal.

2.3 Splices

If required, the Contractor shall supply splice kits of Slope Indicator Model 51150250 (male) or 51150251 (female) or equal.

2.4 Bottom Caps

The Contractor shall supply bottom caps of Slope Indicator Model 51150230 or equal.

2.5 Top Caps

The Contractor shall supply top caps of Slope Indicator Model 51101500 or equal.

2.6 Grout

The annular space between the inclinometer casing and the borehole shall be filled with grout that has similar strength as the surrounding soil, to be designed by the Contractor. The grout mix shall have a low drying shrinkage.

The Contractor shall submit a grout mix design to the Contract Administrator for information purposes, no later than 15 days prior to the start of installation of the Inclinometers.

2.7 Protective Surround During Embankment Construction

The Contractor shall supply a protective surround for the portion of the inclinometer casing in the embankment during construction. The protective surround shall consist of an inner plastic sleeve to reduce friction, and an outer 300 mm diameter corrugated steel pipe filled with compacted sand.

2.8 Protective Surround During Embankment Construction

The Contractor shall supply a protective housing consisting of galvanized steel pipe or box section with a minimum internal dimension of 100 mm and equipped with a locking cap to enclose the portion of the inclinometer caising that is above ground after construction of the embankment at the inclinometer locations.

3.0 INSTALLATION

The Contractor shall install Inclinometers per the manufacturer’s recommendations, in addition to what is shown on the Contract Drawings and the typical installation detail, and stated or emphasized below.

3.1 Boreholes

The boreholes shall be +/- 2 percent of vertical. The boreholes shall be of sufficient diameter to enable installation of the inclinometer casing and grouting of the annular space between the inclinometer casing and borehole.

The inclinometer casing and pipe socket length shall extend a minimum of 1.5 m into the very dense/hard till material, and shall be confirmed by the Contractor during drilling of the borehole.

3.2 Inclinometer Casing

The A+ inclinometer groove shall be aligned parallel to Highway 400.

The B+ inclinometer groove shall be aligned perpendicular to Highway 400, in the direction away from the median centreline.

The A+ and B+ direction grooves shall be permanently marked and identified on each casing.

Care shall be taken not to apply torsion to the inclinometer casing during installation.

The joints in the inclinometer casing shall be wrapped with Denso Petrolatum Tape or equal.

When installing and grouting around the inclinometer casing, the buoyancy force acting on the casing must be balanced. Clean water can be added inside the inclinometer casing, but additional force may be required. If so, the force shall be applied below the lowest telescopic section and is ideally applied at the base of the inclinometer casing. The casing or pipe shall not be pushed down from the top as this will cause telescope sections to prematurely contract or collapse, and thus render the telescopic sections unusable.

3.3 Telescopic Couplings

Two telescopic couplings shall be included per inclinometer. The couplings shall each accommodate up to 0.15 m of contraction.

The telescopic couplings shall be installed at approximately 4 m and 8 m below existing ground level (to be adjusted for casing lengths after base elevation established).

3.4 Grouting

Prior to grouting, the Contractor shall lower the dummy probe to confirm that all grooves are properly aligned and that the probe can reach the bottom of the inclinometer casing.

The annulus between the borehole and inclinometer casing shall be grouted up to the existing ground level. All drilling slurry shall be flushed out of the borehole. Grout shall displace any water from the borehole.

Once grouting is completed, the Contractor shall lower the dummy probe to the bottom of the inclinometer casing to confirm that it has been correctly installed.

Once the grout has set, the water level inside the casing shall be lowered to approximately 6 m below the ground to prevent freezing.

3.5 Protective Surround

A PVC pipe shall be placed around the inclinometer casing/pipe system to a slightly lower height. The internal diameter of the pipe and its couplings shall be such that the inclinometer casing is free to slide inside but without excessive play. (Note that the outside diameter of Slope Indicator QC casing is larger than 70 mm due to coupling alignment pins.)

A protective surround, consisting of a corrugated steel pipe and sand backfill, shall be placed around the portion of pipes that are above ground.

The above-ground portion of the inclinometer casing shall be greater than 0.3 m in length.

3.6 Extension of Inclinometer

As embankment/RSS wall construction proceeds, the inclinometer casing and the protective surround shall be extended so that they are always above the current ground level.

Each inclinometer casing shall be inspected by the Contract Administrator following completion of installation and before the start of embankment construction. The Contractor shall re-grout any casings that are found to be loose or where the grout has settled, at no cost to the Owner.

3.7 Protective Housing After Embankment Construction

Following completion of the embankment/RSS wall construction, the protective housing described elsewhere in this special provision shall be cemented in place around the inclinometer so as to remain secure and stable throughout the duration of the monitoring period.

4.0 COORDINATION WITH MONITORING

4.1 Notification and Reporting

The Contractor shall notify the Contract Administrator no later than three working days after installing Inclinometers (INC). At this time the Contractor shall also supply the following information to the Contract Administrator:

- Northing and easting of each Inclinometer in MTM NAD 83 coordinates;
- Elevation of the ground level and top of casing;
- Dates of installation;
- Magnetic and grid bearings of A+ and B+ groove directions;
- Difference between A-axis bearing and line parallel to Highway 400 centreline;
- Stratigraphic log of subsurface conditions at the Inclinometer locations;
- Installation notes and sketches, including socket details, casing depth, stick-up and telescopic sections, and grouting notes.

4.2 Monitoring

Monitoring of the Inclinometers shall be carried out by others under the Contract Administrator assignment. Monitoring shall be conducted during the embankment and RSS wall construction, throughout the preloading period, and following completion of the preloading period. The Contractor shall provide access to the Inclinometers for monitoring including, but not limited to, a scaffolding platform and ladder if required and snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed for reading the instruments.

5.0 REMOVAL/DECOMMISSIONING

After completion of the settlement monitoring period, the Inclinometers shall be decommissioned in accordance with Ontario Water Resources Act, Regulation 903 (as amended).

6.0 BASIS OF PAYMENT

Measurement for payment will be made on the basis of the number of units of Inclinometers (INC) installed, including extension through the fills, and then decommissioned following completion of the monitoring period.

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, equipment and materials to do the work, including all appurtenances, extension through the fills, the required reporting, and decommissioning.

SHAPE ACCEL ARRAYS – Item No.

Special Provision

1.0 SCOPE

This specification contains the requirements for the supply and installation of Shape Accel Arrays (SAA) and associated Earth Stations (data-logger systems with housing and accessories used to remotely collect data from each SAA).

The purpose of the Shape Accel Arrays is to allow remote monitoring of vertical displacements in the foundation soils under the embankment widening/RSS wall. The settlement readings shall help to establish the timing for completion of the preload period, and shall also be used to monitor the long-term behaviour of the foundation soils under the embankment widening, following completion of construction.

1.1 General Procedure

The Shape Accel Arrays shall be installed horizontally in a shallow trench after completion of the subexcavation and backfilling operation, prior to beginning the embankment widening/RSS wall construction. Each Shape Accel array shall extend from the existing Highway 400 embankment toe, perpendicularly away from the Highway 400 embankment/RSS wall to a fixed survey point located near the ditch beside Wist Road or Davis Road.

1.2 Location

The Contractor shall install Shape Accel Arrays at the approximate locations shown on the Contract Drawings and given in Table 1. The Shape Accel Arrays shall be installed with one end under the existing toe of the Highway 400 embankment, after excavating approximately 1 m to 2 m into the existing toe for installation of the trench and SAA. The Shape Accel Arrays will then extend perpendicularly away from the Highway 400 embankment to a fixed survey point located near the ditch beside Wist Road on the east side, and Davis Road on the west side of Highway 400. The fixed survey point shall be selected by the Contractor to be the maximum distance possible from the limit of the Highway 400 embankment widening/RSS wall, without crossing the drainage ditch adjacent to the local road, and in consideration of the Contractor's operations and access.

Table 1 – Shape Accel Array (SAA) Locations

Monitoring Section	Approx. Station (m)*	SAA Length (m)	Segment Length (m)
Hwy 400 NBL Sta 24+920 to 25+140	24+925	12	0.5
	25+025	11	0.5
Hwy 400 SBL	24+890	17	0.5
	24+940	14	0.5
TOTAL:	**4**		

* The actual location of the SAA in plan shall be selected in the field to avoid potential conflict with other monitoring instruments to be installed around this same station under the widened highway embankment shoulder.

** The Contractor shall provide an additional 5 m of SAA per location to allow for a longer installation than anticipated, should the trench excavation extend more than 1 m into the embankment toe and in order to reach a suitable fixed survey point as selected by the Contractor.

2.0 REFERENCES – not used

3.0 DEFINITIONS – not used

4.0 DESIGN AND SUBMISSION REQUIREMENTS – not used

5.0 MATERIALS

The Contractor shall supply all materials and equipment required for the installation of the Shape Accel Arrays, including Earth Station components.

5.1 Shape Accel Array System

The SAA system shall be an “SAAF” Field Array, for monitoring deformations in the field, as manufactured by Measurand Inc., with contact information as follows:

Measurand Inc.
2111 Hanwell Road, Fredericton, New Brunswick, Canada E3C 1M7

Contact: Christiane Levesque
Telephone: +1 506-462-9119
Email: christiane@measurand.ca

The SAA system shall be constructed to the total lengths shown in the Contract Drawings and specifications, and shall have individual segment lengths of 500 mm.

The Contractor shall supply sufficient cable to route from the reference end of each SAA to the Earth Station(s), compatible with the Measurand Inc. SAA. The cable shall be long enough to provide adequate strain relief.

The Contractor shall supply SAA splice kits manufactured by Measurand Inc., or ScotchCast Signal and Control Cable Inline Splicing Kit 72-N1 manufactured by 3M, for splicing SAA cables if and where this is required. Other splicing kits shall only be used with the SAA manufacturer’s approval.

The Contractor shall provide a five-year warranty for each Shape Accel Array system.

5.2 SAA Installation Trench

5.2.1 PVC Conduit

The Contractor shall supply PVC conduit for housing the SAA, with an inside diameter of 27 mm +1 mm/-0.5 mm. The outside diameter shall be 32 mm +/3 1 mm.

5.2.2 Bedding Sand

The Contractor shall supply bedding sand to be placed within the installation trench below and above the conduit containing the SAA. The bedding sand shall meet the material requirements for concrete fine aggregate (OPSS.PROV 1002).

5.2.3 Geotextile

The Contractor shall supply non-woven geotextile meeting the requirements of OPSS 1860 to line the SAA installation trench, with the geotextile dimensioned so that it can be wrapped over top of the bedding sand fill.

5.3 Earth Station (for remote data collection)

5.3.1 Enclosure

The Contractor shall supply a National Electrical Manufacturers Association (NEMA) 4 rated enclosure to house the Earth Station components.

5.3.2 Logger

The Contractor shall supply a CR800, CR1000 or CR3000 logger, manufactured by Campbell Scientific Inc., or equal, for collecting data from the SAA system.

The Contractor shall supply either SAA232 or SAA232-5 logger interface modules to connect the SAA systems to the logger communication ports. Only one interface shall be connected per logger communication port.

5.3.3 Power Supply

The Contractor shall supply a 12 V, 100 Ah deep-cycle absorbed glass mat (AGM) battery to supply power for the logger and SAA system. The battery shall be housed in a separate NEMA 3R rated enclosure.

The Contractor shall supply a solar panel not exceeding 50W of rated power, to charge the battery for the Earth Station, and a 12 V regulator to control battery charging via the solar panel.

The Contractor shall provide a five-year warranty for the power supply system(s).

5.3.4 Communications

The Contractor shall supply a cell network modem to provide a remote communication interface. The modem shall be a CDMA or GPRS type modem with an antenna sufficient to achieve average communication rates of 57 kilobytes per second.

The Contractor shall provide for five years (60 months) of cellular network coverage, from the time of installation of the SAA and Earth Station.

5.3.5 Steel Post

The Contractor shall supply a 50 mm galvanized steel pipe for mounting the Earth Station. The pipe shall be installed below frost depth and extend to at least 2.5 m above the ground surface.

6.0 EQUIPMENT – not used

7.0 CONSTRUCTION

The Contractor shall install Shape Accel Arrays per the manufacturer's handling and installation recommendations, in addition to what is stated or emphasized below.

7.1 Instrument and Conduit Assembly

The PVC conduit shall be assembled in a generally flat area using PVC cement suitable for the temperature and weather conditions.

The SAA reel shall be placed on a reel stand with a minimum height of 0.6 m, and such that the SAA will be pulled from the bottom of the reel.

The SAA shall be pulled into the conduit using a rope or a cable with swivel attachment to eliminate twisting of the SAA.

The end cap shall be glued onto the "existing embankment toe" end of the conduit, at the eyebolt end of the SAA.

The PEX at the cable end of the SAA shall be secured to the conduit using the set-screw assembly provided in the Manufacturer's SAA installation kit.

7.2 Horizontal Installation in Trench

The SAA and PVC conduit assembly shall be installed into a trench that is no less than 0.3 m deep by 0.3 m wide. The trench shall be extended a minimum of 1 m into the toe of the existing Highway 400 embankment, following completion of the subexcavation and backfilling operation, and prior to commencement of any embankment widening/RSS wall construction operations. The distal end of the trench shall extend to the fixed survey point, such that the trench is constructed perpendicular to the Highway 400 embankment.

The trench shall be lined with geotextile to provide separation between the existing soil/subexcavation backfill, and the bedding sand fill.

A layer of bedding sand shall be placed in the trench above the geotextile. This layer shall be at least 150 mm thick, or the thickness of the largest particle size in the common fill that will be placed above the SAA and PVC conduit, whichever is bigger.

The SAA and PVC conduit assembly shall be placed into the trench on top of the base layer of bedding sand. The reference (cable) end of the SAA shall be attached to the fixed survey point.

A layer of bedding sand shall be placed above the SAA and PVC conduit assembly. This layer shall be at least 150 mm thick, or the thickness shall correspond to the size of the largest particle in the fill that is being placed above the SAA and PVC conduit, whichever is larger.

The geotextile shall be wrapped over top of the bedding sand, such that there is at least 150 mm overlap in the geotextile.

The trench shall then be filled using Granular A or Granular B Type II material (OPSS.PROV 1010).

7.3 Fixed Survey Points

A fixed survey point shall be installed at the reference end of each SAA and PVC conduit. The fixed survey point shall consist of deep, non-settling temporary benchmarks. The approximate elevation for the bottom of each anchor is provided in Table 2.

Table 2 – Fixed Survey Point Locations and Elevations

Approx. Station	Approx. Offset from CL (m)	Approx. Elevation of Bottom of Anchor (m)*	Approx. Length of Rod Incl. Stick-Up (m)*
Hwy 400 NBL 24+925	37 m east	205	15
Hwy 400 NBL 25+025	37 m east	200	20
Hwy 400 SBL 24+890	45 m west	205	15
Hwy 400 SBL 25+050	42 m west	200	20

* The rod anchor elevation is approximate and should be adjusted in the field to extend approximately 1 m to 2 m into soils having Standard Penetration Test "N" values of greater than 50 blows per 0.3 m of penetration.

7.3.1 Materials

The Contractor shall supply all materials and equipment required for the installation of the fixed survey points (Benchmarks).

7.3.1.1 Rod

The Contractor shall supply a steel pipe, Schedule 40, with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Table 2.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

7.3.1.2 Sand

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

7.3.1.3 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 23 kg of bentonite (OPSS 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

7.3.1.4 Rod Anchor Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall consist of 14 kg of bentonite (OPSS 1205), 49 litres of water and 40 kg of cement (Type GU – OPSS 1301).

7.3.1.5 Friction-Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

7.3.2 Installation

The Contractor shall install Benchmarks in accordance with the following:

7.3.2.1 Borehole

The borehole shall be advanced to the rod anchor elevations provided in Table 2 using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction-reducing sleeve and rod anchor. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

7.3.2.2 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.3.2.3 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole to form a concrete/soil anchor.

Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 0.5 m length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

7.3.2.4 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod above the rod anchor and sand, extending up to ground surface.

8.0 QUALITY ASSURANCE

8.1 Development of Web-Based Monitoring Service

The Contractor shall provide and maintain a web-based monitoring service for the Shape Accel Array (SAA) monitoring data throughout the monitoring period under the CA assignment and thereafter following transfer of the SAAs to MTO Foundations Section, for a total duration of five years.

The minimum requirements for the web-based monitoring application shall be as follows:

- The application shall upload the SAA data in raw and/or converted form, and provide any application software necessary to read native files and formats, including a license for application software appropriate for web-based use.
- The application shall integrate the automatic/electronically-collected SAA instrument data with the manually collected survey data at the fixed survey points at the reference end of the SAAs.
- The application shall have secure, password-protected access allowing user access for the Contract Administrator and their Foundation Monitoring consultant, the Contractor, and the MTO.
- The application shall be capable of updating graphs as data becomes available, and shall allow users to graph all data or to allow for comparison of selected monitoring points.
    - The application shall be capable of plotting the following at user-selectable scales:
    - Settlement along the length of each SAA for each monitoring event
    - Settlement versus linear and log time for user-selected points along the length of each SAA, at a minimum corresponding to the following points, and also showing the review and alert levels:
        - The crest of the widened Highway 400 embankment;
        - The mid-point of the widened Highway 400 embankment;
        - The toe of the widened Highway 400 embankment; and
        - The point of maximum settlement along the SAA.
    - Fill height versus time (based on manual input to be entered by others).
    - The application shall provide a comment field tied to each reading that is editable by authorized users.
    - The application shall provide the ability to print reports.

8.2 Coordination With Monitoring

8.2.1 Testing

Each SAA system set-up shall be verified prior to hand-over to the Contract Administrator, in accordance with the requirements of the manufacturer and supplier of the SAA system and loggers.

8.2.2 Notification and Reporting

The Contractor shall notify the Contract Administrator no later than three working days after installing Shape Accel Arrays (SAA). At this time the Contractor shall also supply the following information to the Contract Administrator:

- Northing and easting of the fixed survey point and the end of the SAA/PVC conduit at the existing embankment toe, in MTM NAD 83 coordinates;
- Elevation of the reference end of the SAA and the fixed survey point, referenced to geodetic datum;
- Dates of installation;
- Installation notes and sketches, including the instrument, cable and conduit lengths, installation depth, azimuth direction of X-marks, and azimuth corrections for software;
- Manufacturer calibration sheets and instrument serial numbers; and
- Access and any required licencing for the web-based SAA monitoring application.

8.2.3 Monitoring

Monitoring of the Shape Accel Arrays shall be carried out remotely by others under the Contract Administrator assignment. Monitoring shall be conducted during the embankment and RSS wall construction, throughout the preloading period, and for a six-month period following completion of the preloading period. After this time, long-term remote monitoring of the SAA installations shall be turned over by the Contract Administrator to the Foundations Section of the Ministry of Transportation Ontario; the SAAs and associated Earth Stations (including power supply and communications) shall therefore remain in place for a total period of five years, as outlined in Section 2 of this special provision.

Although the SAAs will be monitored remotely, the Contractor shall provide access to the fixed survey points at the end of the SAAs for surveying purposes, including snow clearing in the winter if required during the periods of embankment and RSS wall construction, preloading, and for six months following preloading.

9.0 MEASUREMENT FOR PAYMENT

Measurement for payment on the number of units of Shape Accel Arrays (SAA) installed shall be by each, as may be revised by Adjusted Plan Quantity.

10.0 BASIS OF PAYMENT

Payment at the contract price for this tender item shall be full compensation for all labour, equipment and materials required to do the work including the associated Earth Stations with power supply, five years of remote communications support, and development of the web-based application for importing and viewing the monitoring data.

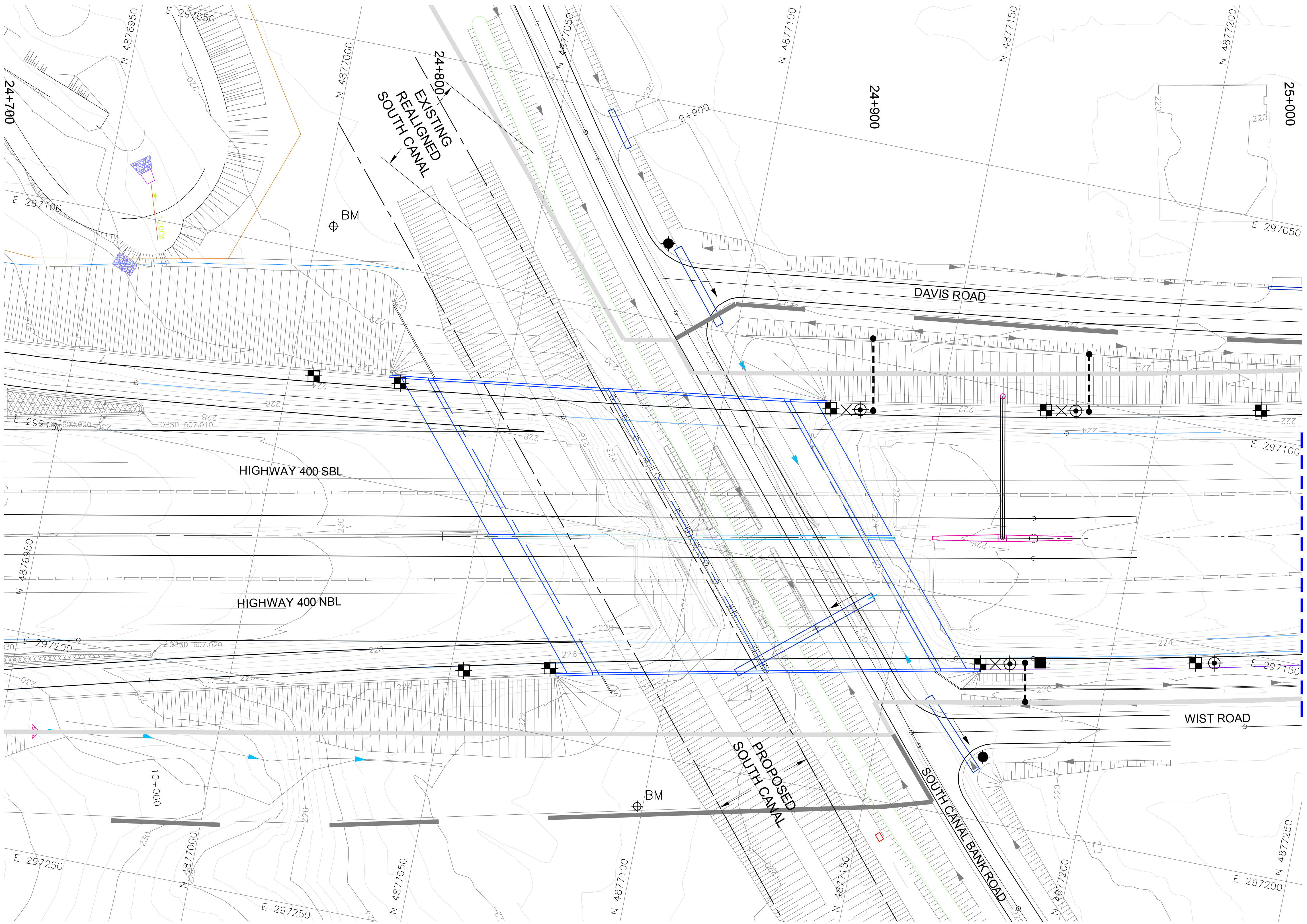
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2015–2004
GWP No. 2025–13–00

HIGHWAY 400 WIDENING
STA 24+700 TO STA 25+000
MONITORING INSTRUMENTATION PLAN

SHEET
531

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

SCALE
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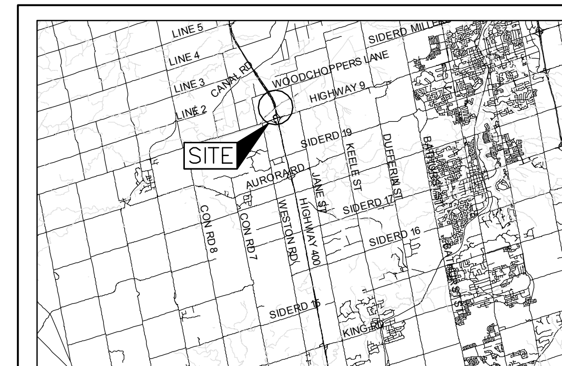


KEY PLAN
SCALE
4 0 4 8 km

LEGEND

- Survey Benchmark (BM)
- Settlement Plate and Settlement Pin (SP/S)
- Vibrating Wire Piezometer (VWP)
- Standpipe Piezometer (SSP)
- Settlement Profiler (PR)
- Inclinometer (INC)
- Shape Accel Array (SAA)

NOTE

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

REFERENCE

Base plans and General Arrangement provided in digital format by URS Canada Inc., (Drawing Files "Hwy400_plan.dwg" and "01_GA_July 10 2012.dwg") received November 13, 2013 and September 26, 2012.

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Mar. 10, 2015 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA Mar. 10, 2015 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No.			
HWY. 400	PROJECT NO. 09–1111–0018		DIST. CENTRAL
SUBM'D. MSD	CHKD. LCC	DATE: Mar. 2015	SITE:
DRAWN: JFC	CHKD. LCC	APPD. JMAC	DWG. 1

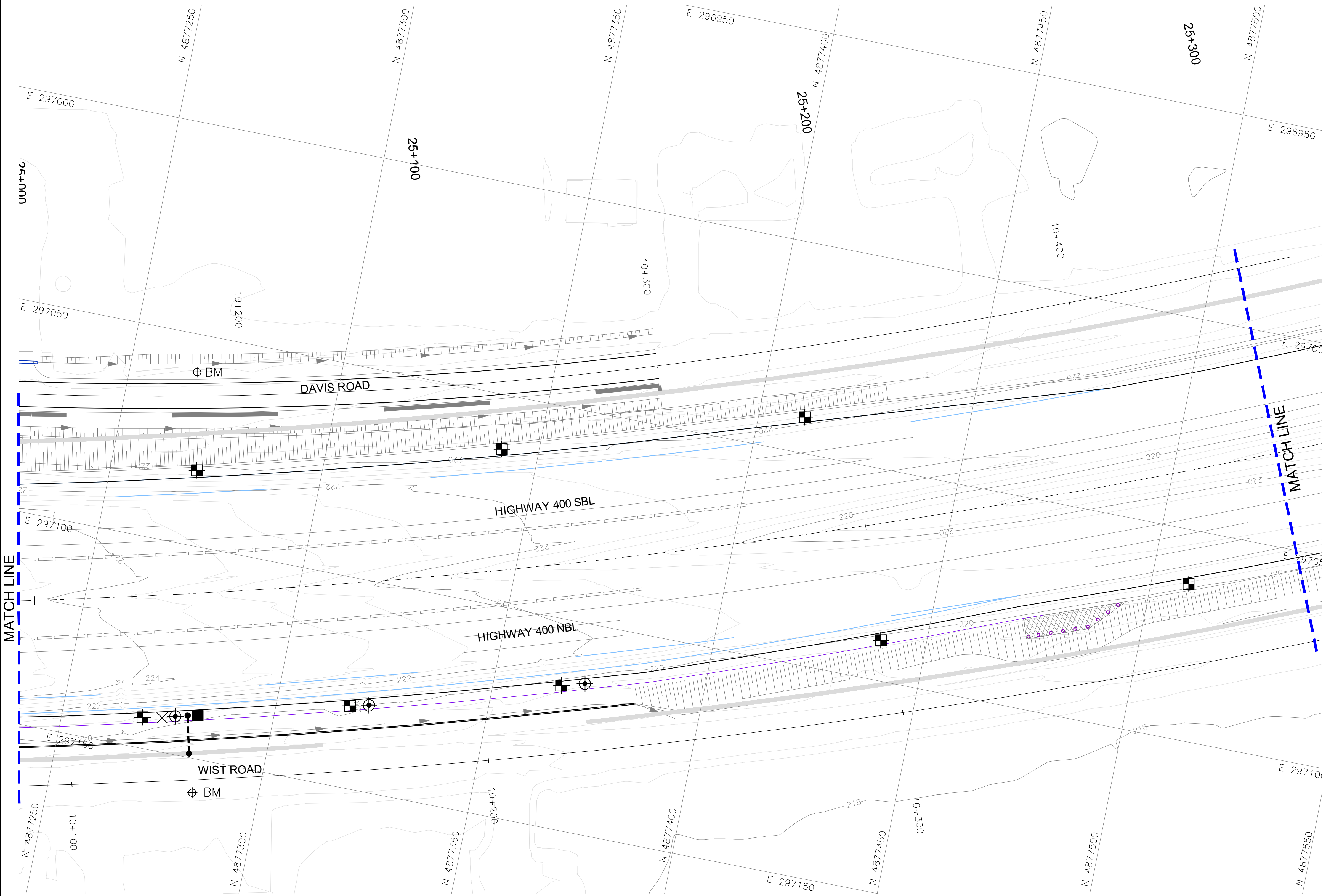
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2015–2004
GWP No. 2025–13–00

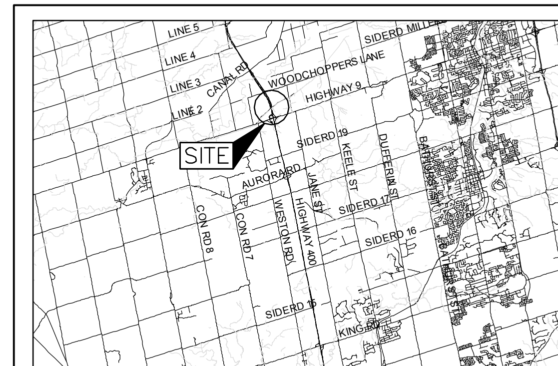
HIGHWAY 400 WIDENING
STA 25+000 TO STA 25+300
MONITORING INSTRUMENTATION PLAN

SHEET 532

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
10 0 10 20 m



KEY PLAN
SCALE
4 0 4 8 km

LEGEND

⌖	Survey Benchmark (BM)
	Settlement Plate and Settlement Pin (SP/S)
	Vibrating Wire Piezometer (VWP)
X	Settlement Profiler (PR)
■	Inclinometer (INC)
	Shape Accel Array (SAA)

NOTE

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NO.	DATE	BY	REVISION

Geocres No.

HWY. 400	PROJECT NO. 09–1111–0018		DIST. CENTRAL
SUBM'D. MSD	CHKD. LCC	DATE: Mar. 2015	SITE:
DRAWN: JFC	CHKD. LCC	APPD. JMAC	DWG. 2

PLOT DATE: March 10, 2015
FILENAME: T:\Projects\2009\09-1111-0018 (URS, York Region)\--UA-- Instrumentation\0911110018UA002.dwg

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

CONT No. 2015–2004
GWP No. 2025–13–00

SHEET 533

HIGHWAY 400 WIDENING

STA 25+300 TO STA 25+550

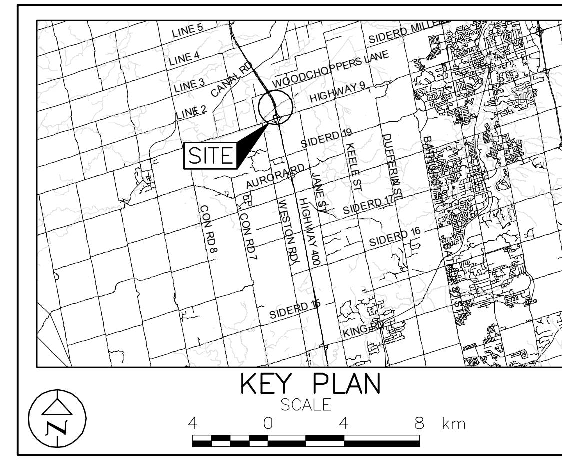
MONITORING INSTRUMENTATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

PLAN

SCALE 10 0 10 20 m



KEY PLAN

SCALE 4 0 4 8 km

LEGEND

⌖	Survey Benchmark (BM)
▚	Settlement Plate and Settlement Pin (SP/S)

NOTE

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

REFERENCE

Base plans and General Arrangement provided in digital format by URS Canada Inc., (Drawing Files "Hwy400_plan.dwg" and "01_GA_July 10 2012.dwg") received November 13, 2013 and September 26, 2012.

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Mar. 10, 2015 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA Mar. 10, 2015 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No.

HWY. 400	PROJECT NO. 09-1111-0018		DIST. CENTRAL
SUBM'D. MSD	CHKD. LCC	DATE: Mar. 2015	SITE:
DRAWN: JFC	CHKD. LCC	APPD. JMAC	DWG. 3

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

CONT No. 2015–2004
GWP No. 2025–13–00

HIGHWAY 400 WIDENING
STA 25+550 TO STA 25+800
MONITORING INSTRUMENTATION PLAN

SHEET 534

Golder Associates
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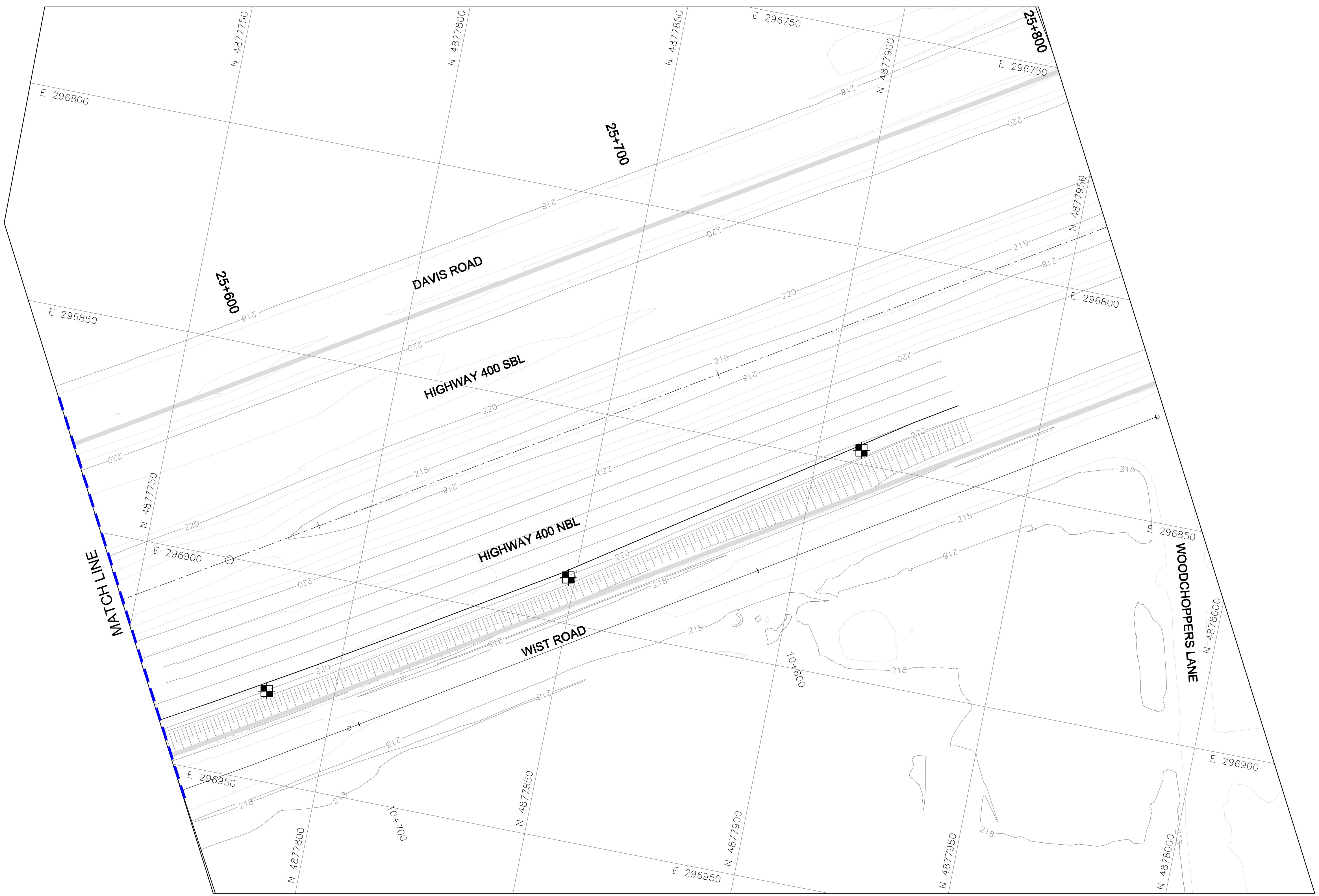
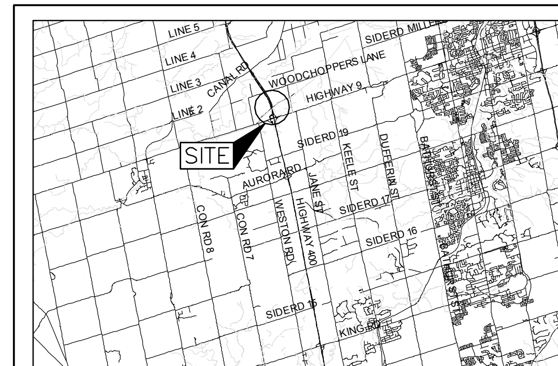


PLAN
SCALE
10 0 10 20 m



KEY PLAN
SCALE
4 0 4 8 km

LEGEND

Settlement Plate and Settlement Pin (SP/S)

NOTE

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

REFERENCE

Base plans and General Arrangement provided in digital format by URS Canada Inc., (Drawing Files "Hwy400_plan.dwg" and "01_GA_July 10 2012.dwg") received November 13, 2013 and September 26, 2012.

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Mar. 10, 2015 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA Mar. 10, 2015 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No.

HWY. 400		PROJECT NO. 09-1111-0018	DIST. CENTRAL
SUBM'D. MSD	CHKD. LCC	DATE: Mar. 2015	SITE:
DRAWN: JFC	CHKD. LCC	APPD. JMAC	DWG. 4

PLOT DATE: March 10, 2015
FILENAME: T:\Projects\2009\09-1111-0018 (URS, York Region)\--UA-- Instrumentation\09111110018UA004.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

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

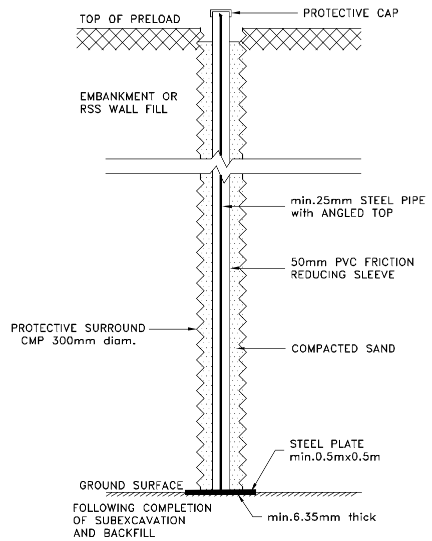
CONT No. 2015–2004
GWP No. 2025–13–00

HIGHWAY 400 WIDENING
TYPICAL MONITORING AND INSTRUMENTATION INSTALLATION DETAILS

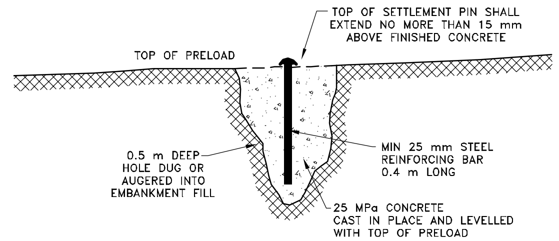
SHEET 535

Golder Associates

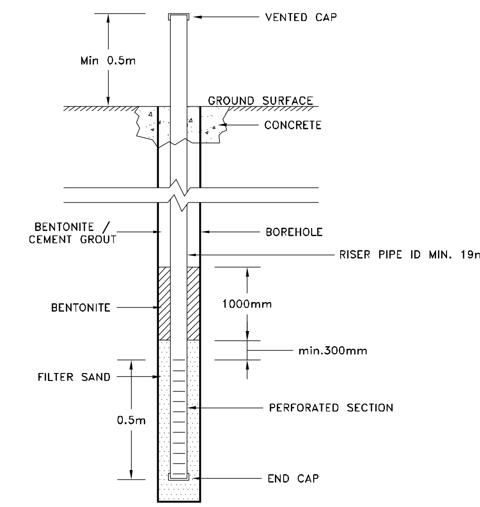
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



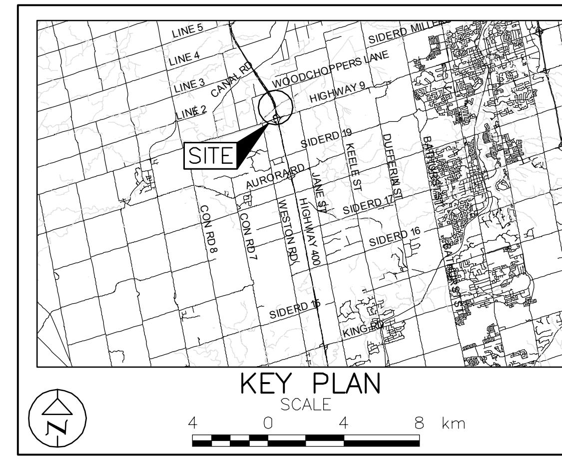
SETTLEMENT PLATE (SP)
NOT TO SCALE



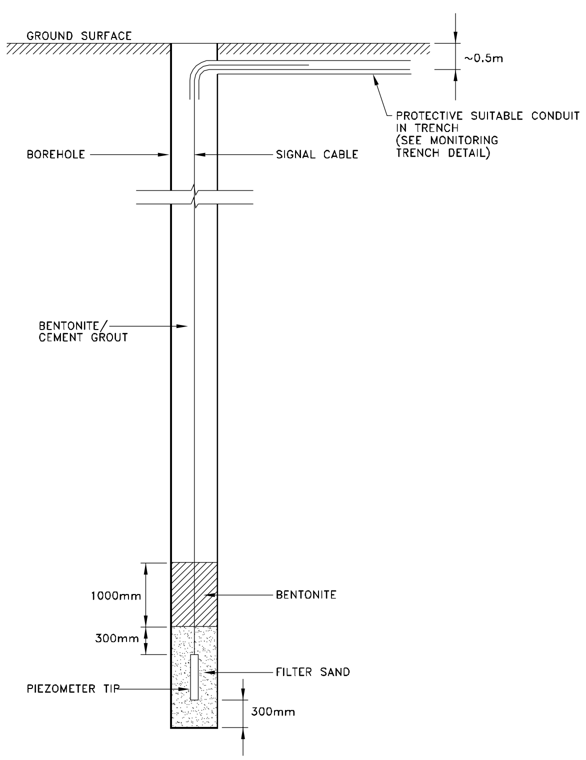
SETTLEMENT PIN (S)
NOT TO SCALE



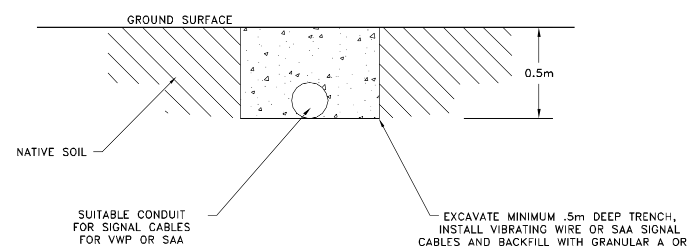
STANDPIPE PIEZOMETER (SSP)
NOT TO SCALE



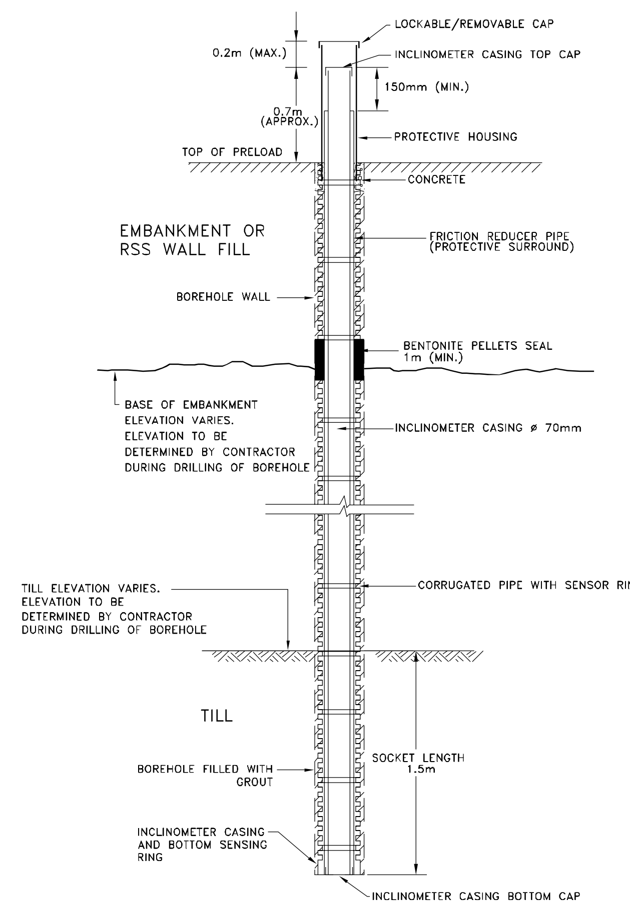
KEY PLAN
SCALE
4 0 4 8 km



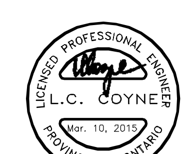
VIBRATING WIRE PIEZOMETER (VWP)
NOT TO SCALE

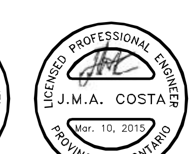


MONITORING TRENCH
NOT TO SCALE



SETTLEMENT PROFILER (PR)
NOT TO SCALE





NO.	DATE	BY	REVISION

Geocres No.

HWY. 400		PROJECT NO. 09–1111–0018	DIST. CENTRAL
SUBM'D. MSD	CHKD. LCC	DATE: Mar. 2015	SITE:
DRAWN: JFC	CHKD. LCC	APPD. JMAC	DWG. 5

PLOT DATE: March 11, 2015
FILENAME: T:\Projects\2009\09-1111-0018 (URS, York Region)\--UA-- Instrumentation\09111100018UA005.dwg

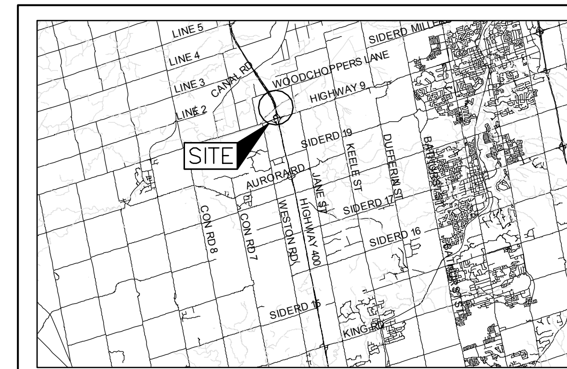
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2015–2004
GWP No. 2025–13–00

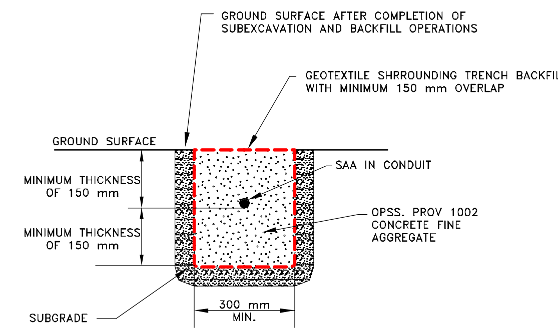
HIGHWAY 400 WIDENING
TYPICAL MONITORING AND INSTRUMENTATION INSTALLATION DETAILS

SHEET 536

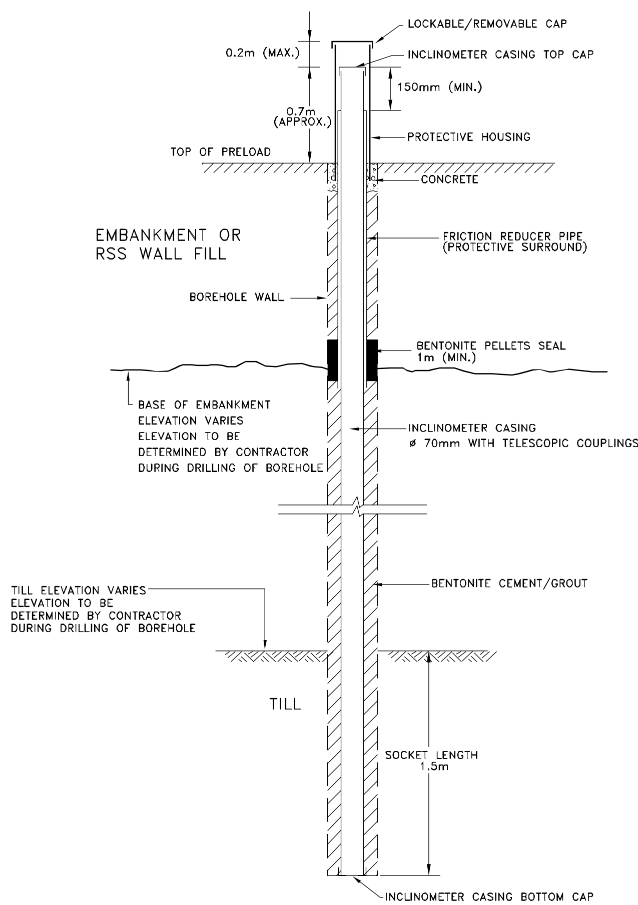
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



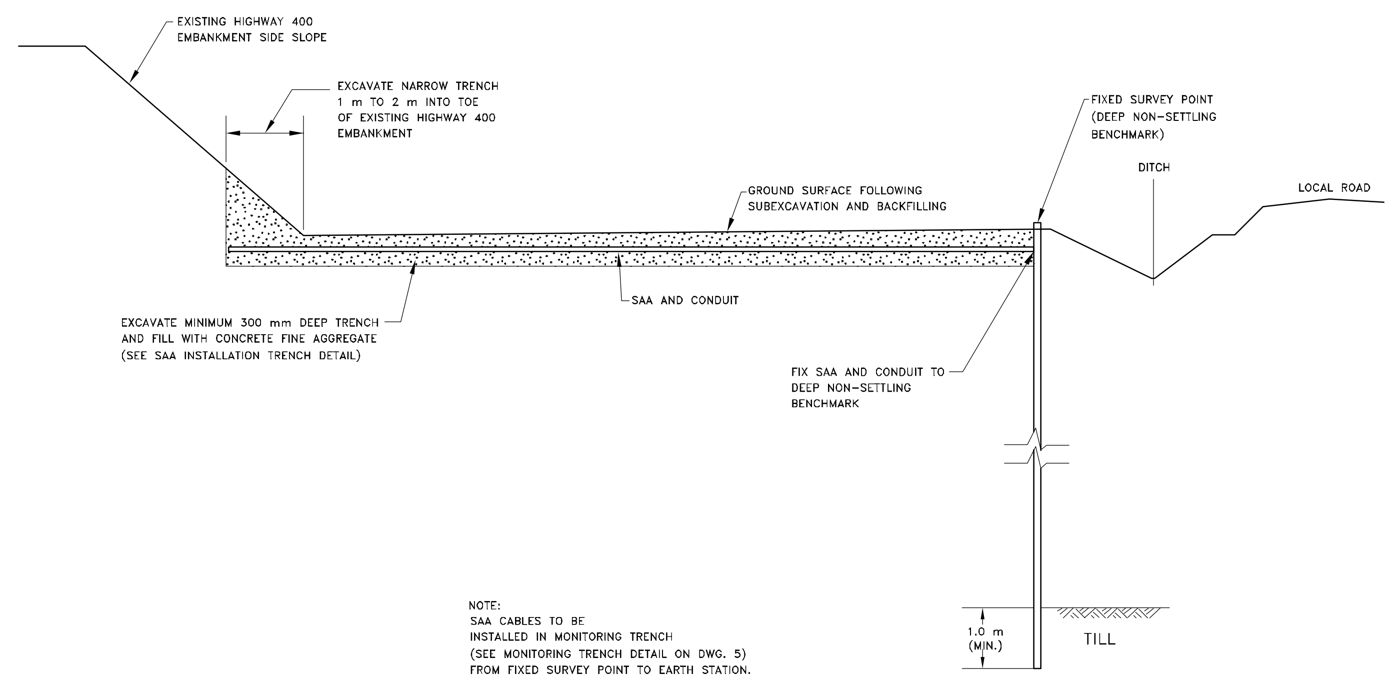
KEY PLAN
SCALE
4 0 4 8 km



SAA INSTALLATION TRENCH DETAIL
NOT TO SCALE

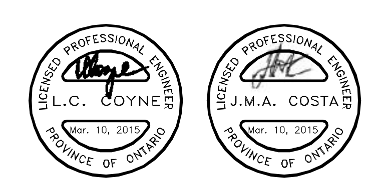


INCLINOMETER (INC)
NOT TO SCALE



NOTE:
SAA CABLES TO BE INSTALLED IN MONITORING TRENCH (SEE MONITORING TRENCH DETAIL ON DWG. 5) FROM FIXED SURVEY POINT TO EARTH STATION.

SHAPE ACCEL ARRAY (SAA)
NOT TO SCALE



NO.	DATE	BY	REVISION

Geocres No.			
HWY. 400		PROJECT NO. 09–1111–0018	DIST. CENTRAL
SUBM'D. MSD	CHKD. LCC	DATE: Mar. 2015	SITE:
DRAWN: JFC	CHKD. LCC	APPD. JMAC	DWG. 6

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. 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 who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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